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Managing Water in the West

Hydrology, Hydraulics and Sediment Studies of Alternatives for the Matilija Dam Ecosystem Restoration Project, Ventura, CA – Final Report

US Department of the Interior
Bureau of Reclamation
Technical Service Center

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Photo on cover courtesy of Paul Jenkin, Chairman of Surfriders Foundation, Ventura, CA

Executive Summary

The Bureau of Reclamation (Reclamation) is providing technical assistance in the Matilija Dam Ecosystem Restoration Feasibility Study -- a cost-shared study between the Corps of Engineers (Corps) and Ventura County Flood Control District (District). In addition to geotechnical, surveying, and mapping tasks, the District requested Reclamation to perform the hydrology, hydraulics, and sedimentation analyses as in-kind credit in this feasibility study. Technical assistance by Reclamation has been funded through an Interagency Agreement with the State Coastal Conservancy. Work elements associated with this task are consistent with items delineated in the Corps Project Management Plan (PMP). To ensure successful achievement of certain items described in the PMP, Reclamation requested assistance from U.S. Geological Surveys (USGS) that will complement investigations and provide deliverables in 2003. This report is the final submittal of the results from the hydrology, hydraulics, and sedimentation studies supporting the final feasibility report at the F8 milestone for the Matilija Dam Ecosystem Restoration Study, Ventura, California. The information contained herein will be used in hydrologic, hydraulic, and sedimentation modeling to evaluate the impacts of various alternatives of restoring the ecosystem.

Matilija Dam is located on Matilija Creek, which joins with North Fork Matilija Creek ½ mile downstream of the dam to form the Ventura River. The Ventura River is predominantly a cobble bed river with a high sediment supply. The sediment production per area from the Ventura River watershed is one of the highest in the nation, at about 1 mm/yr. Sediment transport in the rivers and stream are dominated by large infrequent floods. According to sediment measurements, over 98% of the sediment transport in the Ventura River occurs in less than 1% of the time.

Matilija Dam was constructed in 1947 and has trapped approximately 5.9 million yd^3 of sediment since 1947. Because of the large volume of trapped sediment, the major costs and impacts of the removal of Matilija Dam are primarily those associated with the management of this trapped sediment. There are four major alternatives being considered, and three of those have sub-alternatives:

Alternative	Sub-Alternative	Description
No Action		No removal of dam or sediments
1		Full Dam Removal/Mechanical Sediment Transport: Dispose Fines, Sell Aggregate
2		Full Dam Removal/Natural Sediment Transport
	2a	Slurry "Reservoir Area" Fines Offsite
	2b	Natural Transport of "Reservoir Area" Fines
3		Incremental Dam Removal/Natural Sediment Transport
	3a	Slurry "Reservoir Area" Fines Offsite
	3b	Natural Transport of "Reservoir Area" Fines
4		Full Dam Removal/Sediment Stabilization on Site
	4a	Permanent Stabilization
	4b	Temporary Stabilization

The trapped sediment can be divided into 3 areas: the reservoir area, the delta area, and the upstream channel area. The reservoir area consists primarily of silt and clay sized sediment. The delta is primarily sand and gravel with some silt and the upstream channel is primarily sand, gravel, and cobble. Each alternative has different methods to remove or stabilize each of these sediment areas. Therefore, each alternative will have different impacts associated with it.

The No Action Alternative would cause continued deposition behind Matilija Dam. The reservoir capacity would be expected to be 150 ac-ft in 2010 and less than 50 ac-ft by 2020. At its current capacity of 500 ac-ft, it is estimated to increase the annual diversion by approximately 590 ac-ft of water at Robles Diversion. Presently, the majority of the silt and clay entering the Matilija Reservoir passes over the top of Matilija Dam. However, the dam is continuing to trap sand and larger sizes. Because the dam is continuing to trap coarse sediment, there would be some continued degradation in the reaches downstream of the dam until the Live Oaks Levee. The expected degradation in 50 years would vary from 1 to 3 feet in the reaches from Robles Diversion to Live Oaks Levee. In approximately 40 years, sand and gravel sized sediment would start to pass over the dam crest at large flows, at which time it is estimated that over 9 million yd^3 of sediment would be stored behind the dam. The sediment loads downstream of the dam would then increase. The result would be a slow aggradation or at least a slowing of the degradation process in the reaches immediately below the dam and an increase of deposition that occurs in Robles Diversion area. It is expected that in approximately 100 years, the Ventura River would be in approximate equilibrium, meaning that sediment load entering the river system would be in approximate balance with the sediment load exiting the system. The approximately 2.2 million yd^3 of sand that is presently trapped behind the dam would not be supplied to the beach and approximately an additional 2 million yd^3 of sand would be trapped behind the dam in the next 40 years, for a total of 4.2 million yd^3 of sand stored behind the dam. There are current flood concerns along the Ventura River. Several residences downstream of Robles Diversion may be at risk of flooding during a 100-yr flood. In addition, the levee along the Ventura River at the town of Casitas does not provide protection against the 100-yr flood. Some deposition would be expected in the Casitas Levee Reach, which may increase the flood risk. Approximately 2 feet of deposition would be expected in this reach over the next 50 years.

The Full Dam Removal/Mechanical Sediment Transport: Dispose Fines, Sell Aggregate Alternative (Alternative 1) would removal all the sediment stored behind Matilija Dam from the river system. There would be a natural re-supply of Matilija Creek Sediment to the downstream reaches. This natural re-supply of sediment would have noticeable impact on reaches located between Matilija Dam and Baldwin Road with the greatest impact near the dam and Robles Diversion. The reach below Robles Diversion would cease to degrade and may start to aggrade. The aggradation in the reach below Robles Diversion would raise water surface elevations and require that a small levee be built to protect some of the residences in the town of Meiners Oak. Because of the re-supply of Matilija Creek sediment, the deposition at Robles Diversion may increase by approximately a factor of two. This would increase maintenance costs and perhaps necessitate a re-design of Robles Diversion that reduces the amount of deposition at the site. A sediment bypass structure that lowers the elevation of the diversion during high flows would decrease the amount of sediment excavation required at Robles Diversion and reduce the risk of lost water supply. Silt and clay concentrations in the Ventura River would not be significantly different from the No Action Alternative.

The Full Dam Removal/Natural Sediment Transport alternative (Alternative 2) is spilt into two sub-alternatives: Slurry "Reservoir Area" Fines Offsite (Alternative 2a) and Natural Transport of "Reservoir Area" Fines (Alternative 2b). The main difference between the two alternatives is the management of the "Reservoir Area" Fines. In Alternative 2a, the "Reservoir Area" Fines would be mechanically transported by slurry line to a disposal site located on the floodplain of the Ventura River downstream of the Miners Oak community. In Alternative 2b, the "Reservoir Area" Fines would be transported downstream by the natural river flow. The "Reservoir Area" Fines consist of 30% clay, 53% silt and 17% fine sand and have a volume of 2.1 million yd^3.

The Full Dam Removal/Natural Sediment Transport: Slurry "Reservoir Area" Fines Offsite Alternative (Alternative 2a) uses the natural flows to erode the delta and the upstream channel. The delta is composed of approximately 13% gravel, 54% sand, 28% silt and 5% clay and the upstream channel is composed of approximately 39% cobbles, 39% gravel, 16% sand and 6% silt. When flow starts to erode this material, first a narrow deep channel would be created through the material, followed by gradually widening of the channel through the delta deposits. The rate of widening would be dependent upon the flow rate: the larger the flood, the more material removed and the wider the channel through the delta. Because the amount of silt and clay is small in the delta, the turbidity impact would be of relatively short duration. It would be expected that after the first flood, the turbidity levels would be no more than twice the natural levels. Because the dam would be removed in one-notch in this alternative, all the sediment would be immediately available for transport. There would be approximately 3.9 million yd^3 of material available for transport in this alternative and some of this material would deposit in the upper reaches of the Ventura River. There is considerable uncertainty regarding the deposition downstream of the dam and therefore the levee and floodwall design would be necessarily conservative. Large amounts of sediment would deposit in the area impounded by Robles Diversion Dam. During the first few floods, sediment eroded from the reservoir would fill the diversion until it starts to spill over the diversion dam crest. Re-designing the diversion dam by including a sediment bypass or similar structure would reduce the deposition at the site to acceptable levels.

The Full Dam Removal/Natural Sediment Transport: Natural Transport of "Reservoir Area" Fines Alternative (Alternative 2b) removes the dam all at once and allows natural flows to erode all the sediment stored behind Matilija Dam. The initial erosion would take place vertically and cut a deep channel through the reservoir sediments. The concentration of fine sediment downstream of the dam would be very high, greater than 100,000 mg/l, for a period of days to weeks. After this initial formation of a channel through the reservoir deposits, the flow would begin to cut a deep narrow channel through the delta deposits. When the flow rate increases during a flood, the channel through the delta would become much wider and a significant amount of sands, gravels, and cobbles material would be removed from the delta. The first two to three floods would carry very high sediment loads downstream. The concentration of fine material would decrease after each flood and would be expected to be at natural levels after three floods occur that are as large as the average annual flood. The deposition impacts in the upper reaches of the Ventura River would be large and the deposition elevations are uncertain. Therefore, large levees and floodwalls would be required to provide adequate flood protection. Large amounts of sediment would deposit in the area impounded by Robles Diversion Dam and similar mitigation measures as mentioned in Alternative 2a would be required. In addition to the deposition impacts at Robles, the turbidity impacts would last much longer than in Alternative 2a. Mitigation measures such as a settling basin or alternate sources of water may be necessary to reduce the impact of fine material on Casitas Reservoir.

The Incremental Dam Removal/Natural Sediment Transport: Slurry “Reservoir Area” Fines Offsite Alternative (Alternative 3a) removes only a portion of the dam at first. A flood would be allowed to erode the sediment stored behind the dam and then the remainder of the dam would be removed. This alternative has similar impacts to Alternative 2a, but there would be a greater measure of control of the deposition impacts. If, for example, more deposition than expected occurred at a particular location after the first stage of removal, it would be possible to mechanically remove that sediment from the stream channel or raise levees in that area before the second notch would be started. Therefore, this alternative has a much-reduced risk over Alternative 2a. The levees constructed for this alternative can be smaller than those for Alternatives 2a and 2b because sediment would be released more slowly and causes less downstream aggradation. However, if the region is experiencing severe drought conditions, up to 7 years may pass between the first notch and the second.

The Incremental Dam Removal/Natural Sediment Transport: Natural Transport of “Reservoir Area” Fines Alternative (Alternative 3b) again has similar impacts to Alternative 2b, but the risk to water supply and to flooding would be less. The levees may not have to be constructed as high because the sediment would be eroded from the reservoir more slowly. The turbidity impacts would also be extended over a longer period because new fines would be exposed after each stage of removal. If the region is experiencing severe drought conditions, up to 7 years may pass between the first notch and the second.

The Full Dam Removal/Sediment Stabilization on Site: Permanent Stabilization (Alternative 4a) removes all the sediment storage behind Matilija Dam from the Ventura River System. The sediment would be either mechanically removed or permanently stabilized. Therefore, the downstream impacts associated with this alternative would be practically identical to the Full Dam Removal/Mechanical Sediment Transport: Dispose Fines, Sell Aggregate Alternative (Alternative 1).

The Full Dam Removal/Temporary Sediment Stabilization on Site: Temporary stabilization of sediment (Alternative 4b) requires that a *temporarily* stable channel be constructed through the trapped sediments. The reservoir material would be removed by hydraulic dredge and transported by slurry line to a downstream disposal site. The channel design would allow the low flows to pass through the area of the trapped sediments without eroding excessive amounts of sediment. The revetment that stabilizes the sediment would be removed in stages. After each stage, floods would erode some of the exposed sediment and transport it downstream. The turbidity impacts should be confined to the high flow events during which the sediment would be allowed to erode. The deposition impacts in the downstream river channel associated with this alternative would be less severe than Alternative 2a.

Technical Summary

The Bureau of Reclamation (Reclamation) is providing technical assistance in the Matilija Dam Ecosystem Restoration Feasibility Study -- a cost-shared study between the Corps of Engineers (Corps) and Ventura County Flood Control District (District). In addition to geotechnical, surveying, and mapping tasks, the District requested Reclamation to perform the hydrology, hydraulics, and sedimentation analyses as in-kind credit in this feasibility study. Technical assistance by Reclamation has been funded through an Interagency Agreement with the State Coastal Conservancy. Work elements associated with this task are consistent with items delineated in the Corps Project Management Plan (PMP). To ensure successful achievement of certain items described in the PMP, Reclamation requested assistance from U.S. Geological Surveys (USGS) that will complement investigations and provide deliverables in 2003. This report is the final submittal of the results from the hydrology, hydraulics, and sedimentation studies supporting the final feasibility report at the F8 milestone for the Matilija Dam Ecosystem Restoration Study, Ventura, California. The information contained herein will be used in hydrologic, hydraulic, and sedimentation modeling to evaluate the impacts of various alternatives of restoring the ecosystem. This Technical Summary briefly describes the watershed context in which this project occurs and then following this, summarizes the impacts associated with each alternative.

Watershed Description

The Ventura River Watershed is shown in Figure 1. Section 25 titled “Exhibit M. Location of Cross Section Used in Study” near the end of this report contains a larger map with the River Mile (RM) indicated on the map. The Ventura River starts at the confluence of Matilija Creek and North Fork Matilija Creek, approximately 0.6 miles downstream of Matilija Dam. Several smaller watersheds enter the Ventura River upstream of the next major tributary, San Antonio Creek. Coyote Creek then enters Ventura River from the west just downstream of the confluence with San Antonio Creek. Casitas Dam regulates the flows on Coyote Creek. Downstream, Cañada Larga enters from the east and Cañada de Rodriguez and Cañada del Diablo enter from the west. Over 75% of the Ventura River Watershed is classified as rangeland covered with shrub and brush and 20% of the watershed is classified as forested. In general, the highest sediment producing parts of the watershed are those covered in shrub and brush and are located in the upper parts of the watershed where slopes are greater and annual rainfall is larger. Nearly 45% of the watershed may be classified as mountainous, 40% as foothill, and 15% as valley area (Reclamation, 1954).

For the purposes of this study, reaches have been defined so that, within a given reach, the river and associated habitat has similar characteristics (Table 1 and Figure 2). The reach definitions are used in this report to describe sediment impacts and are referenced throughout the report.

The locations of several landmarks along the river are given in Table 2. There are eight major bridge crossings between Matilija Dam and the ocean, three levees, and two water diversions. There is extensive development along the river with several businesses and communities located in areas where flooding has previously occurred. Many of these developments are now bounded by levees.

Table 1. Major Reaches of Matilija Creek and the Ventura River.

Reach #	River Mile	Reach
8	30 – 17.46	Matilija Creek
7b	17.46 – 16.76	Matilija Delta
7a	16.76 – 16.46	Matilija Reservoir
6b	16.46 – 15	Downstream of Matilija Dam to Canyon opening
6a	15 – 14.15	From Canyon opening to upstream of Robles Diversion
5	14.15 – 11.27	Near Robles Diversion to Baldwin Road Bridge
4	11.27 – 7.93	Baldwin Road Bridge to San Antonio Creek Confluence
3	7.93 – 5.95	San Antonio Creek Confluence to Foster Park Bridge
2	5.95 – 0.60	Foster Park Bridge to Main St Bridge
1	0.60 – 0.0	Estuary

Table 2. Landmarks Along River.

Landmark	River Mile
Upstream End of Matilija Reservoir Delta	17.46
Upstream End of Matilija Reservoir	16.76
Matilija Dam	16.46
Matilija Road Bridge	15.88
Matilija Creek confluence with N. Fork Matilija Creek	15.8
Los Robles Diversion Dam	14.15
Baldwin Road	11.27
End of Live Oaks Levee	10.29
Beginning of Live Oaks Levee	9.39
Santa Ana Blvd	9.38
Confluence of Ventura River and San Antonio Creek	7.93
End of Casitas Levee	7.85
Beginning of Casitas Levee	6.84
Foster Park Diversion	6.31
Confluence of Ventura River and Coyote Creek	6.24
Casitas Vistas Road (USGS stream gage)	5.95
Confluence of Ventura River and Cañada Larga	4.63
Shell Road	3.16
End of Ventura River Levee	2.38
Main Street	0.6
Ventura Freeway (Highway 101)	0.45
Southern Pacific Railroad	0.19
Beginning of Ventura River Levee	0
Ventura River Mouth	0

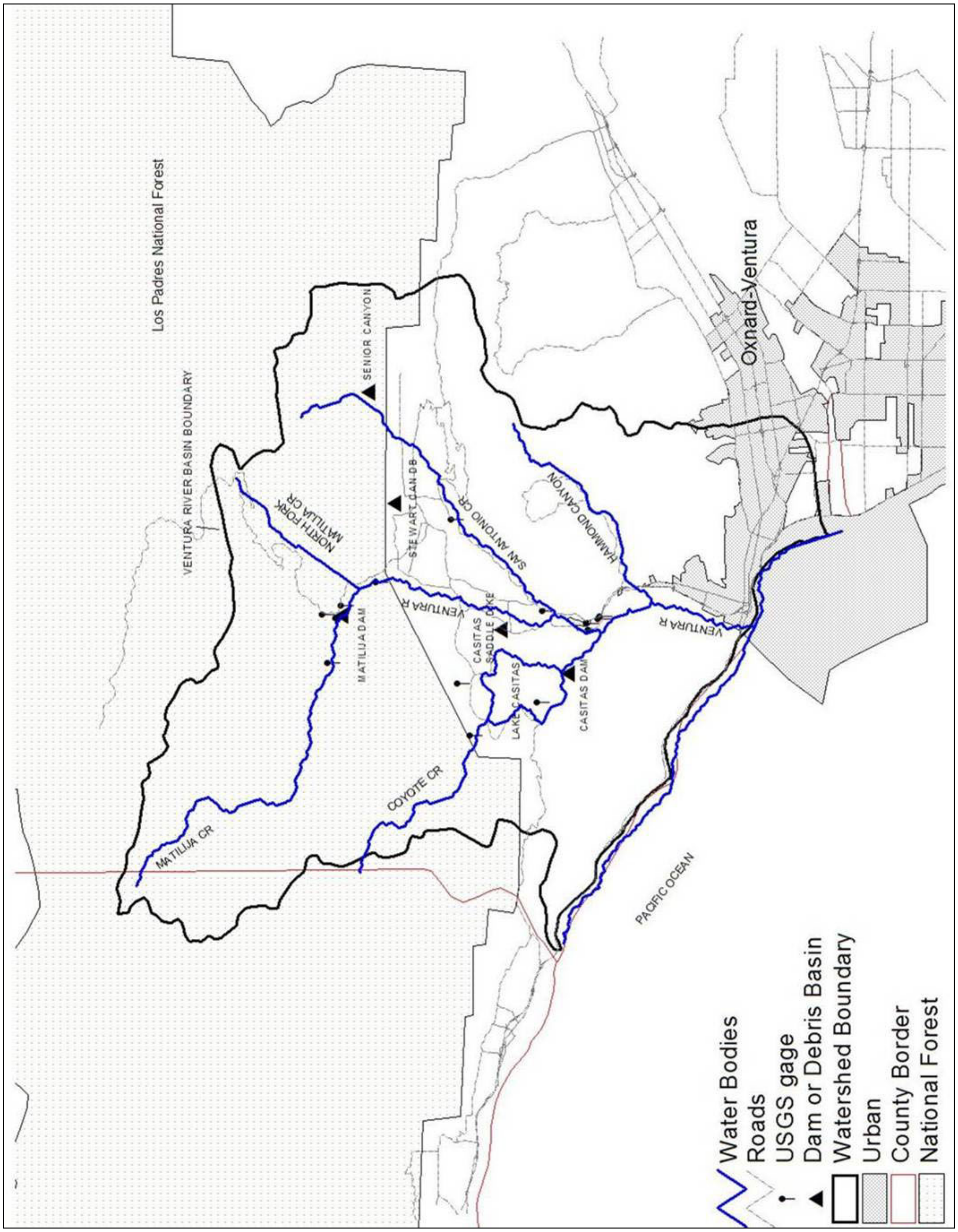

Figure 1. Ventura River Watershed.

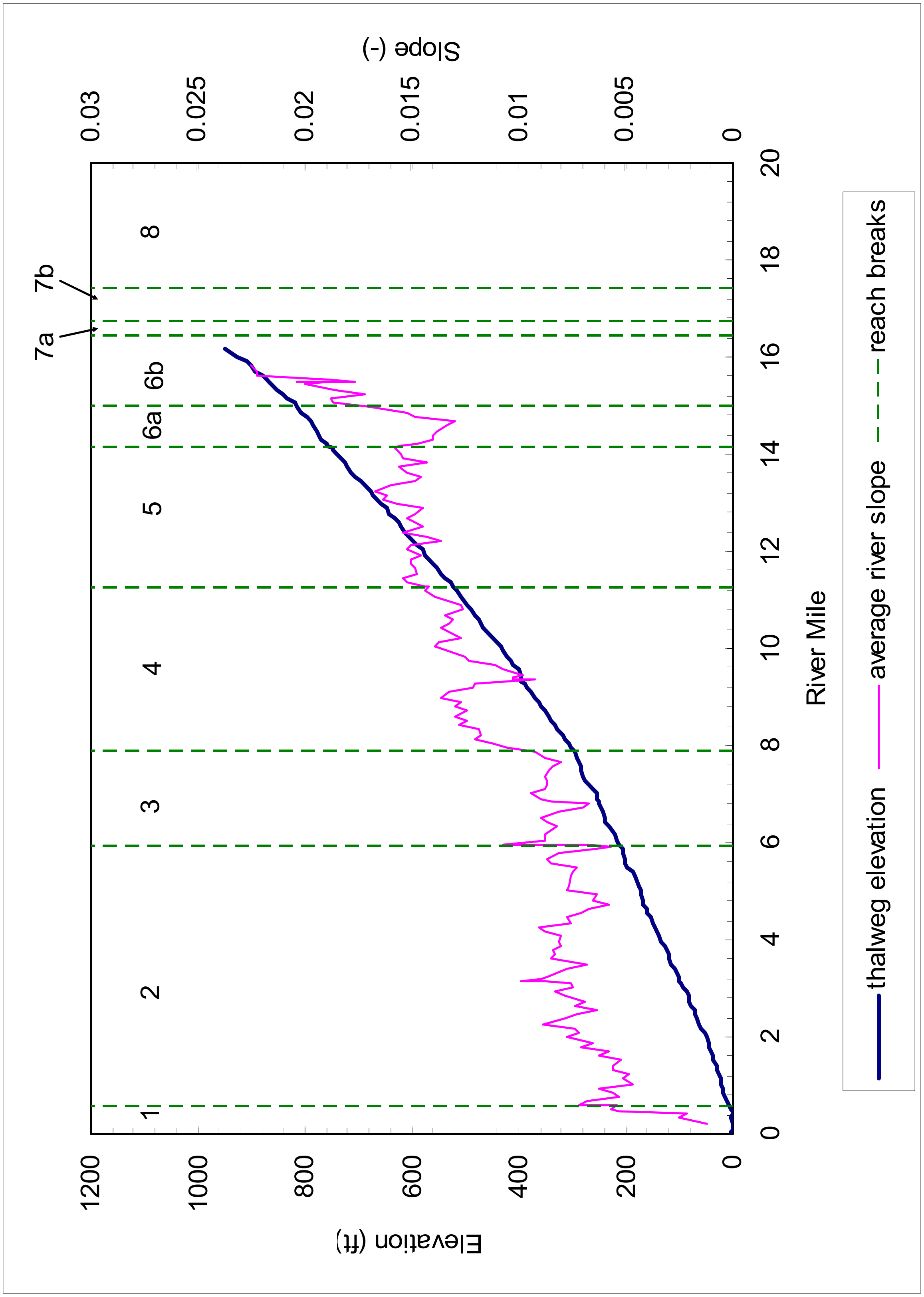

Figure 2. Bed Profile and Reach Definitions in the Ventura River.

There is extreme seasonal variation in the rainfall and over 90% of the rainfall occurs during the six months between November and April (Figure 3). The source of the rainfall data is the National Climatic Data Center rain gages in the cities of Ventura and Ojai (NCDC, http://lwf.ncdc.noaa.gov/oa/ncdc.html). The period of record was from as early as 1874 until as late as 1995. The flows in the river show the same trend, but lag in time. This lag is due to the storage capacity of the soil in the watershed.

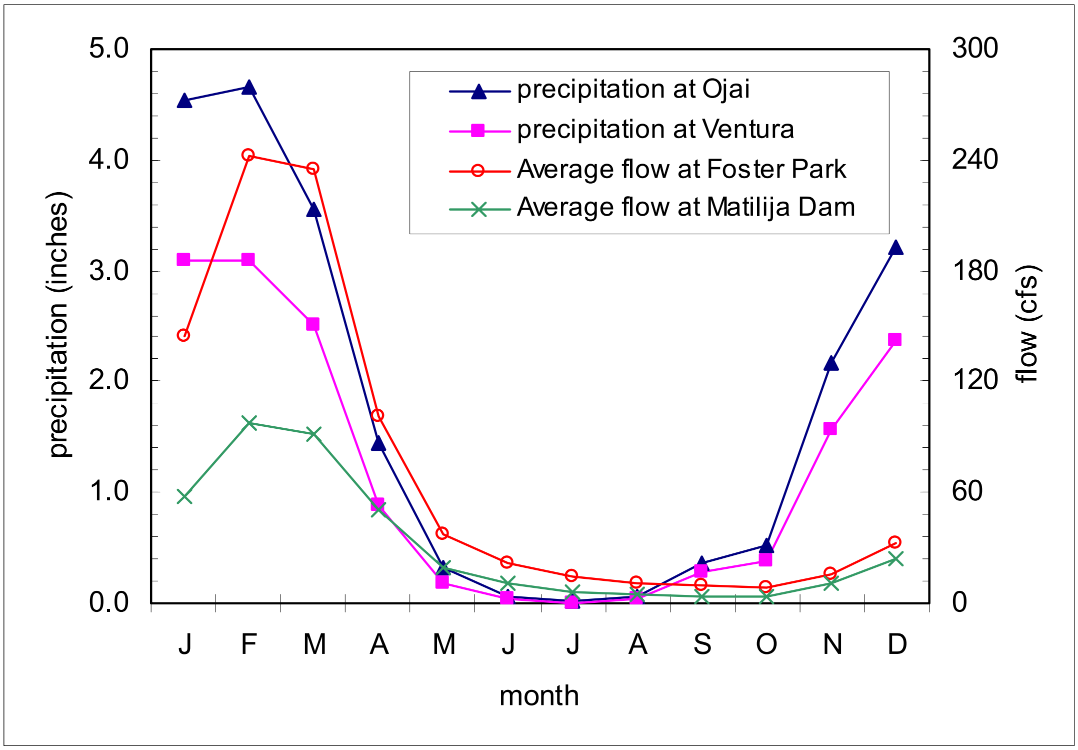

Figure 3. Seasonal variation of average rainfall and flow in Ventura River Watershed.

Water Diversions, Dams, and Levees

Matilija Dam was built in 1947 with an initial capacity of 7,018 ac-ft and impounds Matilija Creek. Matilija Reservoir currently has less than 500 ac-ft of capacity remaining and its ability to trap sediment and attenuate floods has been significantly decreased.

Casitas Dam, which dams Santa Ana and Coyote Creeks, was built in 1958 with an initial capacity of 250,000 ac-ft. Casitas Dam was built as part of the Ventura River Project by Reclamation. Prior to Casitas Dam, Coyote Creek contributed 18% of the flow in the Ventura River at Foster Park. After construction, significant flow downstream of the Casitas Dam in Coyote Creek only occurs during wet years in which water is spilled from the reservoir. As a result, Coyote Creek contributed approximately only 5 % of the flow in the Ventura River during the period 1971-1980. Casitas Dam effectively traps all the sediment that enters into the reservoir.

Casitas Reservoir yields approximately 21,500 ac-ft/yr of water and an additional 8,000 ac-ft is lost to evaporation and seepage. Based on this, the average detention time of water in the reservoir is 8.5 years.

Robles Diversion Dam, built in 1958, diverts water from the Ventura River into Casitas Reservoir. Most of the diversion at Robles Diversion Dam occurs from December through March and is highly variable. Casitas Municipal Water District's (CMWD) ability to regulate the flows in Matilija Creek is significantly impaired because of the limited storage capacity of Matilija Reservoir. The maximum diversion rate at Robles Diversion Dam is approximately 500 ft^3/s (Chris Morgan, CMWD). It was found that the maximum benefit of Matilija Dam (with its current capacity of 500 ac-ft) to the diversion at Robles is 590 ac-ft/yr.

The City of Ventura diversion structure is located at Foster Memorial Park. An underground dam extending most of the way from the surface to bedrock forces water to the surface at the location. Part of the diversion is surface water and part is subsurface. ENTRIX (1997) states that, on average, 2,500 ac-ft of surface water and 3,900 ac-ft of groundwater is diverted at Foster Park annually. The "surface diversion" is an actually a combination of a shallow intake pipe buried approximately 4 feet below the surface and a surface diversion dam. The surface diversion dam has not been used since 2000 because the river shifted and abandoned the channel leading to the surface diversion.

There are three major levees along the Ventura River and their characteristics are shown in Table 3. The upstream levee is near the Santa Ana Bridge. It protects the Live Oak community along the west bank. The Casitas Springs Levee is along the east bank and protects the town of Casitas Springs. The Ventura Levee is along the east bank and protects the city of Ventura.

Table 3. Levee Characteristics along the Ventura River.

Levee	**Ventura**	**Casitas Springs**	**Live Oaks**
Year Constructed	1947	1978	1995
Downstream River Mile (mi)	0	6.84	9.39
Upstream River Mile (mi)	2.38	7.85	10.29
Downstream Elevation (ft)	14.4	267.4	412.2
Upstream Elevation (ft)	120.0	307.6	465.5

Hydrology

A flood-frequency analysis was performed for the entire length of the Ventura River. Frequency discharges for the 10-, 20-, 50-, 100-, and 500-year events were developed. The analysis is detailed in a separate report (Bullard, February 2002). Three stream gage records were used in the initial analysis: Matilija Creek above the Matilija Reservoir (USGS gage 11114500), Matilija Creek at Matilija Hot Springs (USGS gage 11115500) and Ventura River near Ventura (USGS gage 11118500). To determine the selected return period flows, various methodologies were investigated and it was determined that a top-fitting method was most appropriate for the Ventura River. The frequency of the 7 largest floods on record were fit with a regression equation and this regression equation was used to determine the flood magnitudes with a 10-, 20-, 50-, 100- and 500-year return period. To obtain the flood magnitudes with 2- and 5-year return periods, a separate analysis of partial duration series was performed (Bullard, May 2002). Matilija Dam has a negligible impact on the peak flows of large floods (floods with a return interval greater than 10 years). The peak discharges for Matilija Creek (USGS gage 11115500) are show in Figure 4. The peak discharge at this gage has a wide range of variation with some years recording peak discharges less than 100 ft^3/s and a maximum-recorded flow of 19,600 ft^3/s.

Table 4. Recommended Peak Flows for the Ventura River at Existing Stream Gauge Sites.

	Flood Flows at Selected Locations (ft^3/s)					
Return Period (yr)	**Upstream of Confluence with N. Fork Matilija Creek**	**Downstream of Confluence with N. Fork Matilija Creek**	**Baldwin Rd.**	**Casitas Springs**	**Casitas Road Bridge**	**Shell Chemical Plant**
2	3,060	3,250	3,380	4,130	4,520	5,080
5	7,090	7,580	7,910	9,820	11,060	12,250
10	12,500	15,000	16,000	35,200	36,400	41,300
20	15,200	18,800	19,800	44,400	46,400	52,700
50	18,800	24,000	24,800	56,600	59,700	67,900
100	21,600	27,100	28,300	66,600	69,700	78,900
500	27,900	35,200	36,700	89,000	93,100	105,500

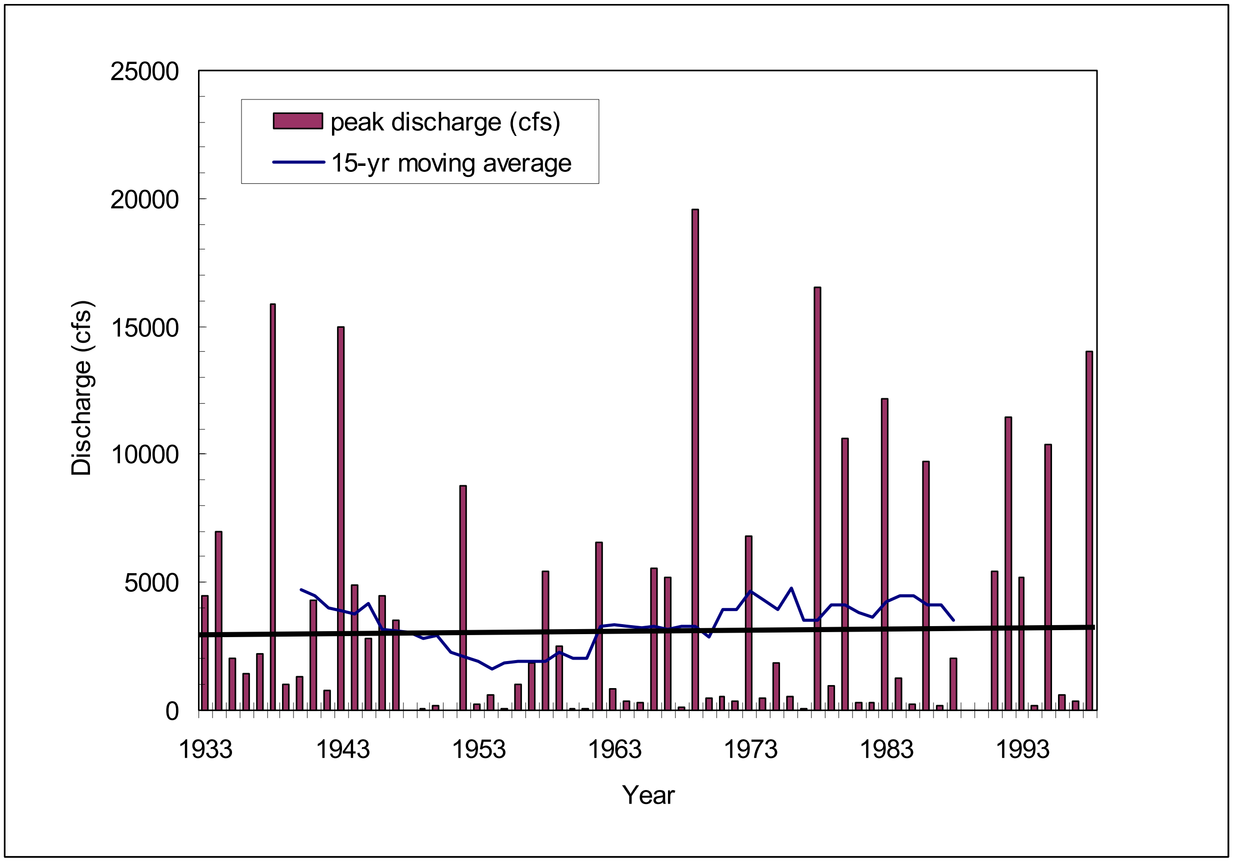

Figure 4. Record of Peak Discharges on Matilija Creek (USGS gage #11115500). Flows between Oct 1 1988 and Sept 30 1990 were not available at this gage.

Turner (1971) estimated that the ground water storage in the Upper Ventura River in the spring of 1970 was 20,410 ac-ft. From 1947 to 1973, Turner states that groundwater use in the Upper Ventura River ranged from 1458 to 6268 ac-ft/yr and that production was over 4000 ac-ft from 1963 to 1973. Entrix (2001) has prepared a report analyzing the surface-groundwater interactions.

Flood Plain Analysis

Overflows were mapped for the 10, 50, 100, and 500-year return periods using results from the HEC-RAS 3.1.1 hydraulic model. The overflow Figures are presented in Exhibit D and show the inundated areas along the Ventura River for the study reach. Mapping assumed constructed levees will not erode or be significantly damaged during flood events. Levees fail to perform only when overtopped. The hydraulic model treated portions of a section inundated because of levee overtopping as ineffective flow areas. Overflow mapping neglected natural levees and expanded the floodplain into areas hydraulically disconnected from the channel under current conditions, but within the historic flow path and below the current water surface elevation. In many cases, this assumption results in similar flood boundaries for events of different magnitudes. This assumption results in a more conservative estimate that accounts for potential changes in planform during large flood events. The properties at risk are identified in the sections below. They are identified by reach and RM.

Reach 6b – RM 16.5-15.0

Reach 6b begins immediately downstream of Matilija Dam and extends downstream to the canyon mouth. This reach contains little development except the "Matilija Hot Springs" facility. While events do not inundate the pool itself, flows above the 50-year event inundate the lower grounds.

Reach 6a – RM 15-14.15

Reach 6a begins at the canyon mouth and extends downstream to Robles Diversion Dam. There are approximately 50 structures located near the river in Reach 6a.

<u>Camino Cielo</u>: There are at least two houses situated along the south bank of the river on the floodplain surface, one upstream, and one downstream of the Camino Cielo Bridge. There are nine structures that appear to be primarily vacation cabins, located upstream of the Camino Cielo Bridge on the north bank of the channel. They are located at a variety of elevations, with the highest being some ten feet above the floodplain surface, and at least five of these being less than one foot above the floodplain surface. The canyon is extremely narrow at this point, with a minimum width of 280 feet, and is only a short distance downstream of Matilija Dam. These structures have a considerable risk of inundation, under both the without- and with-project conditions. Numerous structures are located within 50 feet of the channel bank. All but the structures on the high terrace are within the 100-year floodplain.

<u>Meiners Oaks Area</u>: There are approximately 20 structures located along Oso Road and North Rice Road between RM 14.4 and 14.15 within Reach 6a. (There are additional structures within this community downstream of 14.15, but located in Reach 5.) All of these structures are

constructed at grade, with no significant first floor elevation above the ground. There is no functional levee and all of these structures are above the 100-year floodplain.

Robles Diversion: Robles Diversion Dam is located at the end of Reach 6a. The diversion crosses the Ventura River channel and is within the 100-year floodplain.

Reach 5 – RM 14.15 – 11.27

Reach 5 starts from downstream of Robles Diversion and continues until Baldwin Road Bridge.

Continuation of Meiners Oaks Area: There is a stable, a residence, and appurtenant structures located south of Meyer Road within the 100-year floodplain. All of these structures are constructed at grade, with no significant first floor elevation above the floodplain. There is no functional levee. Above RM 13.83, the Meiners Oaks area lies within the Cozy Dale drainage basin with a substantial barrier to potential channel migration into the area. The steep slope of the tributary is expected to prevent backwater influence on the inundation level so the area was excluded from the inundation study. Below RM 13.83, historic photos show active channels in the area. The floodplain was extended to the historic migration extents.

Reach 4 – RM 11.27 – 7.93

Reach 4 starts from downstream of Baldwin Road Bridge and continues until San Antonio Creek.

Live Oak Acres: The Live Oak Levee begins at Ventura River Mile 9.39 on the right bank upstream of the Santa Ana Bridge. It extends along the populated area of Live Oaks to approximately river mile 10.23. The levee itself is joined to the fill of Burnham Road at the upstream side preventing it from being overtopped from the upstream end. This levee contains the 100-yr flood. However, it was necessary to lower the bed elevations at the Santa Ana Bridge based on the maintenance program of the County of Ventura. The Santa Ana Bridge is a severe constriction on the flow. This causes a backwater upstream of the bridge and increases the likelihood that the Live Oak Levee will be over topped. Another repercussion of the bridge constriction is that the scour around the bridge is increased, as evidenced in the photo taken after the 1998 flood (see Figure 4.3). Downstream of Santa Ana road, the floodplain was extended to the limits of historic activity due to uncertainty in the future location of the river.

Reach 3 – RM 7.93-5.95

Casitas Springs: There are at least fifty mobile homes in close proximity to the channel at RM 7.85. The channel at this location is less than 10 feet deep and highly choked with vegetation. The entire mobile home park is at risk of flooding. There is no protective levee at this location. There are numerous structures on Ranch Road, Edison Drive, and Sycamore Drive at Casitas Springs. The protective levee at this location does not provide protection during the 100-year flood.

The Casitas Springs Levee starts on the left bank at approximately Ventura River Mile 6.84 and extends upstream to approximately river mile 7.77. Inundation occurs at the Casitas Levee. Specifically, the 100 and 500-year flood peaks overtop the levee at approximately river mile

7.77. This effectively causes split flow to occur between the channel and the left over bank. Except for the 500-year flood peak, additional flow from the main channel does not flow over the levee between river mile 7.77 and 7.39. Between river miles 7.39 and 7.29, the 50, 100, and 500-year events all overtop the levee and can add additional flow to the floodplain. River flow returns to the main channel and is contained again at approximately Ventura River Mile 6.72. Figure 4.2 is photographic evidence of the potential flood risk at Casitas Levee. It is a picture of the river at near peak flood stage during the 1998 flood event, an event with a return period less than 20 years. The water surface elevation for this flood is within 2 feet of the top of the levee.

There are at least three residences located on the south bank of the river downstream of Casitas Vista Bridge (~ RM 6.8). Foster Park is located within the 100-year floodplain and is at risk of flooding.

Reach 2 – RM 5.95-0.6

Further downstream, there are residences, a school, the City of Ventura Water Filtration Plant, and a gasoline refinery located on the south side of the channel. These structures are all located near the 100-year floodplain.

The Ventura Levee extends from the Pacific Ocean at Ventura River Mile 0.05 to 2.37. The hydraulic model indicated that all discharges from the 2-year to the 500-year floods are confined to the main channel by the Ventura Levee.

Geomorphology of Matilija Creek and the Ventura River

The geomorphology of various reaches of Matilija Creek and the Ventura River is described in Table 5. All reaches are generally steep with a large coarse sediment supply in the channel. The surface riverbed is dominated by large cobbles and has very little material finer than 4 mm. The active river channel has migrated across the valley frequently and bank erosion is common through most reaches, unless natural or fabricated constrictions exist.

Table 5. Geomorphic Descriptions of Reaches of Matilija Creek and Ventura River. The reach numbers correspond to those found in Figure 1 and Table 1.

Reach #	Land Marks	River Miles	General Geomorphic Characteristics
7a	Matilija Dam and reservoir	16.8 - 16.47	Reach covered by Matilija Dam and reservoir.
6b	Matilija Dam – North Fork Matilija Creek	16.47 - 16.0	Narrow, steep and sinuous bedrock controlled canyon reach; channel characterized by very coarse bedload and a single very narrow (<300 feet) alluvial terrace (e.g., Matilija Hot Springs).
6b	North Fork Matilija Creek – Kennedy Canyon	16.0 - 15.0	Narrow canyon reach opens into narrow linear valley; alluvial fans and low alluvial terraces flank channel; distal margin of alluvial fan deposits truncated by the river; lower end of the reach is controlled by bedrock (Coldwater Formation).
6a	Kennedy Canyon – Robles Dam	15.0 - 14.15	The average valley and river channel widen (400’ to more than 1650’) and the channel slope (0.020 to 0.013) changes significantly relative to the upstream reach.

Reach #	Land Marks	River Miles	General Geomorphic Characteristics
5	Robles Dam – Meiners Oaks	14.15 - 12.3	Similar characteristics to upstream reach with exception that the valley continues to widen to roughly 2-3 times width of reach 5A. River channel takes on braided pattern. The downstream end of the reach constricted between bedrock and older alluvial terrace; controlled by geologic structure (Arroyo Parida-Santa Ana fault).
4/5	Meiners Oaks – Santa Ana Blvd.	12.3 - 9.5	Channel again widens into alluvial valley flanked by high terraces. The channel retains braided character but narrows slightly near Live Oak Acres. Natural constriction created by Devils Gulch and Oak View faults. The Live Oak Acres levee that flanks the channel for almost a mile to the bridge at Santa Ana Blvd.
4	Santa Ana Blvd. – San Antonio Creek	9.5 – 7.93	Similar characteristics to upstream reach; wide alluvial valley flanked by high alluvial terraces. Channel pattern begins to shift from braided to multi-tread with vegetated bars. Downstream end of the reach is controlled by bedrock and geologic structure near the confluence of San Antonio Creek (Ayers Creek syncline).
3	San Antonio Creek – Foster Park	7.93 - 6.1	River channel and valley narrow slightly from upstream reaches. Large portion of the reach is flanked by the Casitas Springs levee. Downstream end of the reach is controlled by bedrock and geologic structure (Cañada Larga syncline).
2	Foster Park – Shell Road	6.1 - 3.0	Narrow canyon reach opens into wide valley flanked by broad flat alluvial terraces. River channel width remains narrow and becomes deeply incised in alluvium in the lower portion of the reach. Bedrock is exposed in the channel bank at several locations in the upper part of the reach (northern flank of the Ventura Avenue Anticline).
2	Shell Road - Estuary	3.0 - 0.6	Similar characteristics to Reach 3B with exception that valley and active channel continue to widen in a downstream direction and no bedrock was observed in the reach.
1	Mouth of the Ventura River/Estuary	0.6 - 0.0	Morphology of the reach formed primarily in response to large floods, tidal influence, and coastal processes. Impacted by channelization and three bridge crossings.

The bed material generally becomes coarser with distance upstream. Near the ocean the d_{50} is approximately 70 – 80 mm, and downstream of Matilija Dam it increases to over 300 mm. In the reach just downstream of the dam, the valley walls are steep and it is possible that some of the large material has its source from the hill slopes in the vicinity. Some of the bed material in this reach may not have been transported by the stream but rather may have been sloughed from the valley walls. Within the study area, the bed material decreases in size upstream of the dam.

The sediment loads in the Ventura River are dominated by infrequent flood events as evidence by the sediment loads in Figure 5. Over the period from 1969 to 1981, more than 96 percent of the sediment load was transported during the floods occuring in just three years (1969, 1978, and 1980). Such conditions continue to occur and significant sediment transport can be assumed to occur only during floods. Most sand is transported as suspended load and the 100-yr flood will suspend particles as least as large as 2 mm.

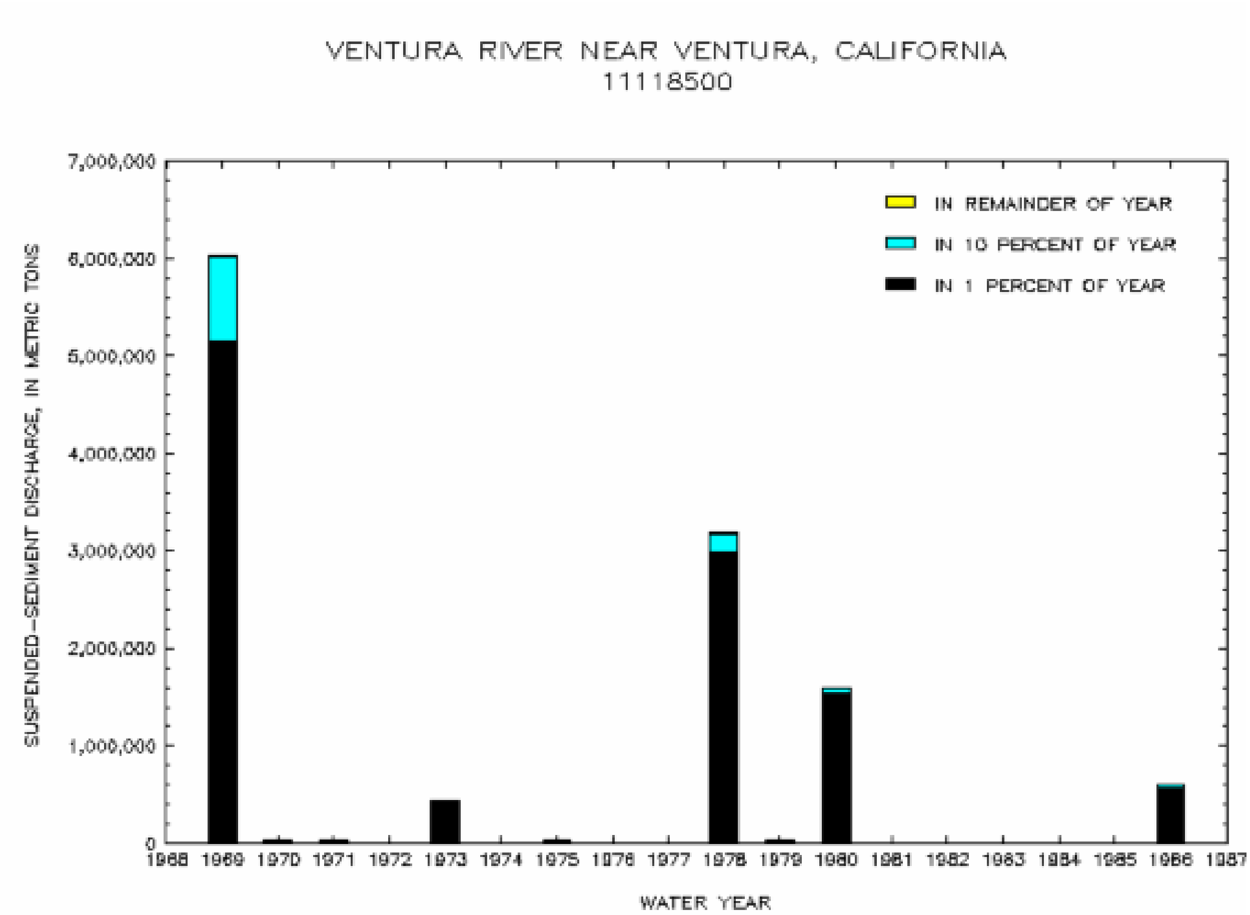

Figure 5. Suspended Sediment Loads in the Ventura River. There was no data recorded from 10/1/73 to 9/30/74 and from 10/1/82 to 9/30/85 (figure from USGS http://webserver.cr.usgs.gov/sediment/). The year 1983 had substantial flow and sediment transport.

The annual sediment volumes supplied to the ocean are listed in Table 6. The 'Current' condition refers to the current supply to the ocean. The Equilibrium condition refers to the conditions when Matilija Dam is completely full of sediment and the downstream river channel is in a state of dynamic equilibrium. This means that there is no net erosion or deposition within the reaches.

Table 6. Average annual sediment delivery to the ocean.

	yd^3/yr of sediment delivered				
type	**fines**	**sand**	**gravel**	**cobbles**	**total**
Current	311,000	136,000	9,400	530	457,000
Equilibrium Estimation	373,000	164,000	11,300	630	548,000

Deposition in Matilija Reservoir

Sedimentation in the Matilija Reservoir has been a concern since its construction (Jamison, 1949; Boyle, 1964). Several surveys have tracked the progression of sedimentation in Matilija Reservoir. In a 1954 report, Reclamation estimated that Matilija was filling in at a rate of 79 acre-ft/yr (Reclamation, 1954). In 1947, a sediment-monitoring program was started to document the sediment deposition occurring in the reservoir. Six silt control lines have been surveyed over a 52 period in the reservoir. These control lines were resurveyed in 1948, 1958, 1964, 1965, 1970, 1986, and 1999. Using CAD technology, the silt control lines were digitized for each year and a volume of sediment trapped in the reservoir was computed using the 1947 silt lines as a baseline. A sediment volume was also calculated for the October 2001 survey. Figure 6 shows the cumulative sediment deposition in Matilija Reservoir over time. In addition, the predicted future sedimentation is shown.

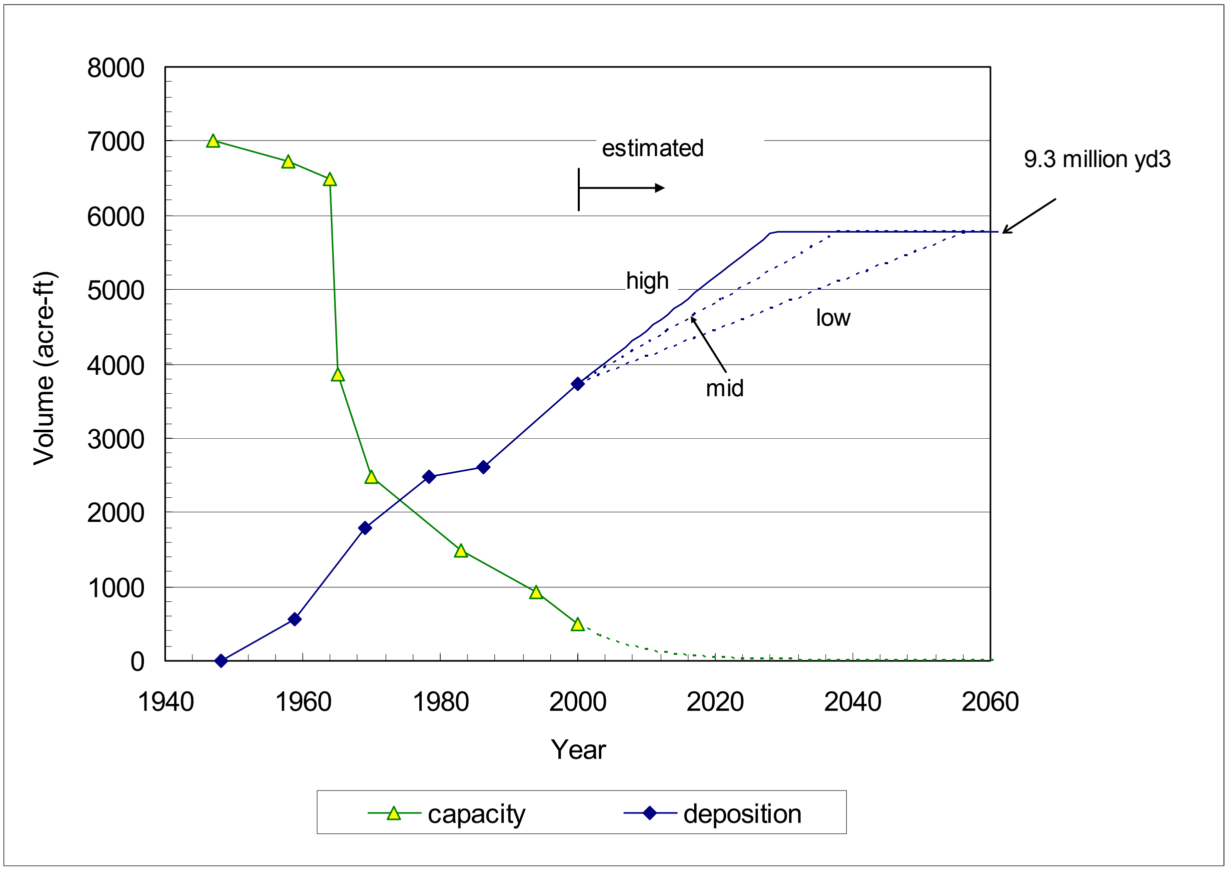

Figure 6. Historical and Future Deposition in Matilija Reservoir.

The sediment trapped in the Reservoir can generally be divided into three areas: The Reservoir area, the Delta area, and the Upstream Channel area. The Reservoir area starts at the upstream face of the dam and continues upstream for approximately 1,400 feet. Its boundaries are approximated by the location of the pond. The total volume of sediment in the Reservoir area is estimated to be 2.1 million cubic yards. The Delta area extends from the upstream edge of the pond approximately 1,500 feet upstream. The total volume of sediment in the Delta area is estimated to be 2.5 million cubic yards. The Upstream Channel area extends from the upstream edge of the Delta area (approximately 2,900 feet upstream of the dam) to the upstream limit of sedimentation (approximately 6,000 feet upstream of the dam). The total volume of sediment in the Upstream Channel area is estimated to be 1.3 million cubic yards.

Based on the core sampling, the Corps determined average gradations for the three different regions of the sediments behind Matilija Dam. The measurements of the bulk density indicate that there is no significant stratification of bulk density in the reservoir. Based on the information from the Corps, the measured current average bulk density of the entire reservoir area is 73 lb/ft^3.

Table 7. Gradations determined from drill data by Corps.

	% finer than		
Grain Diameter (mm)	**Reservoir**	**Delta**	**Upstream Channel**
512	100.0	100.0	100.0
256	100.0	100.0	87.9
128	100.0	100.0	75.9
64	100.0	99.8	60.9
32	100.0	98.4	48.9
16	99.9	95.1	36.9
8	99.8	92.5	29.9
4	99.7	89.9	24.9
2	99.7	87.3	21.9
1	99.5	83.7	18.4
0.5	99.0	77.5	15.0
0.25	97.2	66.5	12.0
0.125	92.2	50.8	9.0
0.0625	82.8	33.2	6.0
0.031	70.9	21.9	4.0
0.016	57.3	14.5	2.0
0.008	43.1	9.7	1.0
0.004	30.1	5.3	0.0
0.002	18.0	0.0	0.0
Total Volume (yd^3)	**2,100,000**	**2,400,000**	**1,400,000**

Alternative Analysis

Seven alternatives were analyzed:

Alternative	Sub-Alternative	Description
No Action		No removal of dam or sediments
1		Full Dam Removal/Mechanical Sediment Transport: Dispose Fines, Sell Aggregate
2		Full Dam Removal/Natural Sediment Transport
	2a	Slurry "Reservoir Area" Fines Offsite
	2b	Natural Transport of "Reservoir Area" Fines
3		Incremental Dam Removal/Natural Sediment Transport
	3a	Slurry "Reservoir Area" Fines Offsite
	3b	Natural Transport of "Reservoir Area" Fines
4		Full Dam Removal/Sediment Stabilization on Site
	4a	Permanent Stabilization
	4b	Temporary Stabilization

[1] Throughout this document, the term "Reservoir Area" will be used to refer to the area normally covered by water due to Matilija Dam.

Analysis will be presented for all alternatives. However, in many cases, the sediment impacts between alternatives are similar. In most all cases, the sediment impacts of Alternative 1 (Mechanical Sediment Transport) would be expected to be similar to those of Alternative 4a (Permanent Stabilization). In addition, the long-term impacts between Alternative 2 and 3 will be similar. Therefore, in many cases the impacts from several alternatives are discussed simultaneously.

Many different analysis methods were used to generate the conclusions stated in this report. Below is a list of the data sources and analysis methods used in this project:

Data Sources

- Aerial Photography from 1947, 1965, 1970, 1992, 2000
- Bed material sampling of the entire river performed in 2001
- Photogrammetry surveys of the Ventura river channel performed in 1970 and 2000.
- Suspended sediment load sampling in the Ventura River from 1969 until present performed by the USGS.
- Analysis of drill cores from the reservoir sediments
- Stream flow records in Matilija Creek, North Fork Matilija Creek, San Antonio Creek, and the Ventura River.
- Clean out records at Robles Diversion

- Diversion records at Robles Diversion
- Diversion records at Foster Park Diversion

Analysis Methods

- Geomorphic assessment using historical photography
- Analytical sediment wave description of downstream sediment deposition
- Numerical simulations using GSTARS-1D

Analytical Model of Deposition

A new analytical sediment wave model was developed for use in this project. It was verified using Laboratory data from St. Anthony Falls Laboratory in Minneapolis, MN. The details of this verification are presented in Greimann et al. (2004). The analytical model was compared against the deposition results from GSTARS-1D for Alternative 2a. The two methods gave similar results in terms of average channel deposition. The agreement between the two methods is one-step in the verification of each method. The analytical model gives approximately depth of deposition expected from the movement of the sediment wave downstream.

GSTARS-1D

GSTARS-1D (Generalized Sediment Transport model for Alluvial River Systems-One Dimension) is a tool for estimating the behavior of rivers in response to sediment loading. It is a one-dimensional model that can simulate steady and unsteady flow, changing river geometry, structures in the river such as diversion and bridges, and cohesive and non-cohesive sediment transport. It also has the flexibility to be easily modified for the specific problems that occur in dam removal studies.

The model was calibrated to the Ventura River using the period from 1971 to 2000. Survey data was available at the beginning and end of this period and it was possible to compare the model results against the measured data. There were two reaches with significant erosion during the period from 1971 to 2001. One reach was from just below Robles Diversion Dam (RM 14) to approximately RM 13. The other reach that degraded extended from Foster Park to the Ventura Levee (RM 6 to RM 2). The model predicts that both of these reaches will degrade. However, there are some discrepancies. From RM 14 to RM 13.5 there is less erosion predicted than actually occurred. There could be several reasons for the discrepancy. One is that the model does not account for the removal of sediment at Robles by CMWD. The diversion is accounted for in the model, but sediment is allowed to go over the top of the diversion after the diversion fills with sediment. Another reason for the discrepancy could be that the sediment was finer in 1970 in this reach. From RM 13.5 to RM 13, erosion is generally well predicted based on the profile and cross section comparison. The erosion in the lower reach begins at approximately RM 5.5; however, the model predicts that the erosion begins at approximately RM 5. From RM 5 to RM 4, the erosion is well predicted, but the model does not predict erosion continuing beyond RM 4. Changes in bed material between 1970 and 2002 could account for this discrepancy. Additional simulations and geomorphic analysis are being conducted to investigate the cause of the erosion downstream of RM 5. A comparison of erosion volumes is not possible because the exact location of the 1970 cross sections is not known.

SUMMARY OF IMPACTS FROM EACH ALTERNATIVE

A summary of the impacts associated with each alternative is presented below. The details of the impact analysis are given in Section 9 titled "Impact Descriptions".

No Action Alternative

The No Action Alternative would cause continued deposition behind Matilija Dam. Over 3 million more cubic yards would be expected to deposit behind the dam in the next 50 years. A majority of that sediment would be sand. The reservoir capacity would be expected to be 150 ac-ft in 2010 and less than 50 ac-ft by 2020. However, the relatively small reservoir still provides a benefit to the diversion at Robles. At its current capacity of 500 ac-ft, it is estimated to increase the annual water diversion at Robles Diversion by 590 ac-ft. As the reservoir fills with sediment, this benefit to water diversion would decrease until it is not significant. From now until the reservoir completely fills with sediment the total benefit of Matilija Dam to the diversion at Robles would be 5000 ac-ft.

The river would be expected to remain relatively stable from Matilija Dam downstream to Robles Diversion. From Robles Diversion to Baldwin Road, the river would continue to erode for the next 50 years. On average, there should be approximately 2 feet of erosion. From Baldwin Road to San Antonio Creek, the Ventura River would remain relatively stable. Nevertheless, excavation of sediment at Santa Ana Blvd Bridge would be required to maintain adequate flood capacity. Downstream of San Antonio Creek, 2 feet of deposition would be expected in the Casitas Springs area. The reach between Foster Park and Shell Road Bridge has experienced significant erosion in the past and this would be expected to continue for the next 50 years, with a maximum erosion depth of 3 feet in this reach.

Most of the silt and clay that enters Matilija Reservoir passes over the top of Matilija Dam. However, there is still a small amount of silt and clay that is trapped behind Matilija Dam at the lower flows. It is expected that the average fine sediment concentrations downstream of Matilija Dam would increase by approximately 30% after the reservoir is nearly filled with sediment, which is expected to occur in approximately 10 years.

In approximately 40 years, sand and gravel-sized sediment would start to pass over the dam crest, at which time it is estimated that over 9 million yd^3 of sediment would be stored behind the dam. When sand and gravel-sized sediments begin to pass over the dam, abrasion from these coarse particles may damage the concrete surface of dam crest. Once coarse sediment starts to pass downstream, the reaches immediately below the dam will begin to aggrade. There will also be an increase in the deposition that occurs in Robles Diversion area. It is expected that in approximately 100 years, the Ventura River would be in approximate equilibrium, meaning that sediment load entering the river system is in approximate balance with the sediment load exiting the system. The approximately 2.2 million yd^3 of sand that is presently trapped behind the dam would not be supplied to the beach and approximately an additional 2 million yd^3 of sand would be trapped behind the dam in the next 40 years.

There are current flood concerns along the Ventura River. Several residences downstream of Robles Diversion may be at risk of flooding during a 100-yr flood. At the Santa Ana Bridge, the riverbed would require excavation after every flood if it is to maintain 100-yr flood capacity. In addition, the levee along the Ventura River at the town of Casitas does not provide protection against the 100-yr flood. Flooding would continue to be a problem unless additional levees are constructed.

Full Dam Removal/Mechanical Sediment Transport: Dispose Fines, Sell Aggregate Alternative (Alternative 1)

The Full Dam Removal/Mechanical Sediment Transport: Dispose Fines, Sell Aggregate Alternative (Alternative 1) would remove all the sediment stored behind Matilija Dam from the river system. There would be a natural re-supply of Matilija Creek Sediment to the downstream reaches. This natural re-supply of sediment would have noticeable impact on reaches located between Matilija Dam and Baldwin Road. However, because it is a canyon area, RM 16.5 to RM 16 of Matilija Creek would remain relatively stable. There would be approximately 2 feet of deposition expected in the reach immediately downstream of Robles Diversion. The river would be expected to remain relatively stable after Baldwin Road until the Casitas Springs area where an additional 2 feet of deposition would be expected over the next 50 years. The reach between Foster Park and Shell Road Bridge has experienced significant erosion in the past and this would be expected to continue for the next 50 years, with a maximum erosion depth of 3 feet in this reach.

Significant levee improvements would be required in several areas to prevent the existing flood risk from increasing. Immediately downstream of the dam, the Matilija Hot Springs Private Resort may need to be evacuated for a period of several years until the river stabilizes in that area. The aggradation there should be relatively minor, but some uncertainty exists as to the final equilibrium elevations.

Proceeding downstream, the bridge at Camino Cielo is a low water crossing that would cause aggradation and may increase the flood risk to those residences. This bridge would have to be modified or these residences may need to be evacuated. Immediately downstream of Robles Diversion, some of the Hawthorn Acres residences are built in the floodplain and a levee would need to be constructed to protect them. The Santa Ana Bridge is a severe constriction on the flow and it is in danger of being overtopped by the 100-yr flood if aggradation occurs at the bridge. Therefore, a bridge replacement would be suggested, where the new bridge would have a higher bridge deck and a wider opening to pass flows. The Casitas Levee is currently undersized and would need to be improved to meet the 100-yr flood protection criteria. An additional 2 feet of deposition would be expected at this site over the next 50 year and therefore, the levee would have to accommodate this as well.

Because of the re-supply of Matilija Creek sediment, the deposition at Robles Diversion may increase by approximately a factor of two if there are no changes to the current diversion facility. This would increase maintenance costs and perhaps increase the risk of missed diversions during high flow events. A sediment bypass structure would be recommended and its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". The Sediment By-pass would allow high flows to pass through the Robles area without being obstructed. Currently, the sluice gates have a capacity of 6,700 ft^3/s. The 10-yr flood in this area is 15,000 ft^3/s and would potentially cause a large amount of deposition behind the Robles Diversion Dam due to the severe backwater caused by the fixed elevation diversion dam. It is estimated that if a sediment bypass is installed, the diversion capability of CMWD should not be adversely affected. In addition, the sediment bypass would reduce the amount of excavation required and deposition amounts should be similar to those presently occurring.

Silt and clay concentrations in the Ventura River would not be significantly different from the No Action Alternative. However, the total sand transported to the ocean over a 50-year period would increase approximately 20% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

The Full Dam Removal/Natural Sediment Transport Slurry "Reservoir Area" Fines Offsite (Alternative 2a)

The Full Dam Removal/Natural Sediment Transport: Slurry "Reservoir Area" Fines Offsite Alternative (Alternative 2a) uses the natural flows to erode the delta and the upstream channel. The delta is composed of approximately 13% gravel, 54% sand, 28% silt and 5% clay and the upstream channel is composed of approximately 39% cobbles, 39% gravel, 16% sand and 6% silt. When flow starts to erode this material, a narrow deep channel would first be created through the material, followed by gradually widening of the channel through the delta deposits. The rate of widening will be dependent upon the flow rate: the larger the flood, the more material removed and the wider the channel through the delta.

Because the fraction of silt and clay is relatively small in the delta sediments, the turbidity impact will be of relatively short duration. After the first flood peak has past, the concentrations of fine material will quickly decrease, however, they will be 2 to 3 times larger than natural conditions. Currently, the fine concentrations fluctuate by a factor of two or more; so the increases, while real, would be within the range of the natural variability. After a flood with a return period greater than 10 years or after a period of 3 years, which ever comes first, the increase in fine sediment concentration would be expected to reduce to 10 % to 50 % greater than background concentrations. Within 10 years and as early as 5 years following dam removal, the fine sediment concentration will be similar to the No Action Alternative.

The rise in turbidity levels may affect the surface diversion potential at Foster Park on the Ventura River because they currently stop surface diversion when the turbidity level is higher than 10 NTU. The fraction of time that 10 NTU is exceeded at the surface intake would be increased significantly until the first flood passes. After the first flood, it is estimated that the concentrations would be increased by a factor of two to ten times and therefore the surface diversion would be shut down more often than presently. After the third flood passes, the concentrations should return to near natural levels. The upper and lower bounds on the volume of missed surface diversions are 7710 and 4680 ac-ft, respectively. It is recommended that the surface diversion at Foster Park be removed and be replaced by subsurface wells. The subsurface wells would not be adversely affected by the increase in turbidity.

Because the dam would be removed in one-notch in this alternative, approximately 3.9 million yd^3 of sediment would be available for transport in this alternative. Some of this material would deposit in the upper reaches of the Ventura River. There is considerable uncertainty regarding the deposition downstream of the dam and therefore the levee and floodwall design would be necessarily conservative.

Large amounts of sediment would deposit in the area impounded by Robles Diversion Dam with the current diversion design. Based on the simulations run using the 1991-2001 hydrology, Alternative 2a would deposit 70,000 yd^3 the first year following dam removal. Under equilibrium conditions, approximately 40,000 yd^3 would be deposited. Deposition in excess of 40,000 yd^3 could effectively shut down the diversion operations at Robles for that first year and therefore a sediment bypass structure would be recommended. The sediment bypass would reduce the deposition at the site and decrease the risk of missed diversions. Its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". The bypass delays the time at which the deposition becomes excessive and allows operators more time to respond to deposition problems.

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

The Full Dam Removal/Natural Sediment Transport (Alternative 2b)

The Full Dam Removal/Natural Sediment Transport: Natural Transport of "Reservoir Area" Fines Alternative (Alternative 2b) removes the dam all at once and allows natural flows to erode all the sediment stored behind Matilija Dam. The initial erosion would take place vertically and cut a deep channel through the reservoir sediments. The concentration of fine sediment downstream of the dam would be exceedingly large, greater than 100,000 mg/l, for a period of days to weeks. After this initial formation of a channel through the reservoir deposits, the flow would begin to cut a deep narrow channel through the delta deposits. When the flow rate increases during a flood, the channel through the delta would become much wider and a significant amount of sands, gravels, and cobbles material would be removed from the delta. The first two to three floods would carry extremely high sediment loads downstream. Concentrations may be more than 10 times greater than natural conditions for a period of several years. The concentration of fine material would decrease after each flood and would be expected to reduce to approximately twice-natural levels after three floods that are equal or greater than an average annual flood.

The deposition impacts in the upper reaches of the Ventura River would be large and the deposition elevations are uncertain. Therefore, large levees and floodwalls would be required to provide adequate flood protection. The deposition at Robles would be expected to be similar to Alternative 2a and similar mitigation measures as mentioned in Alternative 2a would be required.

Because the turbidity impacts would last much longer than in Alternative 2a, additional mitigation measures at Robles Diversion and Foster Park Diversion would be required. At Robles, a settling basin or alternate sources of water may be necessary to reduce the impact of fine material on Casitas Reservoir. The desilting basin would have to be large enough to accommodate the maximum volume of sediment that could enter Robles Canal. Because the fine sediment concentration would be much higher in Alternative 2b than Alternative 2a, it is recommended that the surface diversion at Foster Park be removed and be replaced by subsurface wells. The subsurface wells would not be adversely affected by the increase in turbidity.

Large amounts of sediment would deposit in the area impounded by Robles Diversion Dam with the current diversion design. Based on the simulations run using the 1991-2001 hydrology, Alternative 2b would deposit 80,000 yd^3 the first year following dam removal. Under equilibrium conditions, approximately 40,000 yd^3 would be deposited. Deposition in excess of 40,000 yd^3 could effectively shut down the diversion operations at Robles for that first year and therefore a sediment bypass structure is recommended. The sediment bypass would reduce the deposition at the site and decrease the risk of missed diversions. Its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". The bypass delays the time at which the deposition becomes excessive and allows operators more time to respond to deposition problems.

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

Incremental Dam Removal/Natural Sediment Transport Slurry "Reservoir Area" Fines Offsite (Alternative 3a)

The Incremental Dam Removal/Natural Sediment Transport: Slurry "Reservoir Area" Fines Offsite Alternative (Alternative 3a) removes the dam in two stages. A portion of the dam would be removed then a flood would be allowed to erode the sediment stored behind the dam and then the remainder of the dam would be removed. For this analysis, the elevation of the dam crest after the first notch would be 1030. This alternative has similar impacts to Alternative 2a, but there would be a greater measure of control of the deposition impacts. If, for example, more deposition than expected occurred at a particular location after the first stage of removal, it would be possible to mechanically remove that sediment from the stream channel or raise levees in that area before the second notch is started. Therefore, the flood risk associated with Alternative 3a would be much less than that of Alternative 2a. The levees constructed for this alternative would not have to be as high as for Alternatives 2a or 2b because the sediments would be released more slowly and would cause less downstream aggradation. However, if the region is experiencing severe drought conditions, there may be up to 7 years between floods and therefore up to 7 years may pass before sufficient sediment is eroded to perform the second notch.

The incremental removal alternatives 3a would be expected to deposit approximately 27,000 yd^3 at Robles Diversion the first year. However, the second notch would take place in the second year and cause approximately 70,000 yd^3 of deposition the following year with the current diversion design. Therefore, a sediment bypass structure is recommended at Robles Diversion. The sediment bypass would reduce the deposition at the site and decrease the risk of missed diversions. Its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment Bypass". The bypass delays the time at which the deposition becomes excessive and allows operators more time to respond to deposition problems.

The turbidity impacts would be similar to Alternative 2a; however, the maximum concentrations would be less, but would occur twice because two notchings would be necessary. Similar to Alternative 2a, it is recommended that the surface diversion at Foster Park be removed and be replaced by subsurface wells. The subsurface wells would not be adversely affected by the increase in turbidity.

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

Incremental Dam Removal//Natural Sediment Transport (Alternative 3b)

The Incremental Dam Removal/Natural Sediment Transport: Natural Transport of "Reservoir Area" Fines Alternative (Alternative 3b) again has similar impacts to Alternative 2b, but the risks of reduced water supply and increased flooding would be less. The levees may not have to be constructed as high because the sediment would be eroded from the reservoir more slowly. The turbidity impacts would be extended over a longer period because new fines would be exposed after each stage of removal. If the region is experiencing severe drought conditions, up to 7 years may pass between the first notch and the second.

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

Full Dam Removal/Permanent Sediment Stabilization on Site (Alternative 4a)

In terms of downstream sediment impacts, Alternative 4a is considered similar to Alternative 1.

Full Dam Removal/Temporary Sediment Stabilization on Site (Alternative 4b)

In the Full Dam Removal/Temporary Sediment Temporary Sediment Stabilization Alternative (Alternative 4b), approximately 2.1 million yd^3 of reservoir fines would be removed and deposited in disposal sites. A channel would be then constructed through the remaining 3.9 million yd^3, which is composed of approximately 1 million yd^3 of silt and clay, 1.8 million yd^3 of sand, and 1 million yd^3 of gravel and cobble. The channel would be then stabilized up to a flood between a 2-yr to 10-yr flood. Only sections in the delta and reservoir area would be stabilized. The first flood would erode the residual sediment that is not stabilized. After this first flood passes through the reservoir area, a portion of the stabilization structure would be removed. The next flood that comes through would mobilize some of the sediment that would be exposed. The downstream impacts would be monitored. Then based on the monitoring, the next stabilization structure section would be removed. As before, a flood would be allowed to pass through the reservoir area and erode the exposed sediment. This process would be continued until all the stabilization structure is removed. Another option would be to use rock as bank protection. The rock could be designed to fail at a particular flow rate so that it is naturally eroded. However, no such design has yet been done.

The rate at which the sediment would erode would be a function of the slope stability of the sediment and the shear stress applied to the banks. The deposition impacts in the downstream river channel associated with this alternative are initially slightly less severe than Alternative 2a because sediment would be not released as quickly. However, the long-term deposition would be similar to Alternative 2a for all but the reaches nearest the dam.

Large amounts of sediment would deposit in the area impounded by Robles Diversion Dam with the current diversion design. Based on the simulations run using the 1991-2001 hydrology, Alternative 4b would deposit up to 70,000 yd^3 the first year following dam removal. Under equilibrium conditions, approximately 40,000 yd^3 would be deposited. Deposition in excess of 40,000 yd^3 could effectively shut down the diversion operations at Robles for that first year and therefore a sediment bypass structure is recommended. The sediment bypass would reduce the deposition at the site and decrease the risk of missed diversions. Its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". The bypass delays the time at which the deposition becomes excessive and allows operators more time to respond to deposition problems.

The turbidity impacts for Alternative 4b, however, would be slightly different from Alternative 2a. Because there would be multiple removals of stabilization structures, there would be multiple impacts of fine sediment. After each removal, there would be some fine sediment released into the river as the flood flow passes through the area. The fine sediment would be mobilized as the banks would be eroded. As the flood recedes, the water elevation would recede from the banks and no longer erode the fine sediment. Therefore, the increases in turbidity would be mostly confined to the flood events and the lows flows would not experience large increases in turbidity. The magnitude of the sediment concentration increases would most likely be about 2 to 4 times greater than natural conditions before the removal of the first revetment. After the first revetment would be removed, the concentrations may temporarily increase between by a factor of 2 to 10 times the current condition. After the final removal of revetment, the turbidity levels should stabilize at equilibrium levels after one or two floods of average size pass through the reservoir area. Foster Park diversion would be affected by the increase in sediment concentration. The upper and lower bounds on the volume of missed surface diversions are 8820 and 4950 ac-ft, respectively. It is recommended that the surface diversion at Foster Park be removed and be replaced by subsurface wells. The subsurface wells would not be adversely affected by the increase in turbidity.

The sediment transport modeling to date shows that the gradual release of this material would not substantially change the composition of the Ventura River Bed. Plots of the d_{16}, d_{50} and d_{50} are given in Exhibit G, Section 19.4.5. The d_{16} is the diameter of which 16% of the sediment in the bed is finer. The release of sediment from behind the dam does cause the bed to become slightly finer, but the bed remains coarse and composed primarily of cobbles and gravel. In addition, the bed would eventually return to current conditions. The d_{16} would be greater than 6 mm for all times after dam removal in all reaches upstream of River Mile 2. In most reaches, the d_{16} would be above 10 mm for all times above River Mile 2. The d_{35} would be above 35 mm for all reaches above River Mile 2 for all times after dam removal. The d_{50} remains above 60 mm for all reaches above River Mile 2 for all times after dam removal. The silts and clays would not deposit onto the riverbed. Therefore, silt and clay would not enter into the groundwater aquifer or affect percolation of water into the aquifer

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative, which is similar to the increase under Alternative 2a. The sediment supply may be delayed relative to Alternative 2a, however, because the sediment would be temporarily stabilized in the reservoir area. It would be expected that by year 20, the sand supply of Alternative 4b would be very similar to Alternative 2a. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

Alternative 4b has been identified as the Locally Preferred Plan (LPP). Initial incremental analysis has also identified Alternative 4b as the National Economic Development Plan (NER). The flood protection measures for this alternative were revised based upon a risk and uncertainty analysis. The results are shown in Table 8. The row labeled "Current Level of Protection" contains the approximate level of protection under current conditions and the row labeled "Level of Protection – No Mitigation" contains the level of assuming no mitigation measures were constructed. The row titled "Mitigation to Current Level" shows the height requirements of the new levees and the additional height requirements for existing levees to maintain their respective level of protection. The row titled "Levee Height to Mitigate Impacts and Provide 100-yr FEMA Level" shows the height requirements for new levees and the height additions to existing (upgrade) levees to have FEMA certification. This is based upon 95% chance of non-exceedance). The 95% non-exceedance value is used instead of the typical 90% non-exceedance value due to the large uncertainty associated with dam removal.

There are five locations identified: Hot Springs is located at approximate RM 16. Camino Cielo is at RM 15.5, near the Camino Ceilo bridge. Meiner Oaks is at approximately RM 14, just downstream of Robles Diversion. Live Oak is the town just upstream of Santa Ana Blvd. There is a current levee approximately 1 mile long that protects the town of Live Oak. There is another current levee at Casitas Springs from RM 7.8 to RM 6.8.

Note that Live Oak currently has over 100-year protection, so the mitigation levee would be greater than the 100-year FEMA requirement levee height. The difference would be two feet of levee height. The Camino Cielo site has bank overflow at the 10-year event, however damages to structures/crops do not occur until after the 50-year event for the without project condition.

At Hot Springs and Camino Cielo, preliminary planning and economic screening evaluation indicated that property purchase rather than levee construction would be the most appropriate alternative. Therefore, the alternative of levee construction was dropped from further consideration, and estimate of levee height was prepared for these two locations.

Table 8. Levee Recommendations Based on Risk and Uncertainty Analysis for Alternative 4b.

Location Description	**Hot springs**	**Camino Cielo**	**Meiners Oaks**	**Live Oak**	**Casitas Springs**
HEC-RAS Stationing	16.1932	15.5303	13.7311	9.5644	7.3844
Current Level of Protection	~100-yr	50-yr	100-yr	> 100-yr	50-yr
Level of Protection - No Mitigation	10-yr	10-yr	50-yr	20 yr	< 10-yr
Extent of Levee Construction	Purchase Property	Purchase Property	New	Upgrade	Upgrade
Levee Height to Mitigate Impacts to Current Level of Protection (ft)	-	-	5	6	3
Levee Height to Mitigate Impacts and Provide 100-yr FEMA Level	-	-	5	4	5

Summary Tables

Below are summary tables of the impacts associated with each alternative. Table 9 contains average deposition expected in each project reach. No results are shown for Reach 1 because the model results are not applicable for the estuary region. Table 10 contains impacts at the flow diversion along the Ventura River and sediment delivery to the ocean for each alternative. Table 11 contains the impacts associated with each alternative when a sediment bypass structure is built at Robles Diversion and subsurface wells are constructed at Foster Park to replace the surface diversion there.

Table 9. Summary Table of Deposition for All Alternatives. Results are at Year 50 of a 50-yr simulation.

	Alternative						
Location	**No Action**	**1, 4a**	**2a**	**2b**	**3a**	**3b**	**4b**
Reach 2 (ft)	1.5	2.2	3.4	3.6	3.5	3.6	3.6
Reach 3 (ft)	1.9	2.6	4.2	4.5	4.2	4.5	4.2
Reach 4 (ft)	-0.2	0.7	2.0	1.6	2.2	1.6	2.3
Reach 5 (ft)	-1.6	0.6	2.1	1.9	2.0	1.3	2.2
Reach 6a (ft)	-1.9	4.7	5.0	5.7	5.8	4.1	6.4
Reach 6b (ft)	-2.0	0.5	0.6	3.0	1.1	1.1	0.9

Table 10. Summary Table of Impacts at Diversions and At Ocean **without** Mitigation Measures.

Impact	***No Action***	***Alternatives 1, 4a***	***Alternatives 2a,3a***
Deposition at Robles Diversion	No Change for 40 years	Twice-current levels.	For the first 2 to 3 floods, the deposition may affect diversions. Stabilize at twice-current levels.
Turbidity Impact at Robles	Stabilize at 30 % increase within 10 years	Increase by average of 30%, but within natural variability.	For the first 2 to 3 floods the concentrations would increase by factor of 2 to 10, then stabilize at 30 % increase.
Turbidity Impact at Foster Park	No significant change	Increase by average of 30%, but within natural variability.	May increase period of missed surface diversion
Ocean Delivery	No Change for approximately 50 years	Increase sand delivery by approximately 20 % over 50 yr period	Increase sand delivery by approximately 32 % over 50 yr period
Impact		***Alternatives 2b,3b***	***Alternative 4b***
Deposition at Robles Diversion		For the first 2 to 3 floods, the deposition may affect diversions. Stabilize at twice-current levels.	Each flood following a removal of revetment may affect diversions. Stabilize at twice-current levels.
Turbidity Impact at Robles		For the first 2 to 3 floods, the concentrations would be at least 10 to 100 times higher than current, and then stabilize at 30 % increase. De-silting Basin would be required to mitigate concentrations.	Each flood following a removal of revetment would increase the turbidity by factor of 2 to 10, and then stabilize at 30 % increase. When revetments are not removed, similar to Alt 1, 4a.
Turbidity Impact at Foster Park		May increase period of missed surface diversion	May increase period of missed surface diversion
Ocean Delivery		Increase sand delivery by approximately 37 % over 50 yr period	Increase sand delivery by approximately 32 % over 50 yr period

Table 11. Summary Table of Impacts at Diversions and At Ocean **with** Mitigation Measures*.

Impact	*No Action*	*Alternatives 1, 4a*	*Alternatives 2a,3a*
Deposition at Robles Diversion	No Change for 40 years	Similar to current levels.	For the first 2 to 3 floods, the deposition would be larger than normal, but would not affect diversions. Stabilize at current levels.
Turbidity Impact at Robles	Stabilize at 30 % increase within 10 years	Increase by average of 30%, but within natural variability.	For the first 2 to 3 floods the concentrations would increase by factor of 2 to 10, then stabilize at 30 % increase.
Turbidity Impact at Foster Park	No significant change	Would not affect diversions	Would not affect diversions
Ocean Delivery	No Change for approximately 50 years	Increase sand delivery by approximately 20 % over 50 yr period	Increase sand delivery by approximately 32 % over 50 yr period
Impact		*Alternatives 2b,3b*	*Alternative 4b*
Deposition at Robles Diversion		For the first 2 to 3 floods, the deposition would be larger than normal, but would not affect diversions. Stabilize at current levels.	Each flood following a removal of revetment may increase deposition but would not affect diversions. Stabilize at current levels.
Turbidity Impact at Robles		For the first 2 to 3 floods, the concentrations would be at least 10 to 100 times higher than current, and then stabilize at 30 % increase. De-silting Basin would be required to mitigate concentrations.	Each flood following a removal of revetment would increase the turbidity by factor of 2 to 10, and then stabilize at 30 % increase. When revetments are not removed, similar to Alt 1, 4a.
Turbidity Impact at Foster Park		Would not affect diversions	Would not affect diversions
Ocean Delivery		Increase sand delivery by approximately 37 % over 50 yr period	Increase sand delivery by approximately 32 % over 50 yr period

*Mitigation measures include a sediment bypass structure and subsurface wells to replace surface diversion at Foster Park.

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1. Introduction

The Bureau of Reclamation (Reclamation) is providing technical assistance in the Matilija Dam Ecosystem Restoration Feasibility Study -- a cost-shared study between the Corps of Engineers (Corps) and Ventura County Flood Control District (District). In addition to geotechnical, surveying, and mapping tasks, the District requested Reclamation to perform the hydrology, hydraulics, and sedimentation analyses as in-kind credit in this feasibility study. Technical assistance by Reclamation has been funded through an Interagency Agreement with the State Coastal Conservancy. Work elements associated with this task are consistent with items delineated in the Corps Project Management Plan (PMP). To ensure successful achievement of certain items described in the PMP, Reclamation requested assistance from U.S. Geological Surveys (USGS) that will complement investigations and provide deliverables in 2003.

Included in this report are results from the sediment studies for the alternatives analysis for the Matilija Dam Ecosystem Restoration Study, Ventura, California. The information contained herein will be used in hydrologic, hydraulic, and sedimentation modeling to evaluate the impacts of various alternatives of restoring the ecosystem. Sections 1 to 5 describe the Without-Project Conditions, while sections 6 to 10 describe the With-Project Conditions. This report is the final submittal of the results from the hydrology, hydraulics, and sedimentation studies supporting the final feasibility report at the F8 milestone for the Matilija Dam Ecosystem Restoration Study, Ventura, California.

Any use of trade, product, or firm names in this publication is for descriptive purposes only and does not imply endorsement by the U.S. Government.

1.1. General River and Watershed Description

The Project Location is shown in Figure 1.2. The Ventura River Watershed is shown in Figure 1.4 and Figure 1.3. Section 25 titled "Exhibit M. Location of Cross Section Used in Study" contains a larger map with the River Mile (RM) indicated on the map. The Ventura River starts at the confluence of Matilija Creek and North Fork Matilija Creek, approximately 0.6 miles downstream of Matilija Dam. Several smaller watersheds enter the Ventura River upstream of the next major tributary, San Antonio Creek. Coyote Creek then enters Ventura River from the west just downstream of the confluence with San Antonio Creek. Casitas Dam regulates the flows on Coyote Creek. Downstream, Cañada Larga enters from the east and Cañada de Rodriguez and Cañada del Diablo enter from the west. The drainage basin characteristics associated with the major sub-areas and the minor drainages are given in Table 1.3. Over 75% of the Ventura River Watershed is classified as rangeland covered with shrub and brush and 20% of the watershed is classified as forested. In general, the highest sediment producing parts of the watershed are those covered in shrub and brush and are located in the upper parts of the watershed where slopes are greater and annual rainfall is larger. Nearly 45% of the watershed may be classified as mountainous, 40% as foothill, and 15% as valley area (Reclamation, 1954).

For the purposes of this study, reaches have been defined so that within a given reach, the river and associated habitat has similar characteristics (Table 1.1 and Figure 1.1). The reach definitions in are used in this report to describe sediment impacts and are referenced throughout the report.

The locations of several landmarks along the river are given in Table 1.2. There are eight major bridge crossings between Matilija Dam and the ocean, three levees, and two water diversions. There is extensive development along the river with several businesses and communities are located in areas where flooding has previously occurred. Many of these developments are now protected by levees.

Table 1.1. Major Reaches of Matilija Creek and the Ventura River.

Reach #	River Mile	Reach
8	30 – 17.46	Matilija Creek
7b	17.46 – 16.76	Matilija Delta
7a	16.76 – 16.46	Matilija Reservoir
6b	16.46 – 15	Downstream of Matilija Dam to Canyon opening
6a	15 – 14.15	From Canyon opening to upstream of Robles Diversion
5	14.15 – 11.27	Near Robles Diversion to Baldwin Road Bridge
4	11.27 – 7.93	Baldwin Road Bridge to San Antonio Creek Confluence
3	7.93 – 5.95	San Antonio Creek Confluence to Foster Park Bridge
2	5.95 – 0.60	Foster Park Bridge to Main St Bridge
1	0.60 – 0.0	Estuary

Table 1.2. Landmarks along River.

Landmark	River Mile
Upstream End of Matilija Reservoir Delta	17.46
Upstream End of Matilija Reservoir	16.76
Matilija Dam	16.46
Matilija Road Bridge	15.88
Matilija Creek confluence with N. Fork Matilija Creek	15.8
Los Robles Diversion Dam	14.15
Baldwin Road	11.27
End of Live Oaks Levee	10.29
Beginning of Live Oaks Levee	9.39
Santa Ana Blvd	9.38
Confluence of Ventura River and San Antonio Creek	7.93
End of Casitas Levee	7.85
Beginning of Casitas Levee	6.84
Foster Park Diversion	6.31
Confluence of Ventura River and Coyote Creek	6.24
Casitas Vistas Road (USGS stream gage)	5.95
Confluence of Ventura River and Cañada Larga	4.63
Shell Road	3.16
End of Ventura River Levee	2.38
Main Street	0.6
Ventura Freeway (Highway 101)	0.45
Southern Pacific Railroad	0.19
Beginning of Ventura River Levee	0
Ventura River Mouth	0

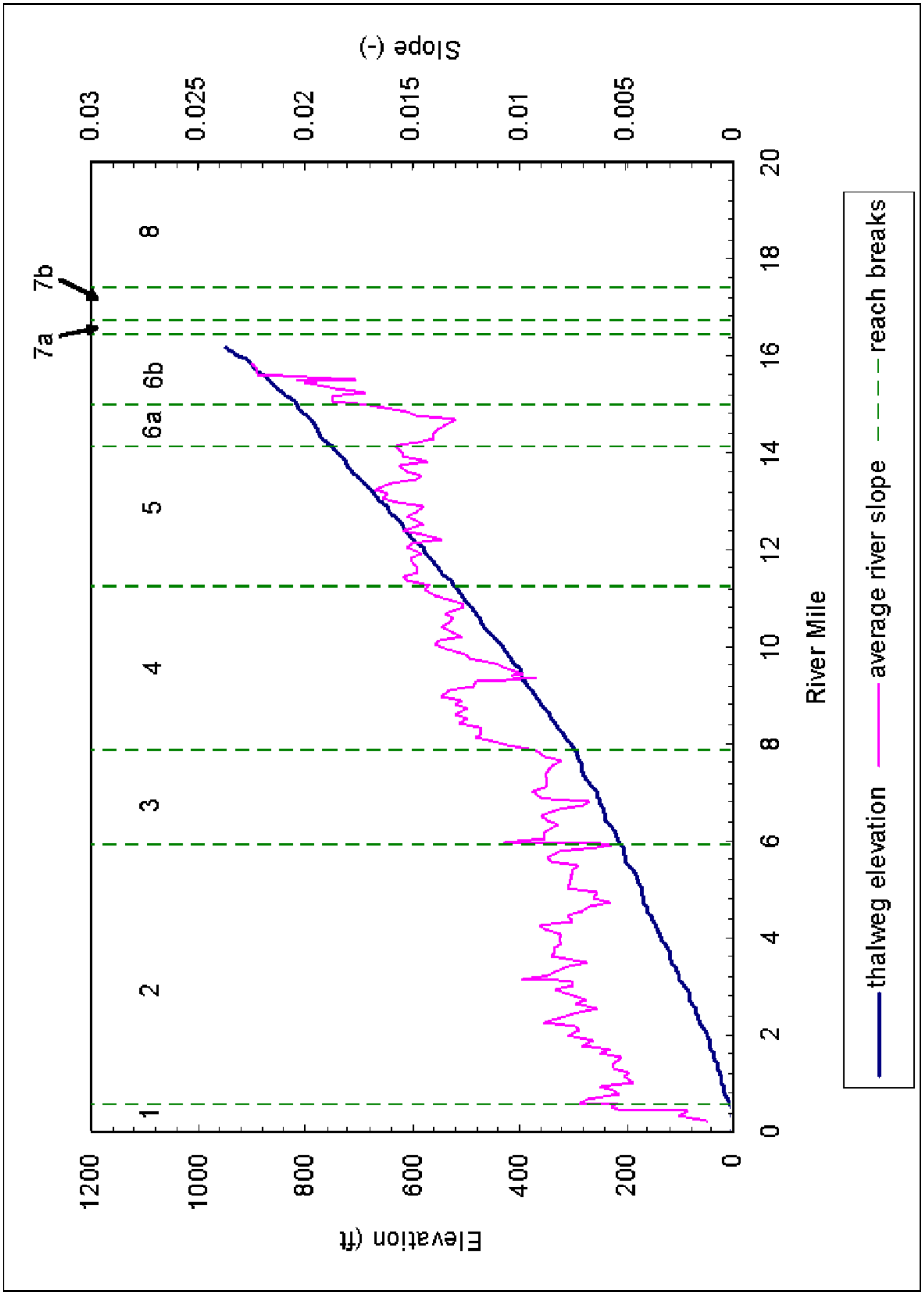


Figure 1.1. Bed Profile and Reach Definitions in the Ventura River.

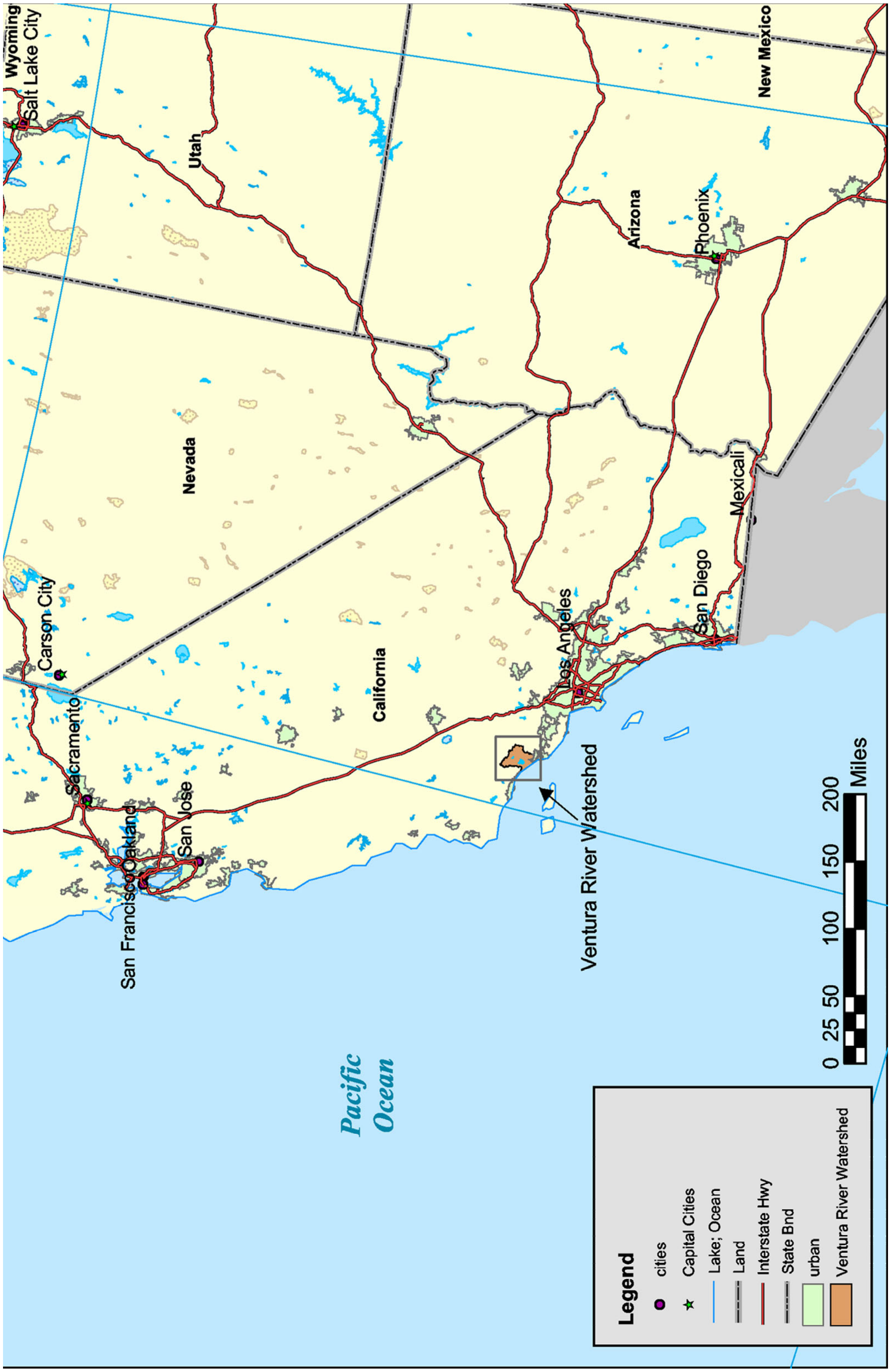


Figure 1.2. Large Scale Map Showing Project Location.

Figure 1.3. Ventura River Basin. From USGS 1:250,000 Scale Los Angeles, California Map.

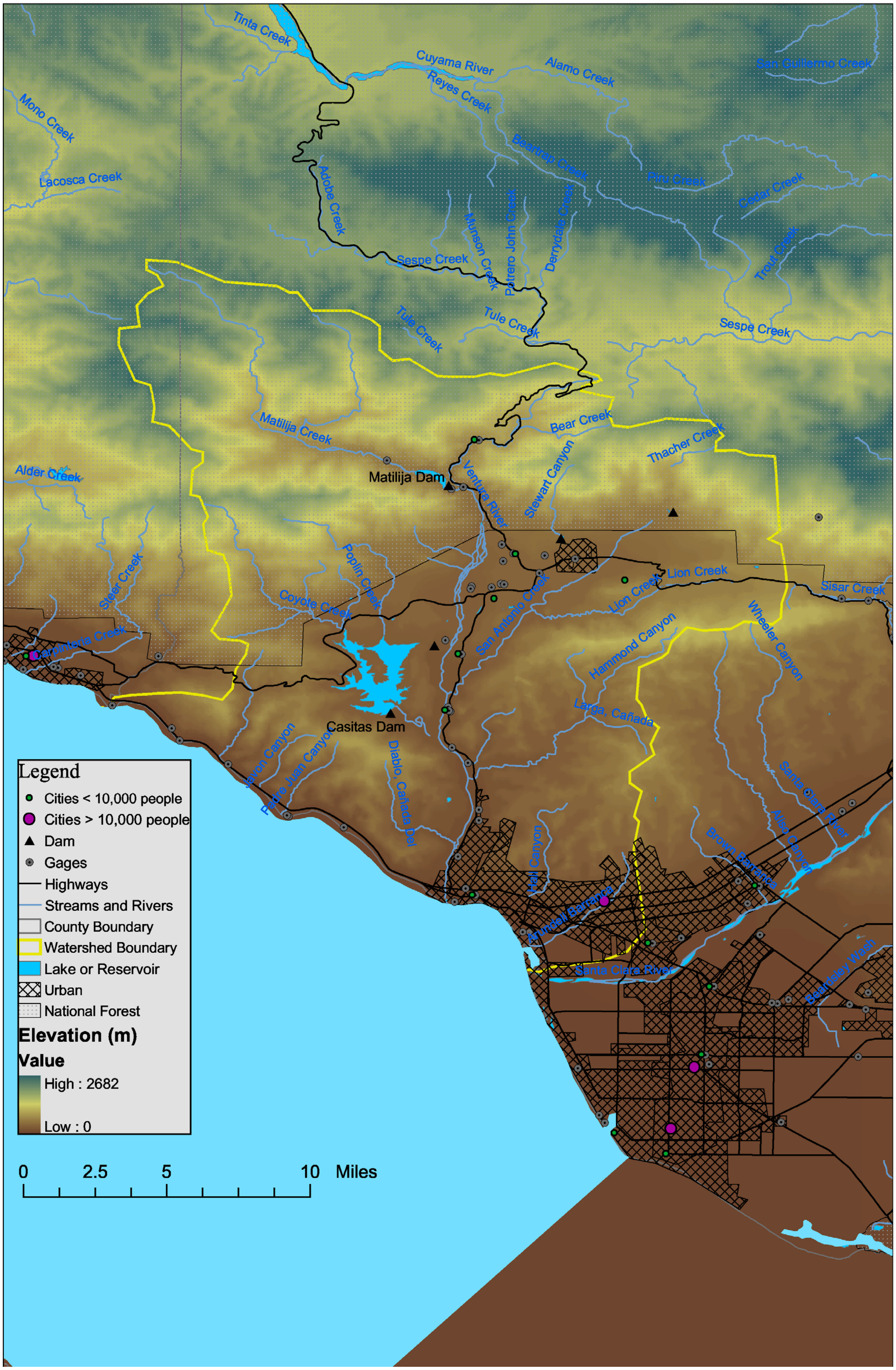


Figure 1.4. Ventura River Watershed.

Figure 1.5. Aerial View of Matilija Dam (taken April 2004).

Table 1.3. Drainage Areas of Sub-Watersheds in the Ventura River Watershed.

Local Area Watershed Name	Drainage Area (mi^2)	Maximum Length of Watershed (feet)	Minimum Elevation of Watershed (feet)	Maximum Elevation of Watershed (feet)	Mean Annual Precipitation (inches)
Matilija at Matilija Dam	54.6	83363	1009.3	5456.8	23.5
North Fork Ventura River - Matilija	16.2	40554	1009.3	5006.7	22.1
Ventura River D/S of Willis Canyon	7.4	22090	696.9	4278.6	20.2
Ventura River at Live Oak Creek	11.6	45685	290.6	2310.0	17.8
San Antonio Creek	51.0	79331	290.4	5410.7	18.3
Santa Ana Creek at Lake Casitas	9.5	38211	528.6	4645.9	18.7
Coyote Creek above Lake Casitas	13.4	36127	560.9	4769.5	21.1
Drainage area that includes Lake Casitas	15.3	31470	515.0	2342.6	18.2
Ventura River Sub area to Foster Park	9.3	25313	195.4	1302.8	17.3
Cañada Larga Sub area	19.3	50752	195.8	2788.0	17.9
Lower Ventura River Sub area	15.5	35470	0.00	2117.6	16.9
Entire Ventura River Watershed	223.1		0.0	5456.8	19.9

1.2. Geology

The drainage watershed of Matilija Dam is primarily composed of Tertiary marine sandstone and shale of the Juncal Formation, the Matilija Sandstone, and the Cozy Dell Shale with small areas of unnamed Cretaceous marine strata (Dibblee, 1985a; 1985b; 1987a; 1987b). Matilija Dam is founded in the Matilija Sandstone and the reservoir area is predominantly underlain by Juncal Formation with a smaller area of Matilija Sandstone. Downstream of the dam the river canyon is cut in Matilija Sandstone (Dibblee, 1987b). The river valley widens downstream where it flows through Cozy Dell Shale (Dibblee 1987b).

Downstream of Matilija Dam, Rockwell et al. (1984 and 1988) have documented rapid uplift of marine terraces and fluvial terraces associated with the Ventura River. It appears that the rate of incision of the Ventura River has kept pace with the rate of uplift in this area (Rockwell et al., 1984). The closest identified geologic structure associated with active uplift is the Arroyo Parida Fault about five miles (RM 11) downstream of Matilija Dam (Rockwell et al., 1984; Dibblee, 1987b). Rockwell et al. (1984) conclude that incision rates upstream of this geologic structure are about 0.8 mm/yr.

1.3. Climate

1.3.1. Rainfall

The average annual rainfall for each drainage basin is shown in Table 1.3. In general, the higher elevations receive more rain. The average annual rainfall near the mouth of the Ventura River is approximately 16.9 inches per year. The average annual rainfall of the drainage basin upstream of Matilija Dam is 23.9 inches per year. The average for the entire watershed is approximately 20 inches per year.

There is extreme seasonal variation in the rainfall and over 90% of the rainfall occurs during the six months between November and April (Figure 1.6). The source of the rainfall data is the National Climatic Data Center (NCDC, http://lwf.ncdc.noaa.gov/oa/ncdc.html) rain gages in the cities of Ventura and Ojai. The period of record was from as early as 1874 until as late as 1995. The flows in the river show the same trend, but lag in time. This lag is due to the storage capacity of the soil in the watershed.

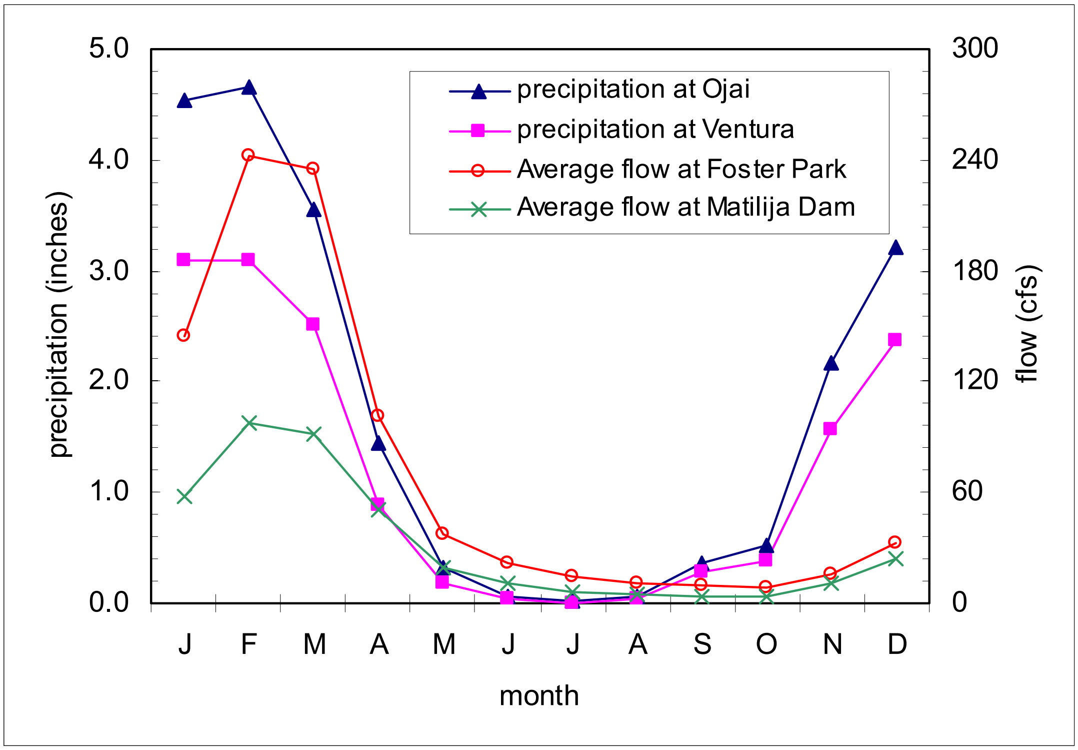


Figure 1.6. Seasonal variation of average rainfall and flow in Ventura River Watershed.

1.3.2. TEMPERATURES

The temperature characteristics near the cities of Oxnard (located approximately 8 miles SE of Ventura) and Ojai (located approximately 12 miles North of Ventura) are shown in Figure 1.7 and Figure 1.8, respectively. Due to the regulating presence of the ocean, the temperature near the ocean has generally smaller seasonal and daily variations. The mean high varies between 64 F in the winter months to a mean high of 76º F in the summer months. The mean low varies between 44º F in the winter months to 60º F in the summer months. Further inland, at Ojai, the mean highs varying between 64º F and 90º F, while the mean lows vary between 36º F and 56º F.

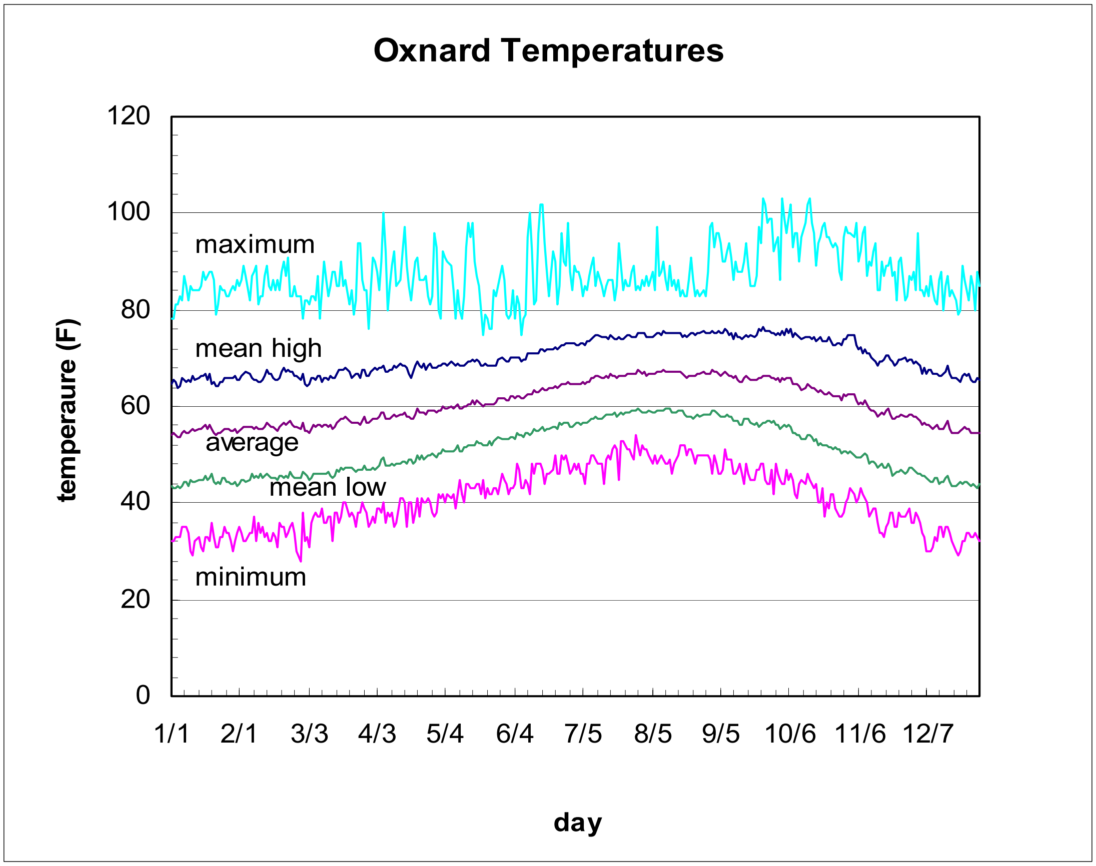


Figure 1.7. Temperatures at Oxnard (34º 11' 00" N 119º 10' 00"W, El. 49 ft) for the period of 1948 to 2000.

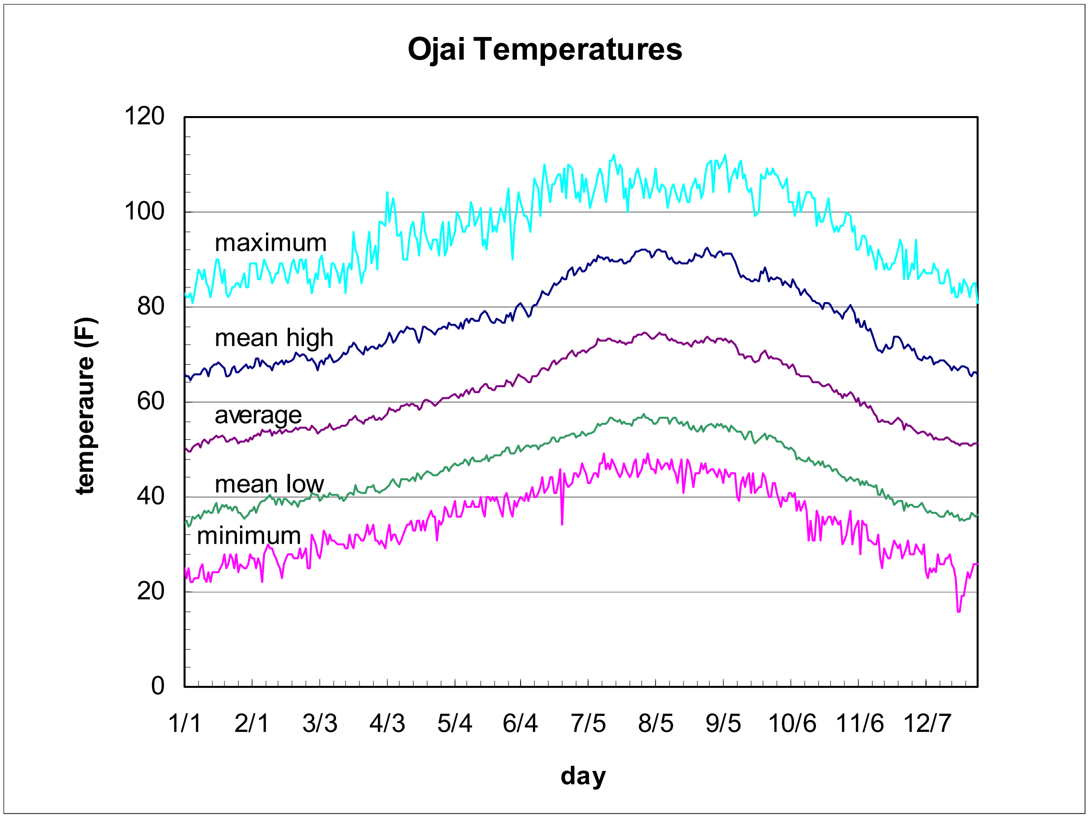


Figure 1.8. Temperatures at Ojai (34º 26' 00"N, 119º 13' 00"W, El. 750 ft) for the period 1948 to 2000.

1.4. Structures Affecting Runoff

1.4.1. DAMS AND DIVERSIONS

Matilija Dam

Several structures affect the flow in the Ventura Watershed. Matilija Dam was built in 1947 with an initial reservoir capacity of 7,018 ac-ft and impounds Matilija Creek. Matilija Reservoir currently has less than 500 ac-ft of capacity remaining and its ability to trap sediment and attenuate floods has been significantly decreased. Its present sediment trap efficiency is estimated to be 45% (Table 5.4). There are no written operating criteria for Matilija Reservoir, other than CMWD's (Casitas Municipal Water District) criteria for the operation of Robles stated below. The general operating criteria for the reservoir is to maintain outflow equal to inflow when diversions are not taking place at Robles Diversion Dam. When diversions are being performed at Robles Diversion Dam, the reservoir level is cycled to produce larger flows in the Ventura River to optimize the amount of the diversion. There is a 36-inch, a 12-inch, and a 6-inch release valve at Matilija Reservoir with the potential to release a maximum of 250 ft^3/s.

Casitas Dam

Casitas Dam, which dams Santa Ana and Coyote Creeks, was built in 1958 with an initial reservoir capacity of 250,000 ac-ft. Casitas Dam was built as part of the Ventura River Project by Reclamation. A schematic of the project is shown in Figure 1.10. Prior to Casitas Dam, Coyote Creek contributed 18% of the flow in the Ventura River at Foster Park. After construction, significant flow downstream of the Casitas Dam in Coyote Creek only occurred during wet years in which water is spilled from the reservoir. As a result, Coyote Creek contributed only 5 % of the flow in the Ventura River during the period 1971-1980. Casitas Dam effectively traps all the sediment that enters into the reservoir.

Casitas Reservoir yields approximately 21,500 ac-ft/yr of water and an additional 8,000 ac-ft is lost to evaporation and seepage. Based on this, the average detention time of water in the reservoir is 8.5 years. A record of the storage in Casitas Reservoir is given in Figure 1.9. Note that the storage in the lake dropped below 150,000 ac-ft only once since its original filling. However, the storage in the lake is currently at its lowest level since January 1992.

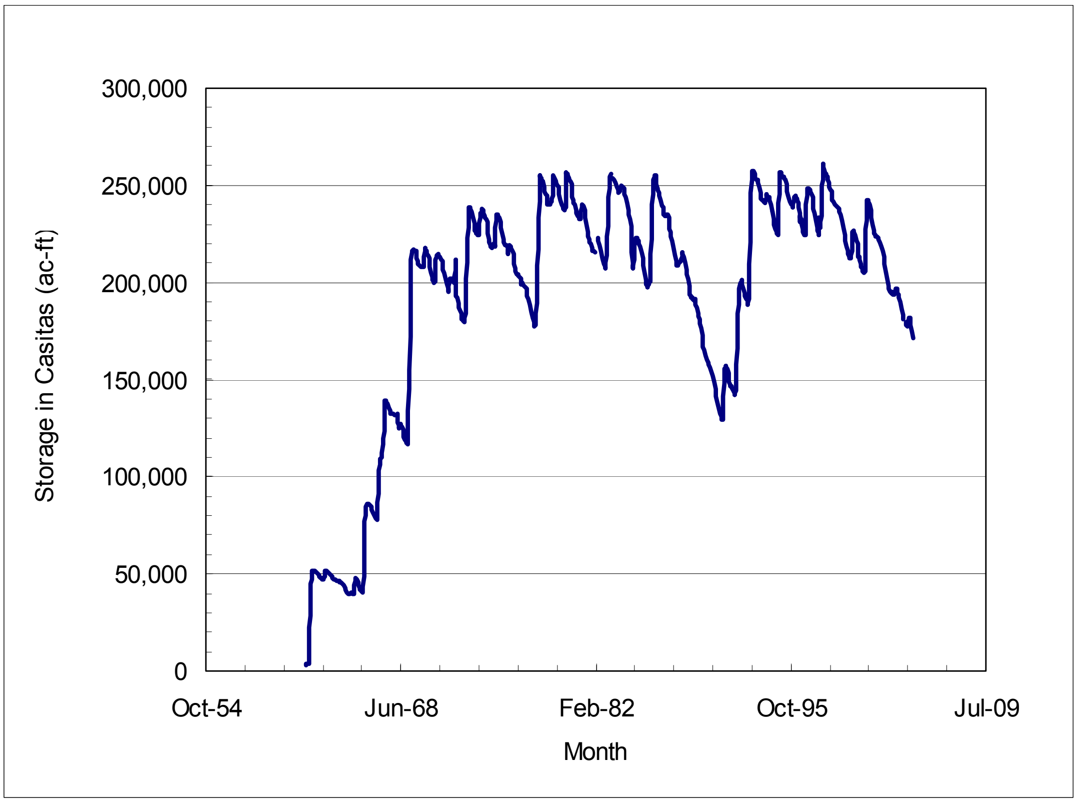


Figure 1.9. Storage in Lake Castitas.

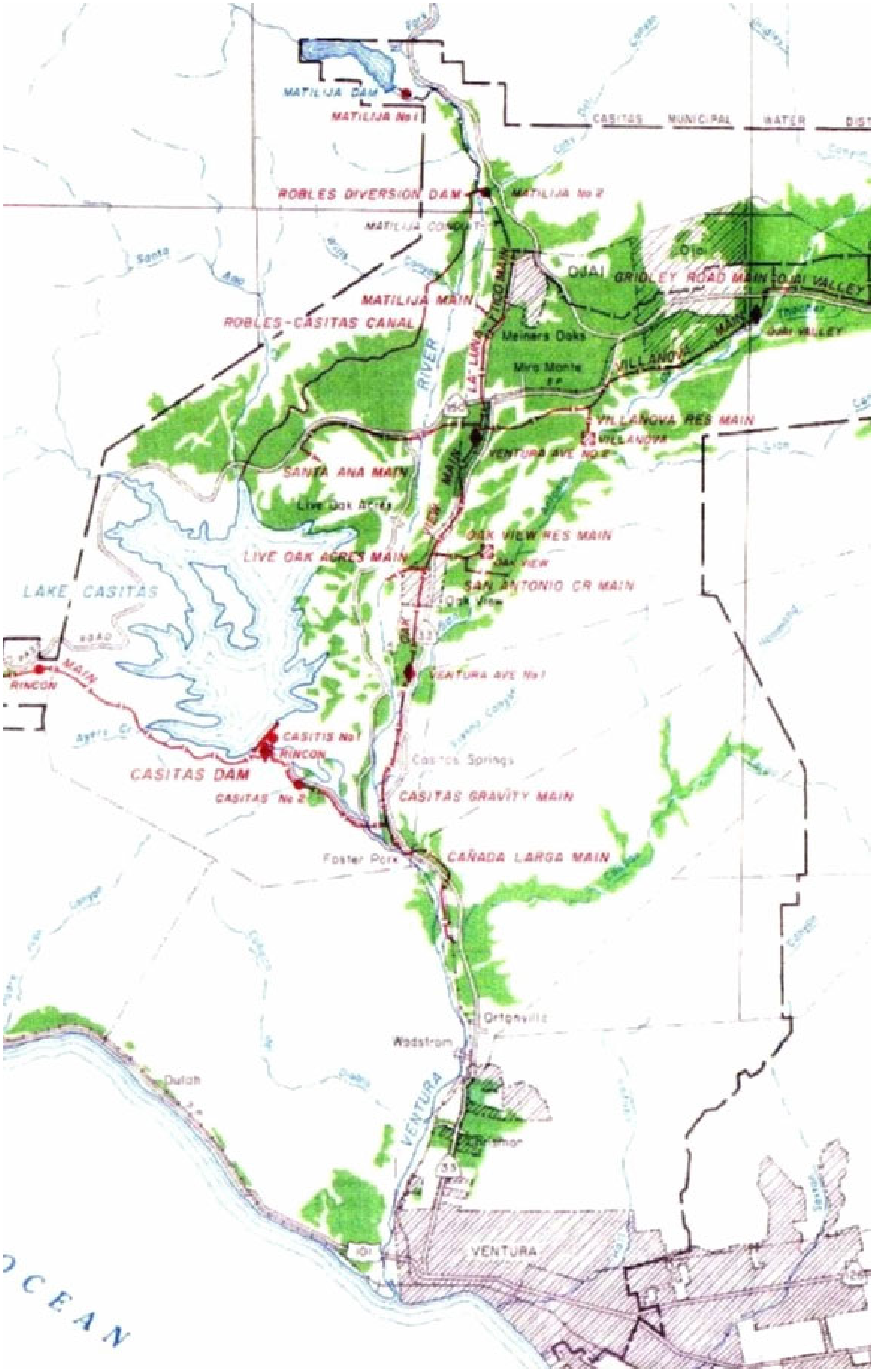


Figure 1.10. Schematic of the Ventura River Project (from Reclamation web site, http://dataweb.usbr.gov/).

Robles Diversion Dam

Robles Diversion Dam was built in 1958 and diverts water from the Ventura River into Casitas Reservoir. Most of the diversion at Robles Diversion Dam occurs from December through March and is highly variable. CMWD's ability to regulate the flows in Matilija Creek is significantly impaired because of the limited storage capacity of Matilija Reservoir. The maximum diversion rate at Robles Diversion Dam is approximately 500 ft^3/s (Chris Morgan, CMWD). In dry years, there is often almost no diversion because the diversion is currently subject to the following operating criteria (CMWD):

> Commencing with 1959-1960 water year, the following criteria will govern the operation of Robles Diversion Dam:
>
> In general, when the natural flow of the Ventura River at the Robles Diversion Dam is less than 20 ft^3/s, the entire flow will be passed down river and when the natural flow is greater than 20 ft^3/s, no less than 20 ft^3/s will be passed down river; provided that such release down river shall be increased or decreased under the following circumstances:
>
> 1. If the water level in the river gravels fails to rise to the extent that would be expected under natural conditions for the time of year and type of year as evidenced by periodic measurements of wells along the river, the release shall be increased to correct this condition.
>
> 2. If surface flow occurs at Santa Ana Boulevard, river releases shall be decreased appropriately.
>
> 3. If rising water above the mouth of San Antonio Creek occurs in such amounts that it is apparent that water will waste to the ocean, the river release shall be decreased so that such waste shall not occur.
>
> Under integrated project operation, flood flows temporarily stored in Matilija will be released down river for diversion to Casitas Reservoir at the Robles Diversion Dam. Such operational releases will be deducted from the total flow at Robles in order to determine the amount of natural flow available for release at the Robles Diversion Dam.

These operating rules may be modified based on a new fish passage study to the following:

- Diversions will typically occur Dec to March, but can on occasion can occur between July-Dec.
- The Low Season flows will be between June 1 – Oct 31
- The Fish Passage Augmentation Season will be between Jan 1 and June 30, but will officially start after sand bar breached at least once.
- The Minimum Fish Migration Flow will be 50 cfs.
- The Fish way Operating Criteria is to release a minimum of 50 cfs during the first 10 days of each flood of 150 cfs or greater. The flows will be gradually decreased to 30 cfs over a 12-day period.

A fish ladder is under construction at Robles Diversion that will be completed before Nov 2004. An aerial view is given in Figure 1.11.

Robles Diversion is subject to larges amounts of sediment deposition during floods. It is not large enough to trap the suspended material transported by the river, but it does trap a significant portion of the bed load. In the Ventura River, the suspended material is mostly clays, silts, and sands, while the bed load is composed of gravels, cobbles, and boulders.

The record of CMWD's sediment removal is in Table 1.4. Significant sediment removal is necessary after every major flood. CMWD has recorded a total of 419,000 yd^3 of sediment removed from the period from 1966 to 1998. Each removal was 46,000 yd^3 on average. There was a major flood in 1969 and the amount removed was not recorded, but it is estimated that it would have been near 100,000 yd^3, because it was of similar magnitude to the 1978 flood in which 91,000 yd^3 was removed. The 1993 removal was not recorded either and it is estimated that approximately 40,000 yd^3 was removed in 1993. Adding the estimated removal amounts in 1969 and 1993 brings the total sediment removed from 1958 to 2000 to 559,000 yd^3 (346 ac-ft). The amount of sediment removed averages 13,300 yd^3/yr (8 ac-ft/yr) if it is assumed that no sediment removed occurred prior to 1966 and that the Diversion was built in 1958.

For comparison purposes, approximately 1,400,000 yd^3 of material of gravel sized or coarser was deposited behind Matilija Dam during this same period. This is approximately 2.5 times what was deposited behind Robles.

Table 1.4. Record of Sediment Removal at Robles Diversion Dam.

Year	Amount of Sediment Removed (yd^3)
1966	30,000
1969	Data Not Available
1973	50,000
1978	91,000
1980	71,000
1983	57,000
1986	30,000
1991	20,000
1993	Data Not Available
1995	35,000
1998	35,000

Figure 1.11. Aerial View of Robles Diversion under Construction in April 2004.

Foster Park Diversion

The City of Ventura diversion structure is located at Foster Memorial Park. An underground dam extending most of the way from the surface to bedrock forces water to the surface at the location. Part of the diversion is surface water and part is subsurface. ENTRIX (1997) states that, on average, 2,500 ac-ft of surface water and 3,900 ac-ft of groundwater is diverted at Foster Park annually. The “surface diversion” is an actually a combination of a shallow intake pipe buried approximately 4 feet below the surface and a surface diversion cam. The record of the annual diversion volumes is given in Figure 1.12. The surface diversion has not been used since 2000 because the river shifted and abandoned the channel leading to the surface diversion.

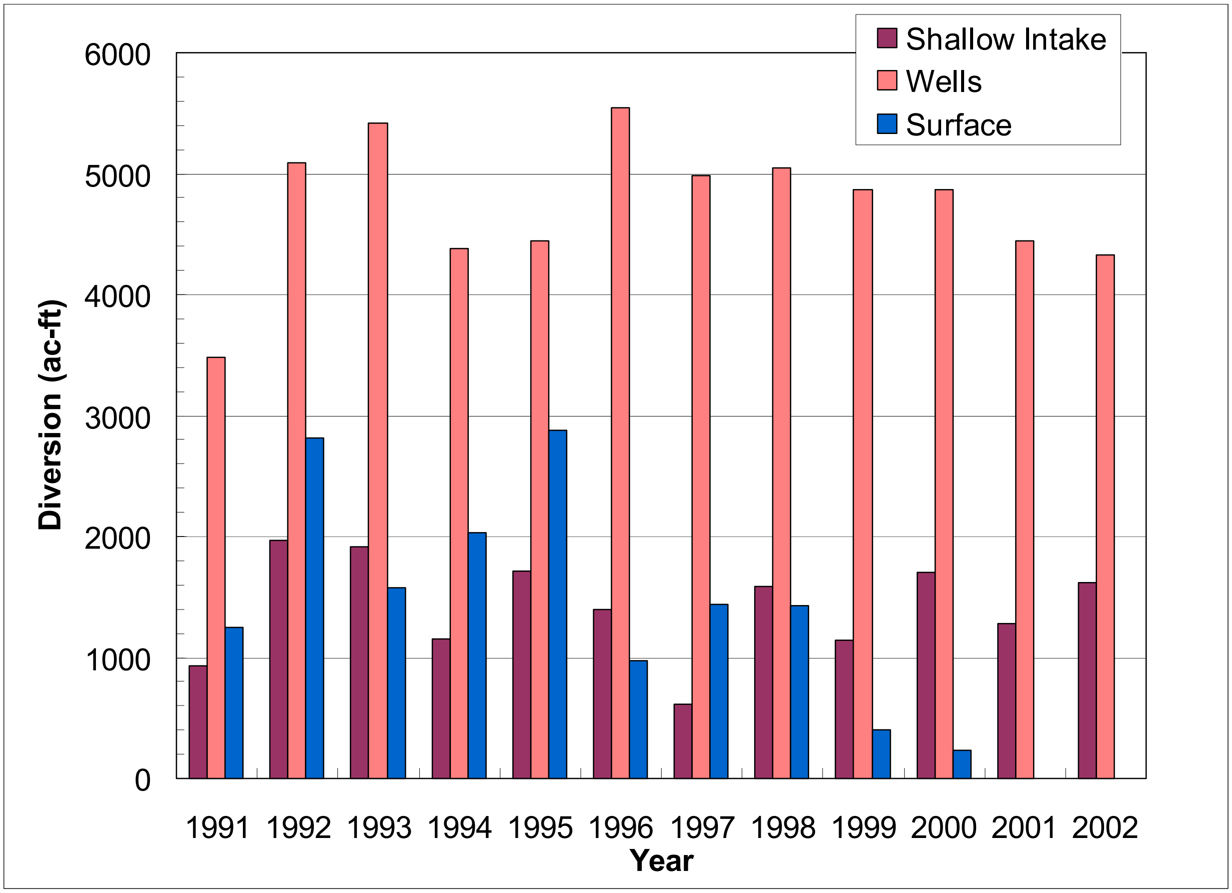


Figure 1.12. Annual Surface Diversions at Foster Park on the Ventura River.

1.4.2. WASTEWATER PLANTS

Ojai Valley Sanitary District (OVSD) Wastewater Treatment Plant was constructed in 1963 with 1.5 million gallons per day (mgd) capacity and expanded in 1965 to its current capacity of 3 MGD (4.64 ft^3/s). It was upgraded to tertiary treatment in 1997. Based on their release data from 1990 to 2001, they released treated sewage at an average rate of 2.31 ft^3/s into Ventura River approximately ½ mile downstream of Foster Park. The average discharge for each day of the year for the period 1990 to 2001 is shown in Figure 1.13.

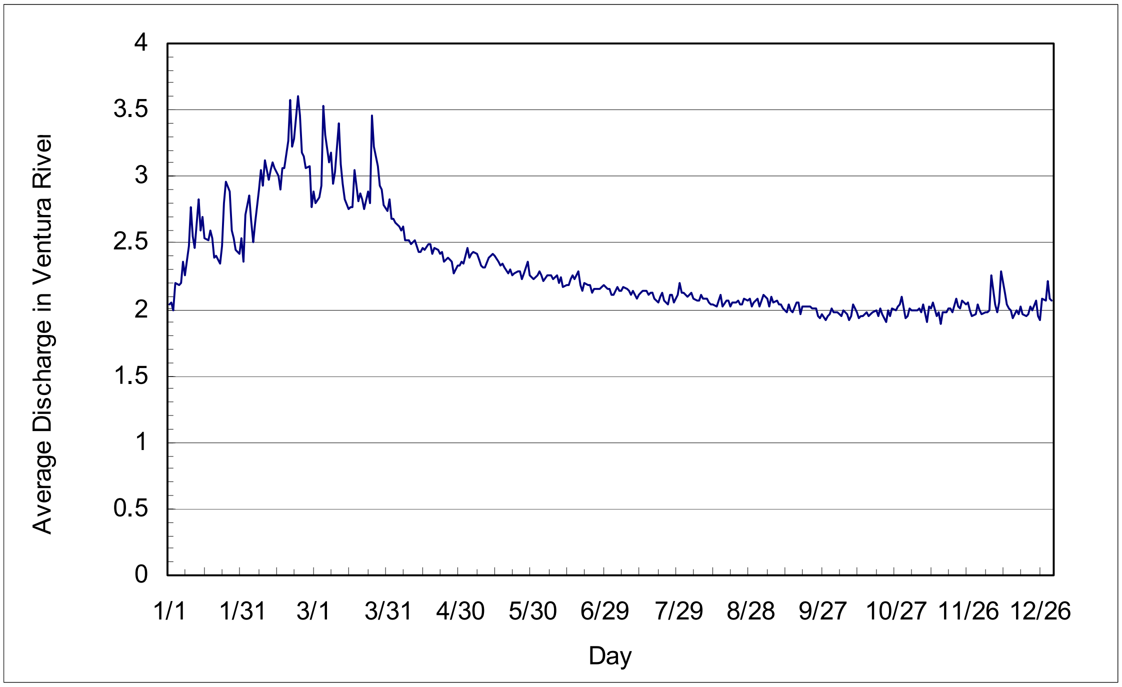


Figure 1.13. Average Discharge of the OVSD Wastewater Treatment Plant for Each Day of the Year.

1.4.3. LEVEES

There are three major levees along the Ventura River and their characteristics are shown in Table 1.5. The most upstream levee is near the Santa Ana Bridge. It protects the Live Oak community along the west bank. The Casitas Springs Levee is along the east bank and protects the town of Casitas Springs. The Ventura Levee is along the east bank and protects the city of Ventura.

Table 1.5. Levee Characteristics along the Ventura River.

Levee	**Ventura**	**Casitas Springs**	**Live Oaks**
Year Constructed	1947	1978	1995
Downstream River Mile (mi)	0	6.84	9.39
Upstream River Mile (mi)	2.38	7.85	10.29
Downstream Elevation (ft)	14.4	267.4	412.2
Upstream Elevation (ft)	120.0	307.6	465.5

1.4.4. DEBRIS BASINS

There are four debris basins in the Ventura River watershed and their properties are listed in Table 3. McDonald and Dent Canyons are direct tributaries to the Ventura River, while Stewart Canyon is a tributary to San Antonio Creek.

Table 1.6. Debris Basin characteristics in the Ventura River Watershed (NA = not applicable).

Characteristic	**McDonald Detention Basin**	**San Antonio Creek Debris Basin**	**Stewart Canyon Debris Basin**	**Dent Debris Basin**
Year Constructed	1998	1986	1963	1950, 1981
Location (approximate State Plain coordinates)	N 1,991,083 E 6,177,000	N 6,199,062 E 1,994,583	N 1,991,581 E 6,184,900	N 1,934,162 E 6,172,619
Watershed area (acres, mi^2)	565 (0.88)	6280 (9.8)	1266 (1.98)	27 (.042)
Level Capacity (yd^3)	23,400	14,600	104,215	4,100
Maximum Debris Capacity (yd^3)	NA	30,000	328,300	4,100
Spillway elevation, NGVD 29 (ft)	816	970	920	143.4
100-yr Flow (ft^3/s)	1,252	5,800	2,642	82
100-yr debris (yd^3)	20747	455600	209000	1,624
50-yr debris (yd^3)	15862	480200	157000	1,255
25-yr debris (yd^3)	11393	249693	112000	928
Size	20' wide x 24' high	112' wide x 5' high	80' wide x 14' high	80' wide x 3' high

2. Without-Project Hydrology

This section will discuss the hydrology of the Ventura River Basin. It summarizes the information contained in the hydrology reports in Exhibit B.

There are several stream gages in the Ventura River watershed with some having a record extending as far back as 1927 (Table 2.1). Originally, the USGS operated them all, but starting in the 1980s, the District and the Casitas Municipal Water District (CMWD) have operated several of the gages. The operation of some gages has been discontinued for various reasons. The gage above Matilija Dam (11114500) was destroyed in the 1969 flood. The records for gages above Casitas Lake (11117600 and 11117800) are not considered reliable for high flows after 1988 because CMWD took over their operation at that time and is not concerned with recording high flows in this area. Project funds were used to install a new stream gage upstream of Matilija Dam on Matilija Creek (11114495). It started recording data in February 2002.

Table 2.1. Stream Gages in the Ventura River Watershed.

Description	USGS Gage Number	Drainage Area (mi^2)	Period of Record (reason of no record)	Data Source
Matilija Creek Ab Res Nr Matilija Hot Springs Ca	11114500	50.7	1948 - 1969 (destroyed)	USGS
Matilija Creek Near Reservoir near Matilija Hot Springs	11114495	47.8	2002 - present	USGS
Matilija Creek At Matilija Hot Springs	11115500	54.7	1927 - present	USGS and CMWD
North Fork Matilija Crk At Matilija Hot Sprs	11116000	15.6	1928 - present	USGS and County
Ventura R Nr Ojai Ca	11116500	70.7	1911 - 1984 (not maintained)	USGS
Ventura River Near Meiners Oaks Ca	11116550	76.4	1959 - present	USGS and CMWD
Robles Diversion Canal	--	--	1958 - present	CMWD
San Antonio Creek Nr Ojai Ca	11117000	33.7	1927 - 1932 (???)	USGS
San Antonio Creek At Casitas Springs	11117500	51.2	1949 - present	USGS and County
Coyote Creek Near Oak View	11117600	13.2	1958 - 1988 (not reliable)	USGS
Santa Ana Creek Near Oak View	11117800	9.11	1958 - 1988 (not reliable)	USGS
Coyote Creek Nr Ventura Ca	11118000	41.2	1927 - 1982	USGS and CMWD
Ventura R Div Nr Ventura Ca	11118400	--	1969 - present	USGS
Ventura R Nr Ventura	11118500	188	1929 - present	USGS
Ventura R Nr Ventura+ Div. Ca	11118501	188	1932 - present	USGS

2.1. Flood Frequency Analysis

A flood-frequency analysis was performed for the entire length of the Ventura River. Frequency discharges for the 2-, 5-, 10-, 20-, 50-, 100-, and 500-year events were developed. The analysis is detailed in a separate report (Bullard, February 2002). Three stream gage records were used in the initial analysis: Matilija Creek above the Matilija Reservoir (USGS gage 11114500), Matilija Creek at Matilija Hot Springs (USGS gage 11115500) and Ventura River near Ventura (USGS gage 11118500). To determine the selected return period flows, various methodologies were investigated and it was determined that a top-fitting method was most appropriate for the Ventura River. The standard method recommended in Bulletin 17B that uses the Log-Pierson Type III Probability distribution did not fit the data. It is expected that the distribution does not work well in this region of the county because of the peculiarities of the weather patterns. The top fitting method used the 7 largest floods and the frequency of those floods were fit with a regression equation and this regression equation was used to determine the flood magnitudes with a 10-, 20-, 50-, 100- and 500-year return period. To obtain the flood magnitudes with 2- and 5-year return periods, a separate analysis of partial duration series was performed (Bullard, May 2002).

Table 2.2. Recommended Peak Flows for the Ventura River at Existing Stream Gauge Sites.

	Flood Flows at Selected Locations (ft^3/s)					
Return Period (yr)	**Upstream of Confluence with N. Fork Matilija Creek**	**Downstream of Confluence with N. Fork Matilija Creek**	**Baldwin Rd.**	**Casitas Springs**	**Casitas Road Bridge**	**Shell Chemical Plant**
2	3,060	3,250	3,380	4,130	4,520	5,080
5	7,090	7,580	7,910	9,820	11,060	12,250
10	12,500	15,000	16,000	35,200	36,400	41,300
20	15,200	18,800	19,800	44,400	46,400	52,700
50	18,800	24,000	24,800	56,600	59,700	67,900
100	21,600	27,100	28,300	66,600	69,700	78,900
500	27,900	35,200	36,700	89,000	93,100	105,500

Matilija Dam has a negligible impact on the peak flows of large floods (floods with a return interval greater than 5 years). Until the flood of 1969, a stream gage upstream of the dam recorded the peak flows entering the dam. The peak flows recorded for the largest events at the upstream and downstream gage are shown in Table 2.3. Before the 1969 flood, the dam had approximately 3,500 acre-ft of storage remaining and this storage did not attenuate the 1969 flood. In fact, according to stream gage records, the peak flow was larger downstream of the dam than upstream of the dam. The increase could be accounted for by measurement error or due to the slight increase in drainage area downstream of the dam.

Currently, the storage capacity of Matilija Dam is less than 500 acre-ft and the reservoir would quickly fill during a major flood. For example, the 10-year flood peak of 9,900 ft^3/s in Matilija Creek would completely fill a dry reservoir in less than 40 minutes. Therefore, it can be assumed that it provides no practical attenuation of the peak flow for larger flood events.

Table 2.3. Historical Impact of Matilija Dam on Peak Flows in Matilija Creek.

Date	Upstream Flow (ft^3/s)	Downstream flow (ft^3/s)
1/15/1952	8800	3530
4/3/1958	5440	5130
2/16/1959	2500	1990
2/10/1962	6570	5130
12/29/1965	5540	5530
12/6/1966	5190	3410
1/25/1969	19600	20000

The flow in the Ventura River and its tributaries can vary rapidly. A comparison between the instantaneous flow recorded at 15-minute intervals and the daily average flows is shown in Figure 2.1 for the flood of 1992. The daily average recorded flow for February 12, 1992 was 8670 ft^3/s while the peak for that day was 44,200 ft^3/s. Because of these rapid changes, it is important to use the instantaneous flows values recorded in 15-minute increments rather than the daily average flows when simulating sediment transport.

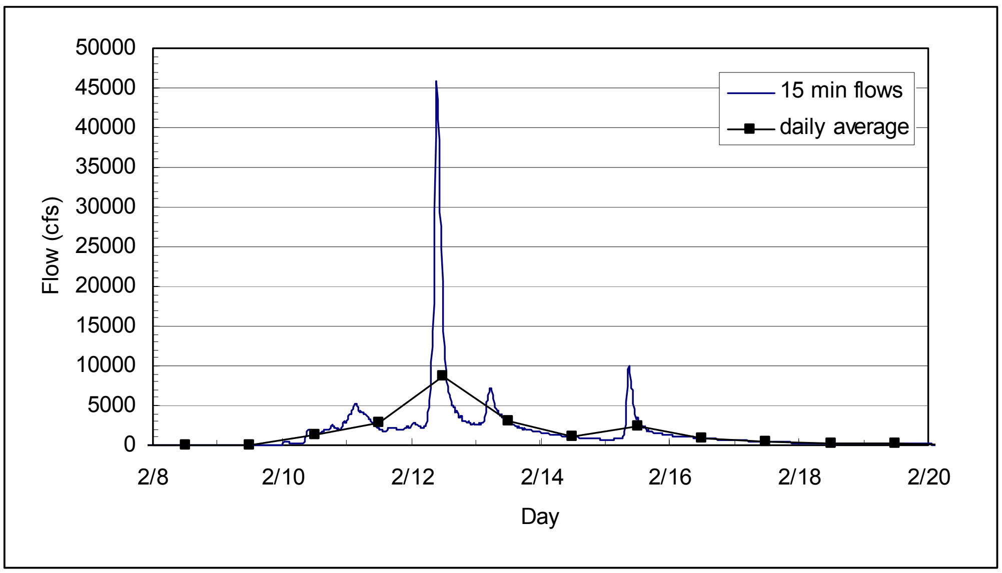


Figure 2.1. Comparison of 15-minute instantaneous hydrographs and daily average hydrographs for the 1992 flood at Foster Park gage on the Ventura River (USGS gage 11118500).

The Ventura River experiences large annual variations in peak flow magnitudes (Figure 2.2 and Figure 2.3). From the 1930s to the mid 1940s, the floods were relatively frequent. From mid 1940s until the late 1960s, the floods were less frequent and of smaller magnitude, except for the large flood of 1969. From the 1970s until the present, floods have occurred relatively frequently and several have been very large, with the largest flood of record occurring in 1978. It is difficult to extrapolate the variation in peak flow into the future and to predict if the present relatively wet period will continue or if we will enter into a relatively dry period.

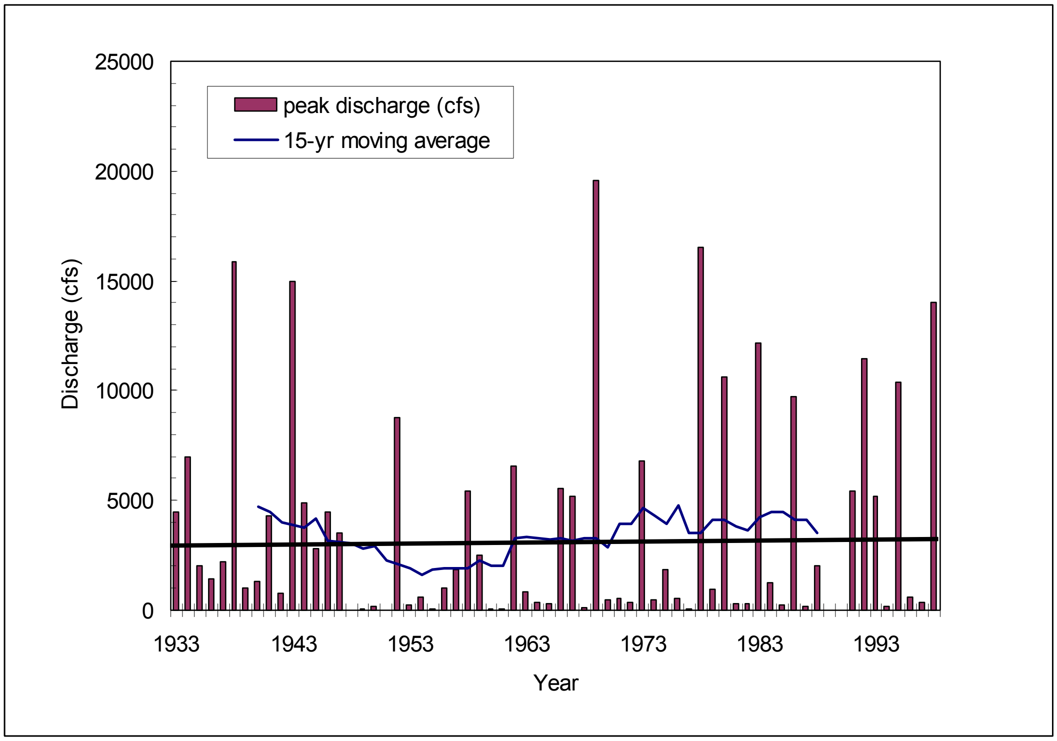


Figure 2.2. Peak Discharge at USGS gage 11115500, downstream of Matilija Dam on Matilija Creek. Flows between Oct 1 1988 and Sept 30 1990 were not available at this gage

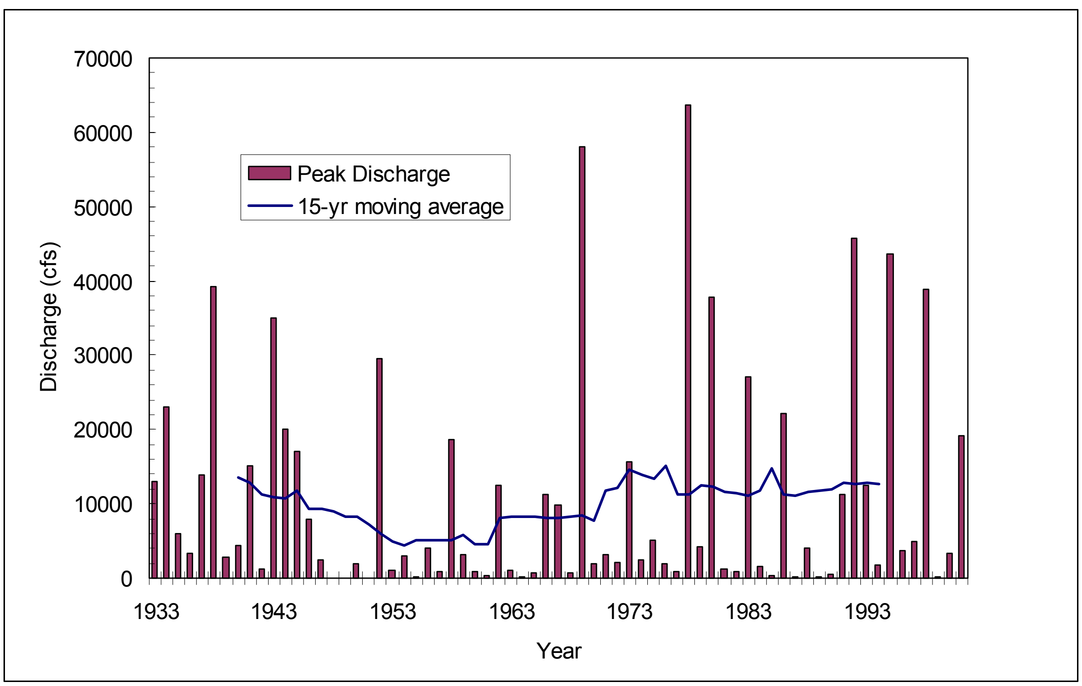


Figure 2.3. Peak Discharge at USGS gage 11118500, near Foster Park on the Ventura River.

2.2. Analysis of Average Daily Flows

The mean average daily flow for each day for the stream gage immediately downstream of Matilija Dam (USGS gage 11115500) and the gage on the Ventura River at Foster Park (USGS gage 11118500) is shown in Figure 2.4 and Figure 2.5, respectively. The annual volume of discharge is for the years 1928 to 2000 is shown for the same gages in Figure 2.6 and Figure 2.7. The most striking feature is the large variation from year to year. The hydrology is such that an average year is atypical. It is more likely that the annual discharge is much greater or much less than the average. A plot of mean average daily discharges is shown in Figure 2.8 and a plot of maximum average daily discharge Figure 2.9. Three types of hydrology are identified: 1948 would be considered an extremely dry year, 1991 would be considered an average year and 1969 would be considered an extremely wet year.

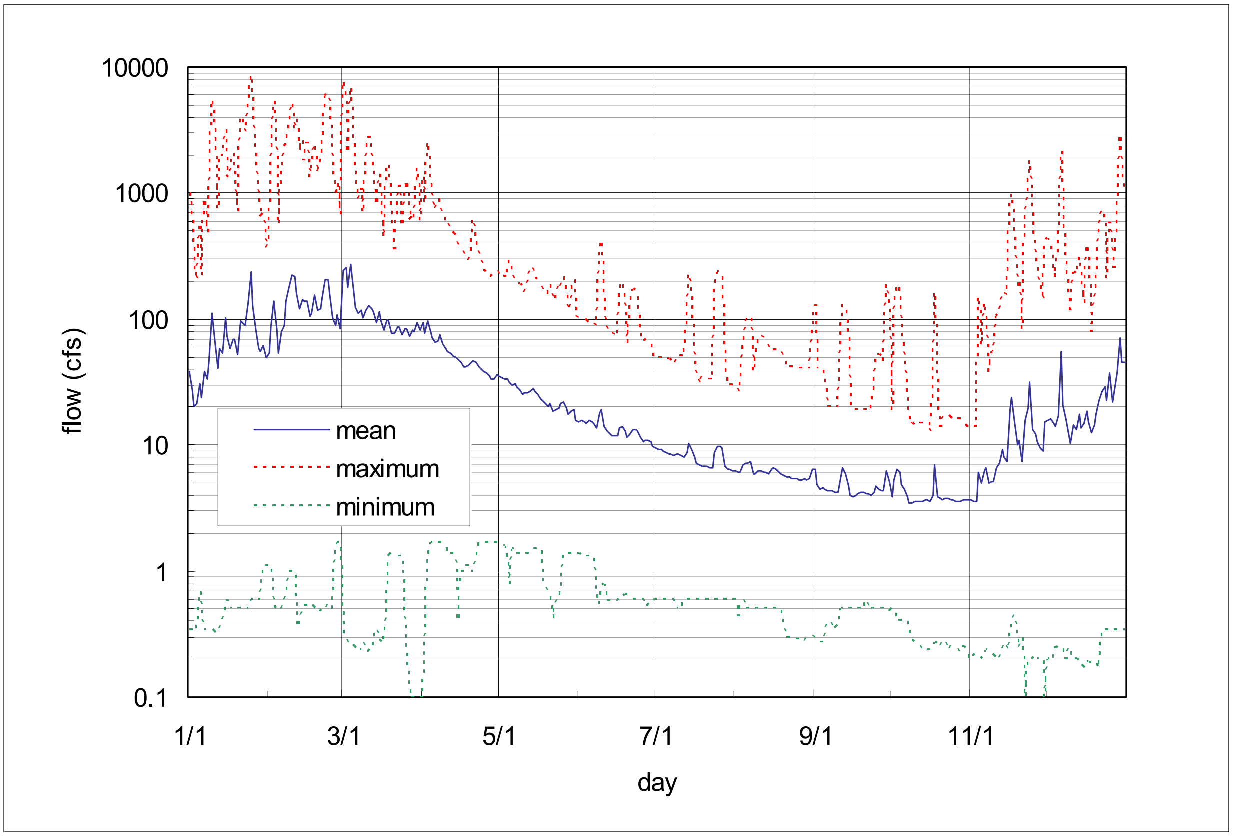


Figure 2.4. Mean, Maximum, and Minimum Daily Average Discharges for Every Day of the Year at USGS gage 11115500, downstream of Matilija Dam on Matilija Creek.

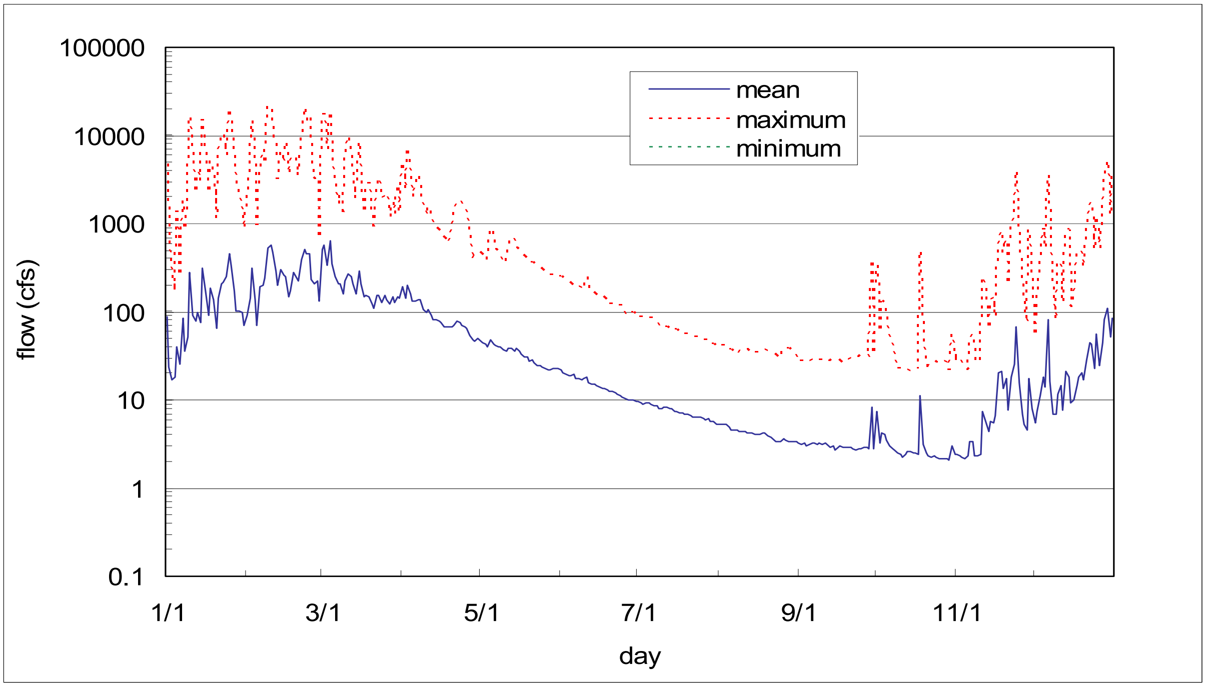


Figure 2.5. Mean, Maximum, and Minimum Daily Average Discharges for Every Day of the Year at USGS gage 11118500, Ventura River at Foster Park.

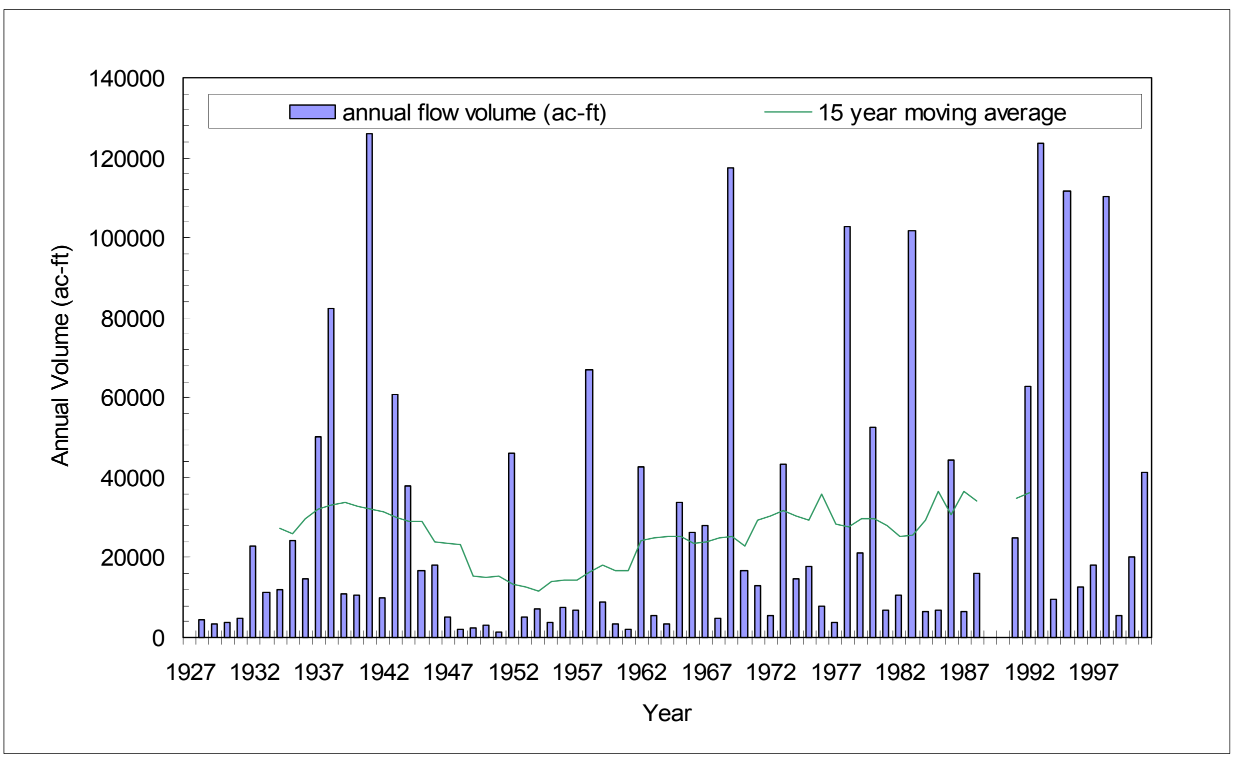


Figure 2.6. Annual flow volume at USGS gage 11115500, downstream of Matilija Dam on Matilija Creek.

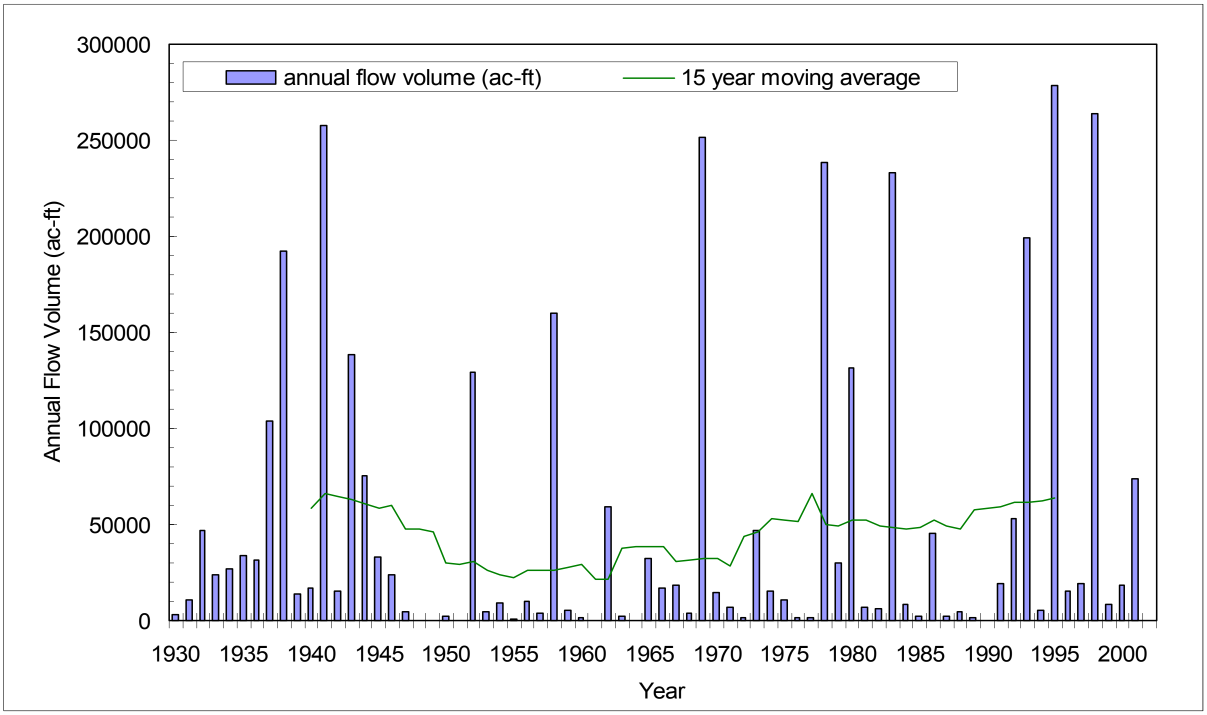


Figure 2.7. Annual flow volume at USGS gage 11118500, near Foster Park on the Ventura River.

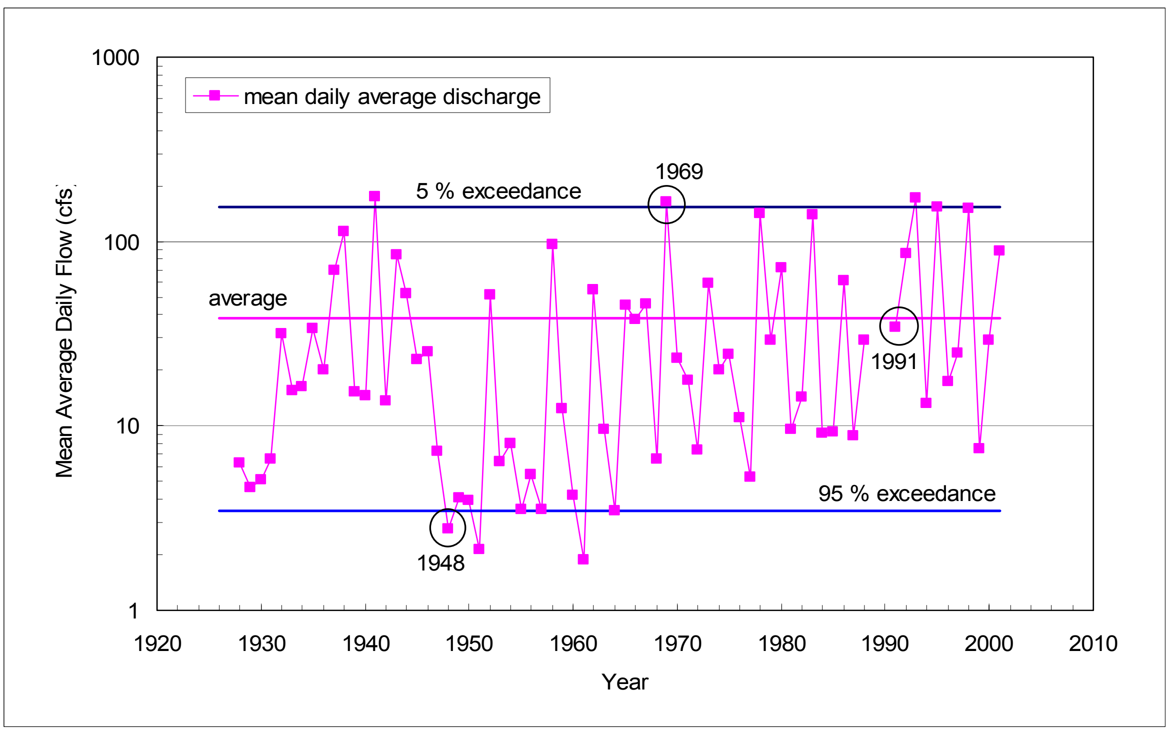


Figure 2.8. Analysis of mean daily average flows at USGS gage 11115500, downstream of Matilija Dam on Matilija Creek.

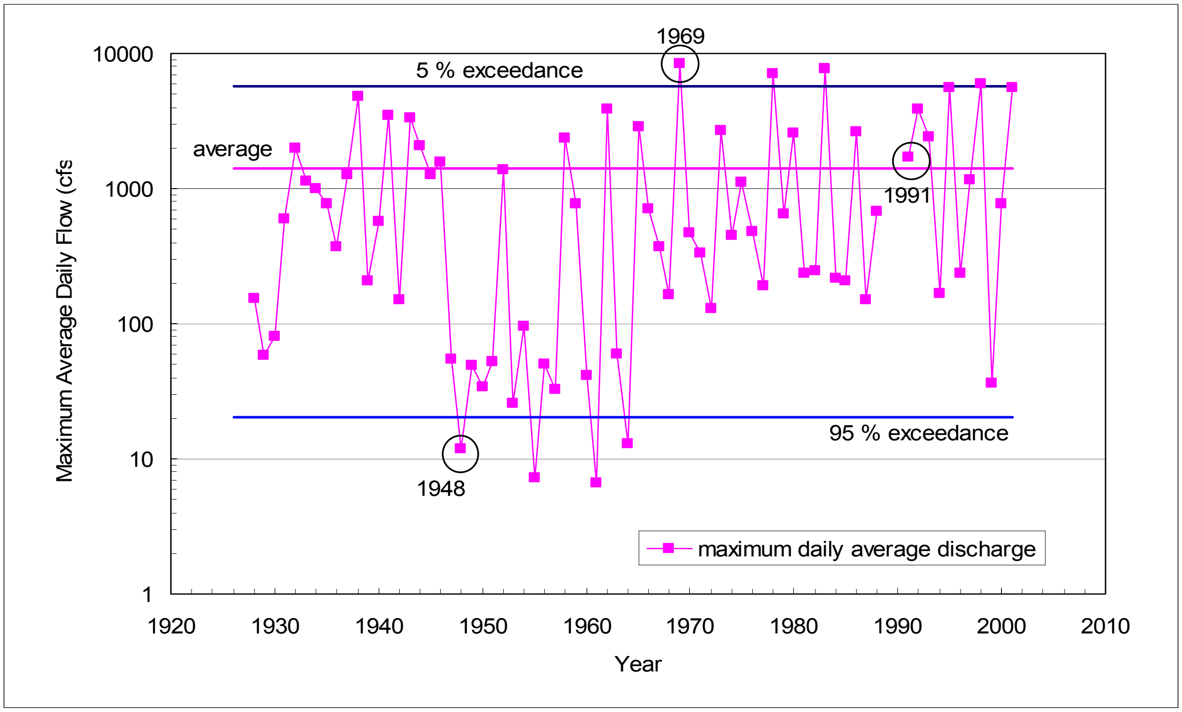


Figure 2.9. Analysis of maximum daily average flows at USGS gage 11115500, downstream of Matilija Dam on Matilija Creek.

2.2.1. PROBABILITIES DURING CONSTRUCTION PERIODS

Because construction will not take an entire year, it is necessary to develop exceedence probabilities for various periods of the year. Two different periods are analyzed: March through November, and December through February. The maximum daily averages for each of those periods from 1928 to 2001 are plotted in Figure 2.11 and Figure 2.12. The daily averages were used instead of the instantaneous peak flow because the peak flow record sometimes does not have measurements for the specified periods for every year. A comparison between the peak flow and the daily average flow is given in Figure 2.10. A power function was fit to the data to relate the maximum average daily flow to the instantaneous peak.

The results for the two periods are shown in Table 2.4 and Table 2.5. The probability of high flows is lower during the period March through November. However, the months of March, April and November can still have large flows and therefore there is a risk of having an extreme flow during these months. The period of December to February has the majority of peak flows and therefore the risks of peak flow are significant higher during this period.

Table 2.4. Exceedence Probabilities for Mar-Nov. Probability of exceeding an instantaneous peak of 500 cfs and an average daily flow of 1500 cfs are italicized.

Return Period (yr)	Exceedence probability (-)	Average daily flow (cfs)	Calculated Instantaneous Peak (cfs)
2	0.50	170	350
2.33	***0.43***	***235***	***500***
5	0.20	820	1850
8.15	***0.123***	***1500***	***3500***
10	0.10	1880	4440
10.6	***0.09***	***2000***	***4740***
25	0.04	4230	10430
50	0.02	6490	16370
100	0.01	8480	21690

Table 2.5. Exceedence Probabilities for Dec- Feb. Probability of exceeding an instantaneous peak of 500 cfs and an average daily flow of 1500 cfs are italicized.

Return Period (yr)	Exceedence probability (-)	Average daily flow (cfs)	Calculated Instantaneous Peak (cfs)
1.58	***0.63***	***237***	***500***
2	0.50	460	1010
2.33	0.43	630	1400
4.22	***0.24***	***1500***	***3500***
5	0.20	1800	4240
5.6	***0.18***	***2000***	***4740***
10	0.10	3160	7670
25	0.04	5210	12990
50	0.02	6850	17330
100	0.01	8490	21720

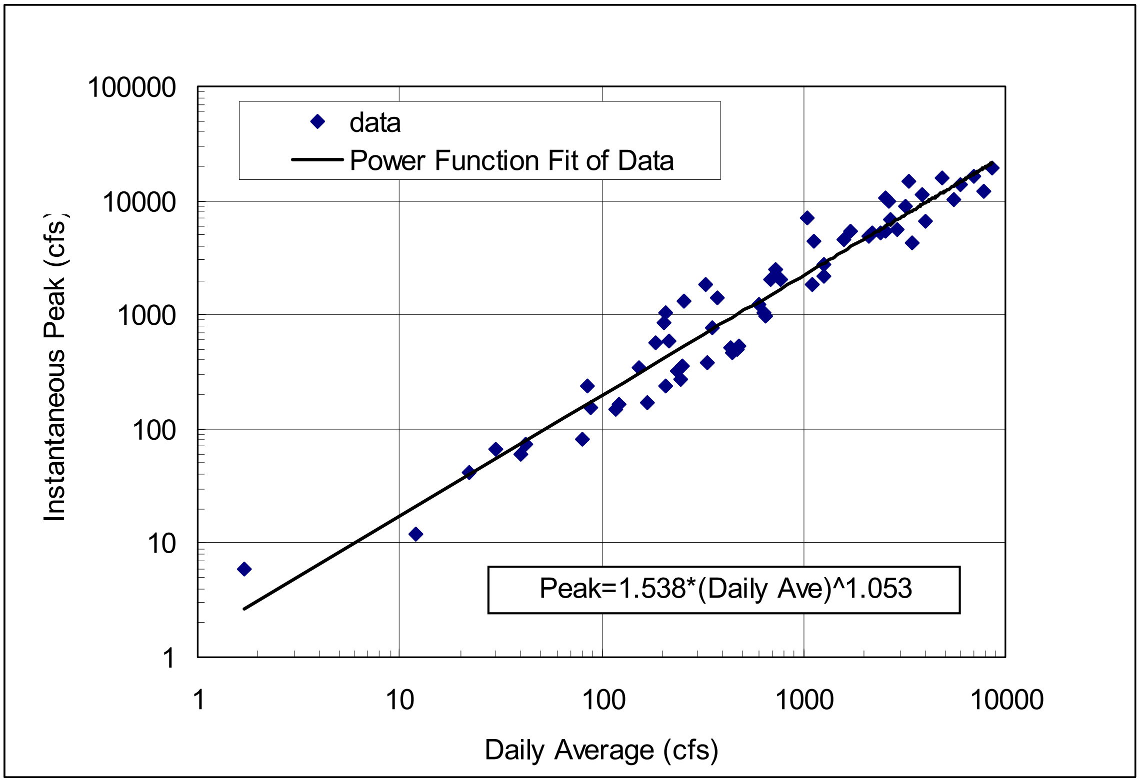


Figure 2.10. Comparison of Daily Average Flow and Instantaneous Peak for Combination of Stream Gage 11114500 and 11115500.

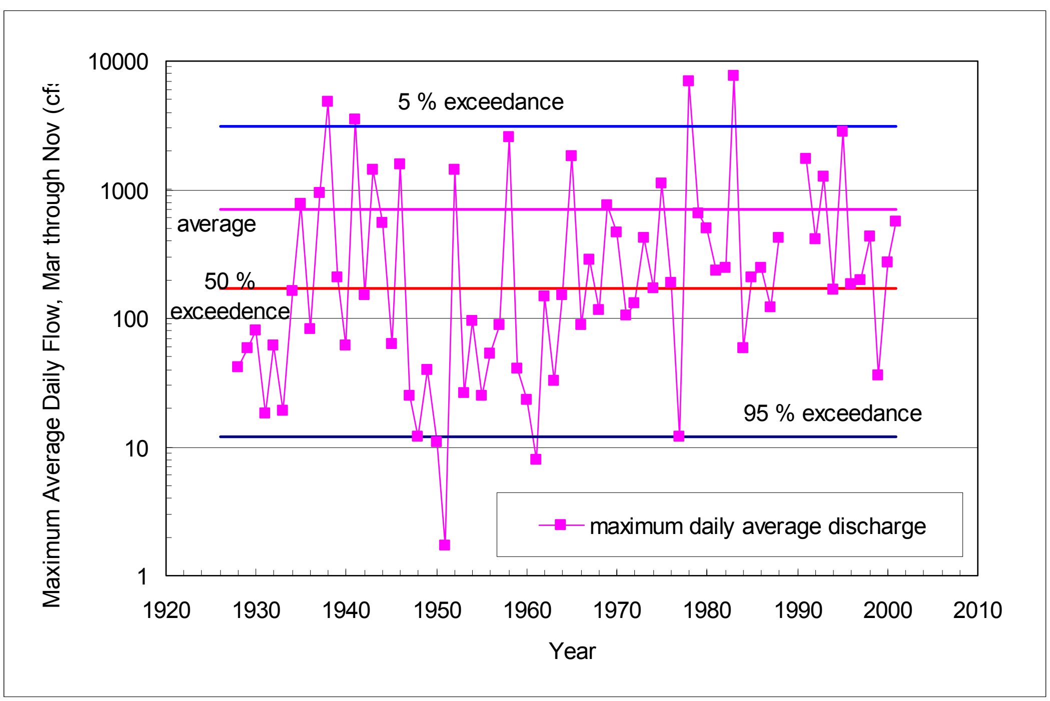


Figure 2.11. Maximum Daily Average Flows for the period March through November.

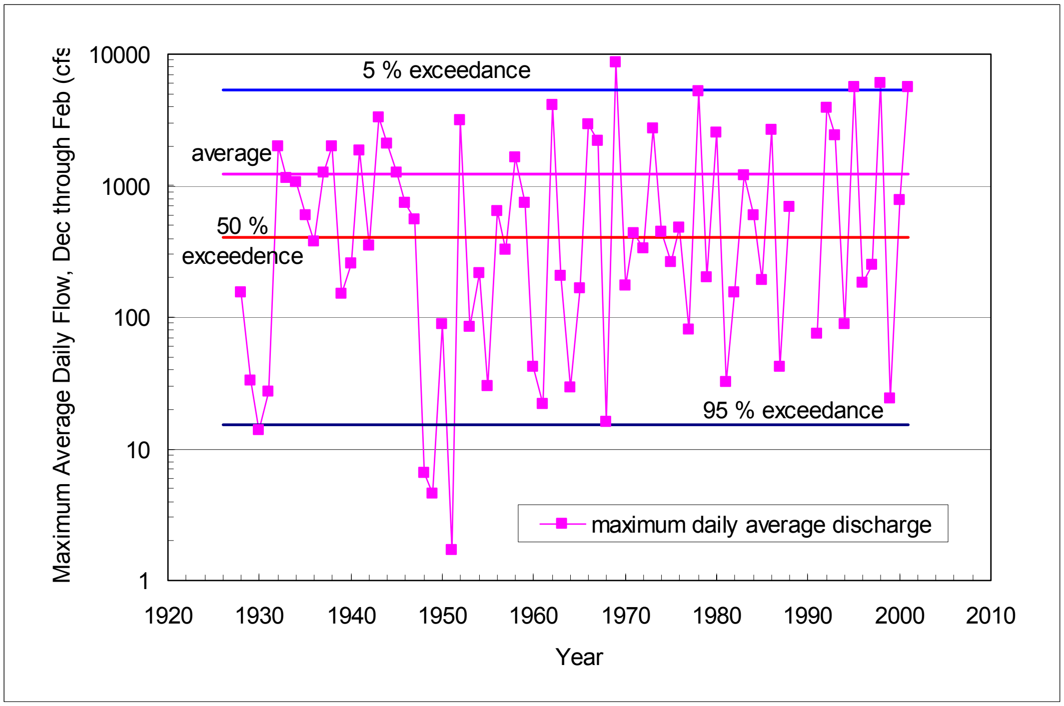


Figure 2.12. Maximum Daily Average Flows for the period December through February.

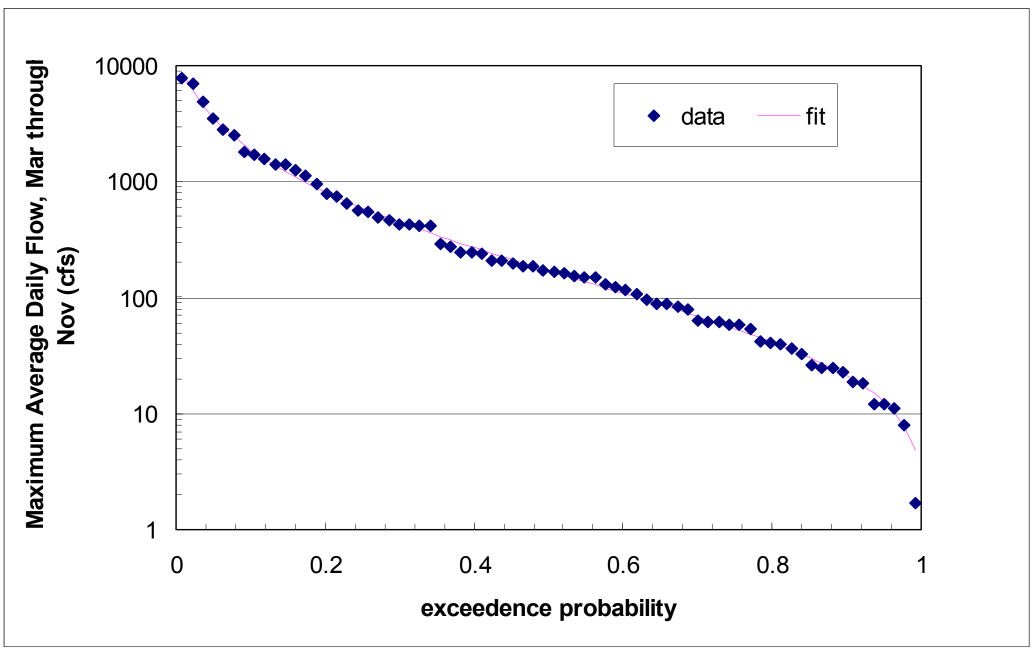


Figure 2.13. Exceedence Probabilities of Maximum Daily Average Flows for the period March through November.

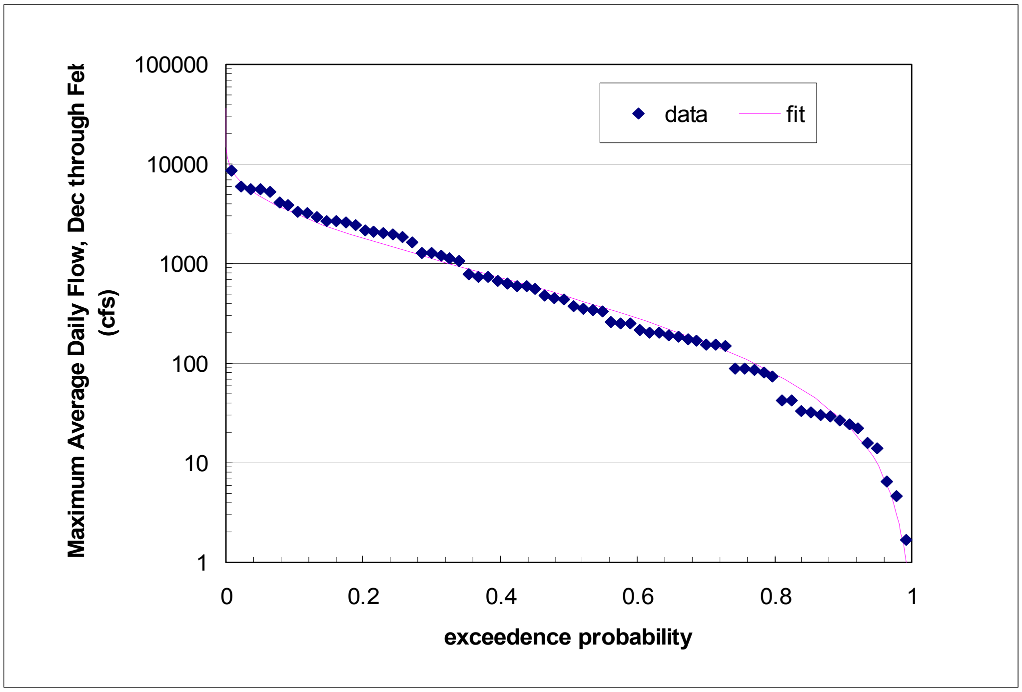


Figure 2.14. Exceedence Probabilities of Maximum Daily Average Flows for the period December through February.

2.2.2. FLOW DURATION CURVES

Flow duration curves were developed for the stream gages shown in Table 2.6 and Table 2.7. Over 60% of the time, the flow is less than 10 ft^3/s in the Ventura River at Foster Park, and approximately 80% of the time the flow is less than 10 ft^3/s in the Ventura River at Meiners Oaks. The river has no flow at least 30% of the time at Meiners Oaks. Flood duration is very short and large flows occur infrequently. For example, the 2 – yr flood value is only exceeded 0.2% of the time in the Ventura River. Exhibit A contains flow duration curves for each month.

Table 2.6. Values of Flow Duration Curves at Stream Gages.

Location	Matilija Creek ab Reservoir at Matilija Hot Springs	Matilija Creek At Matilija Hot Springs	North Fork Matilija Creek at Matilija Hot Springs	Ventura River near Ojai, California	Ventura River nr Meiners Oaks, CA
Gage Number	11114500	11115500	11116000	11116500	11116550
Begin Year	1949	1933	1933	1922	1959
End Year	1969	1988	1983	1924	1988
Number of Years	21	56	51	3	30
Drainage Area (mi^2)	15.6	54.7	15.6	70.7	76.4
Gauge Datum (ft)	1160.2	900.0	1142.02	NA	NA
% of time below	**Flow (ft^3/s)**				
0	0.3	0.1	0.10	1.0	0.0
10	1.0	1.3	0.50	3.5	0.0
20	1.6	2.3	0.85	4.5	0.0
30	2.2	3.2	1.2	5.5	0.0
40	3.3	4.2	1.5	9	0.2
50	4.5	5.5	2.2	12	1.5
60	7.2	7.5	3.0	14	3.7
70	9.5	11	4.1	20	6.9
80	14	19	6.5	34	10
90	35	53	15	58	15
91	39	60	17	63	16
92	46	70	19	69	17
93	53	83	23	75	19
94	63	103	27	87	21
95	78	128	34	100	25
96	96	163	43	124	30
97	130	210	57	145	48
98	212	276	84	181	158
99	386	470	156	252	298
99.5	738	775	275	373	585
99.7	1070	1070	378	452	919
99.9	2890	2120	830	755	5120
99.95	4050	3480	1390	764	7140
99.99	6210	6840	2810	837	10600
100	8610	8340	4980	910	13300

Table 2.7. Values of Flow Duration Curves at Stream Gages (continued).

Location	San Antonio Creek at Casitas Springs	Coyote Creek near Oak View CA	Santa Ana Creek Near Oak View	Coyote Creek Near Ventura, CA *	Ventura River near Ventura*
Gage Number	11117500	11117600	11117800	11118000	11118500
Begin Year	1950	1959	1959	1927	1930
End Year	1983	1988	1988	1982	2000
Number of Years	34	30	30	56	71
Drainage Area (mi^2)	51.2	13.2	9.11	41.20(2.00)	188.0
Gauge Datum (ft)	307.55	577.37	612.43	224.95	205.23
% of time below	**Daily Average Flow (ft^3/s)**				
0	0.0	0.0	0.0	0.00	0.00
10	0.0	0.11	0.0	0.00	0.0
20	0.0	0.24	0.0	0.00	0.0
30	0.0	0.4	0.0	0.03	0.3
40	0.4	0.56	0.1	0.06	1.2
50	0.9	0.8	0.2	0.09	3.0
60	2.0	1	0.5	0.14	6.2
70	3.6	1.5	0.9	0.23	11
80	5.7	2.6	2.0	0.37	22
90	15	6.9	6.5	0.68	63
91	17	7.8	7.4	0.78	73
92	20	9.2	8.7	1.0	88
93	24	11	10	1.2	109
94	28	14	12	1.8	140
95	36	18	15	2.5	189
96	49	23	20	5.3	275
97	70	32	27	12	410
98	102	50	43	30	609
99	218	127	96	68	1180
99.5	421	240	193	167	2100
99.7	746	417	333	232	3300
99.9	1880	950	753	318	7130
99.95	2920	1825	1010	430	10400
99.99	4300	2500	1730	575	20000
100	10400	2980	1900	612	22000

*Flow Duration Curve for Coyote Creek Near Ventura, CA, USGS No. 11118000 and for Ventura River near Ventura, CA USGS No. 11118500 are both from 1959 to the present after the construction of Casitas Dam.

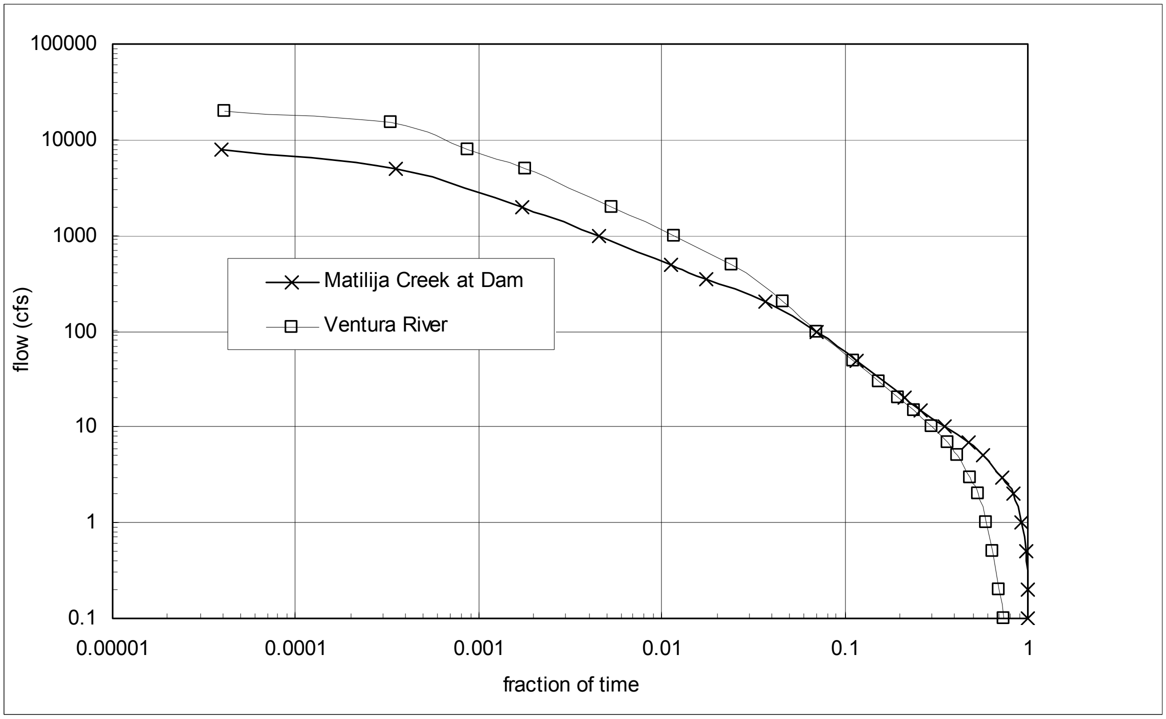


Figure 2.15. Plot of flow duration curves for USGS gage 11115500 at Matilija Dam and USGS gage 11118500, near Foster Park on the Ventura River.

2.3. Flow Diversion at Robles

Some additional analysis of daily stream flow records and diversion flow records made available by the Casitas Municipal Water District (CMWD) have been conducted to understand better the operation of the Robles Diversion Works. The actual diversion record from 1991 to 2001 was used to analyze their diversions. On the average, diversions at Robles diversion have occurred between January 1 and April 15 each year (Figure 2.16). The largest diversions will occur in the months of January and February in average years. This corresponds directly with the largest inflows that historically occur at Matilija Dam. In extremely wet years, the diversions occur as early as November 4 and as late as August 7. Historically, diversions are as high as 540 ft^3/s, but are usually limited to less than 500 ft^3/s.

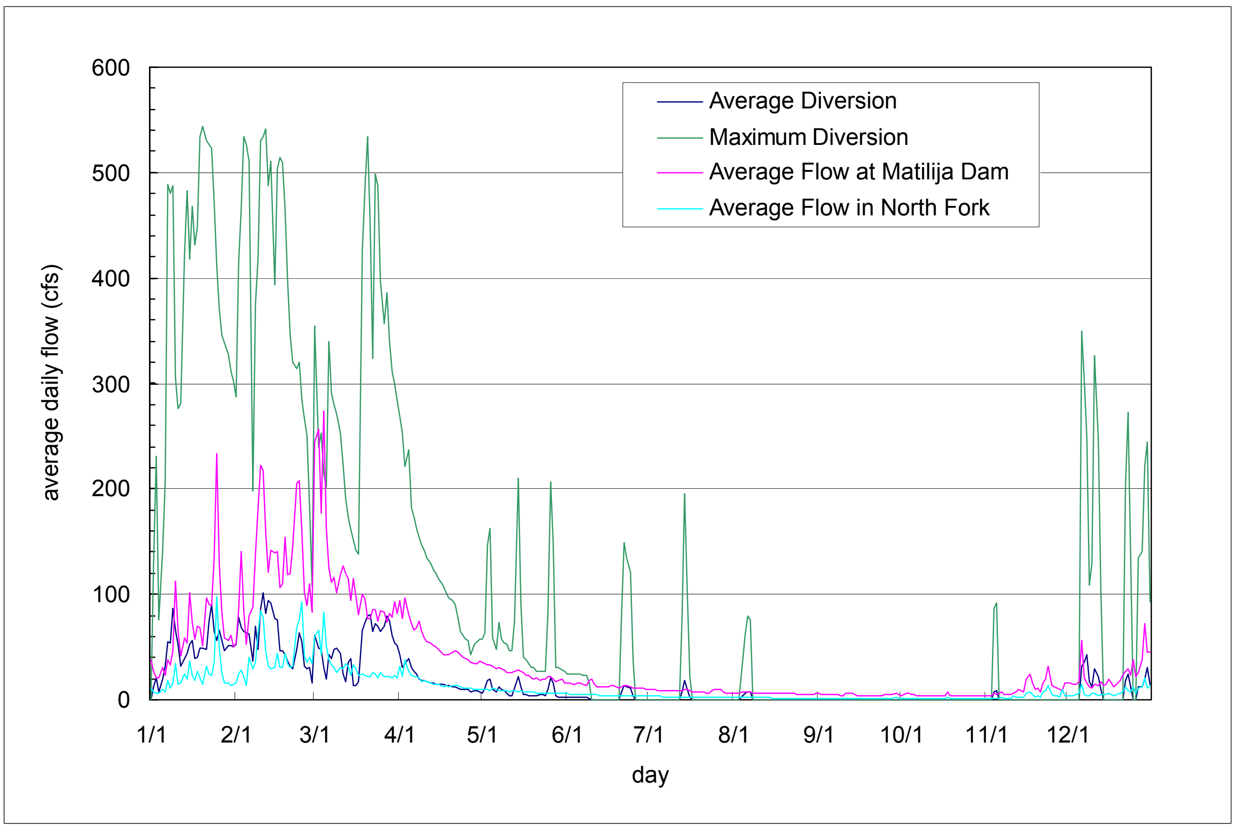


Figure 2.16. Average flow in Matilija Creek, North Fork Matilija Creek, and Robles Diversion.

The annual volume of water diverted at Robles diversion was computed from daily discharge records for the years 1991 to 2000 supplied by the Casitas Municipal Water District. The average annual volume of water diverted is 13,000 ac-ft for these 10 years. The average annual flow for Matilija Creek at Matilija Dam is 49,900 ac-ft for the same 10-year period. Additional flow from North Fork Matilija Creek occurs between Matilija Dam and the Robles diversion structure. This corresponds to the historical average diversion as stated by Casitas of 12,500 acre-ft (Leo Lentsch, person communication).

Based on the available diversion flow data for the years 1991 to 2000 there appears to be no set date for the diversion to be shut down each year. Diversions will occur as late as August 7 or July 16 in wet years such as 1992 and 1993. In normal years, the diversion appears to be shut down by the end of May. It appears that diversions can occur during all summer months if water is available for diversion.

The benefit of Matilija Dam with a current storage capacity of 500 ac-ft was analyzed using available average daily inflows to Matilija Dam and some assumed operation rules for storage and release of water from Matilija Dam for the diversion. These assumed rules are designed to get the most benefit from storage at Matilija Reservoir for later diversion at Robles. These assumed operation rules do not necessarily reflect current operation procedures for Matilija Reservoir. These assumed operation rules are described below:

1. If Matilija Dam has less than 500 ac-ft water in storage, additional storage will be added from Matilija Creek daily inflows if those average daily inflows are greater than 500 ft^3/s. The difference between the average daily inflow and 500 ft^3/s will be stored until 500 ac-ft of storage is obtained. The remaining 500 ft^3/s each day will be pass downstream and diverted at Robles diversion.

2. If Matilija Dam has 500 ac-ft of water in storage, and average daily inflows are greater than 500 ft^3/s, then all inflows will be passed downstream without attenuation.

3. If the average daily inflows to Matilija Dam are less than 500 ft^3/s, and Matilija Reservoir is empty no additional storage is allowed. The average daily inflows are assumed to pass downstream, and Matilija Dam provides no beneficial storage.

4. If average daily flows below Matilija Dam are less than 500 ft^3/s, and Matilija Reservoir has available water then additional flows from Matilija Dam storage will be released until the average daily downstream flows are 500 ft^3/s or until the storage in Matilija Dam is emptied.

The accumulation of the flows released from Matilija Dam to provide for 500 ft^3/s total average daily flow below Matilija Dam provides the beneficial amount of additional volume available for diversion due to the presence of Matilija Dam. This analysis also assumes that all 500 ft^3/s below Matilija Dam will then be diverted at Robles diversion, and that any additional flow from the ungaged tributaries will be sufficient to satisfy any minimal flow requirements downstream from Robles diversion. This analysis also ignores the finite capacity of Casitas Reservoir. During wet years, the actual benefit of Matilija Dam is unimportant if Casitas Reservoir is full. Because diversion is ceased once Casitas is full, the benefit of Matilija Dam is only realized during years when Casitas is empty. During the 1990's, for example, Casitas Reservoir was full and the much of the water available for diversion was passed downstream. Therefore, the benefit of Matilija Dam reported here, should be considered the **maximum potential** benefit, not the realized benefit.

To assess the benefit, the flow used in the analysis should be the unimpaired flow in Matilija Creek. From 1928 to 1947, it is possible to use USGS gage 11115500, located below Matilija Dam, because it was before the construction of Matilija Dam. From 1947 to 1969, the stream flow record of the USGS gage 11114500, located upstream of Matilija Dam could be used. From 1969 to 2001, the CMWD estimated the unimpaired flows using USGS gage 11115500, located downstream of Matilija Dam on Matilija Creek and USGS gage 11116000 located on North Fork Matilija Creek.

Table 2.8. Flow records used to assess the benefit of Matilija Dam.

Period	Gages Used	Comments
1927 - 1947	11115500	Before construction of Matilija Dam
1947 - 1969	11114500	Gage located upstream of Matilija Dam
1969 - 2001	11115500, 11116000	Estimated unimpaired flows in Matilija Creek

The flows records listed in Table 2.8 are used in conjunction with the operational rules listed above to asses the average benefit of Matilija Dam. The result of this operation study shows that on average about 590 ac-ft/yr of additional water could be made available for Robles diversions by operation of Matilija Dam in accordance with the assumed operation rules. The average of 590 ac-ft/yr represents 4.5 percent of the average annual Robles diversion volume of 13,000 ac-ft. Figure 2.17 graphically displays the Matilija Creek flows at Matilija Dam. The total annual diversions and the beneficial releases from Matilija Dam that could be diverted at Robles diversion for the years 1991 to 2000 based on these operation rules and assumptions.

In 1968, Reclamation estimated that releases from Matilija Dam would contribute about 1,900 ac-ft per year to the safe annual yield of Lake Casitas. At that time, the capacity of Matilija Dam was 3850 ac-ft/yr. In 1989, Murray, Burns, and Kienlen (Casitas, 1989) reduced this number to 420 ac-ft/yr. The reduction was mainly due to the sedimentation in Matilija Dam, which had decreased its capacity to approximately 1000 ac-ft. This estimate is based on the benefit of Matilija Dam during the dry years that comprise the period on which the safe yield is based.

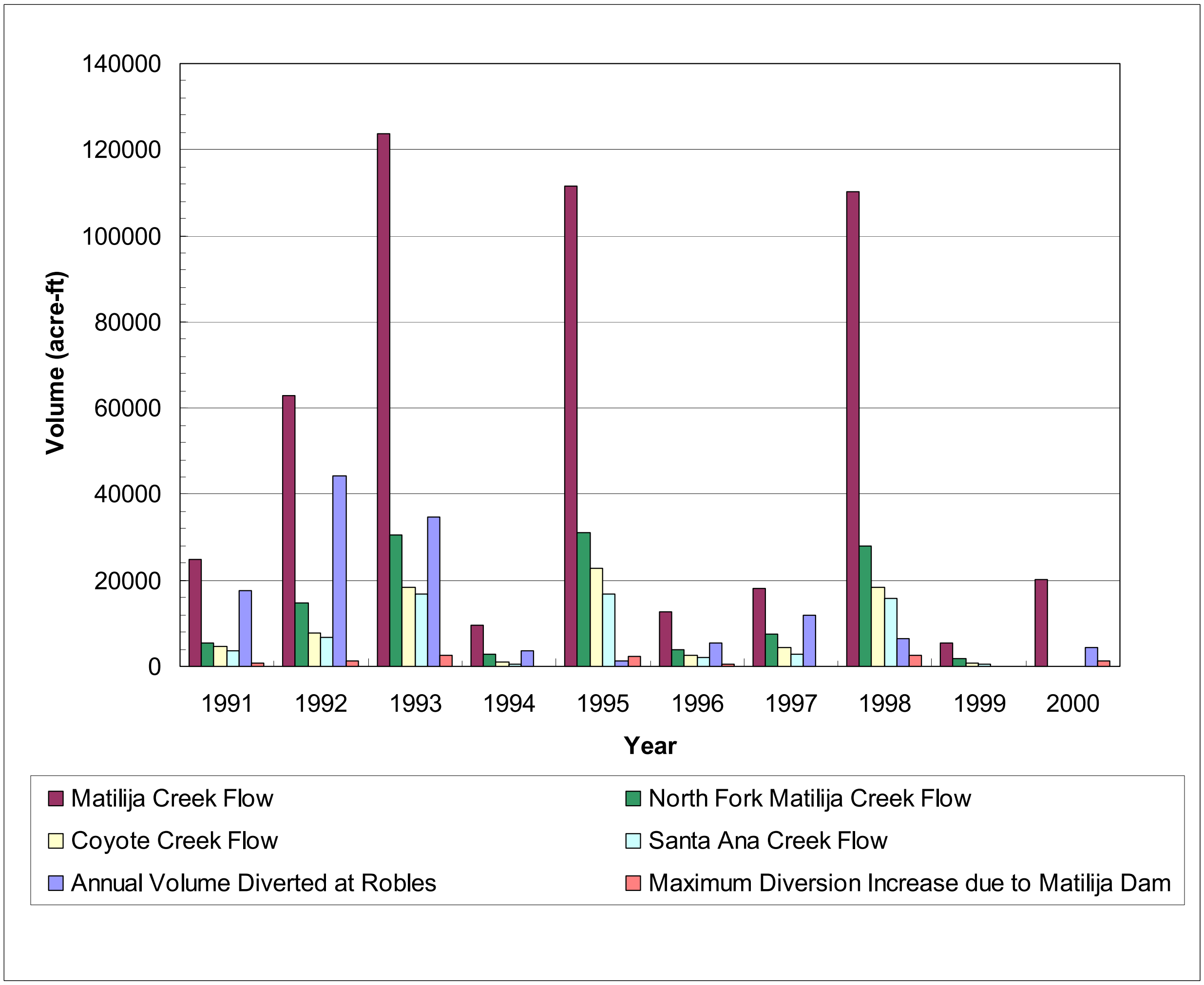
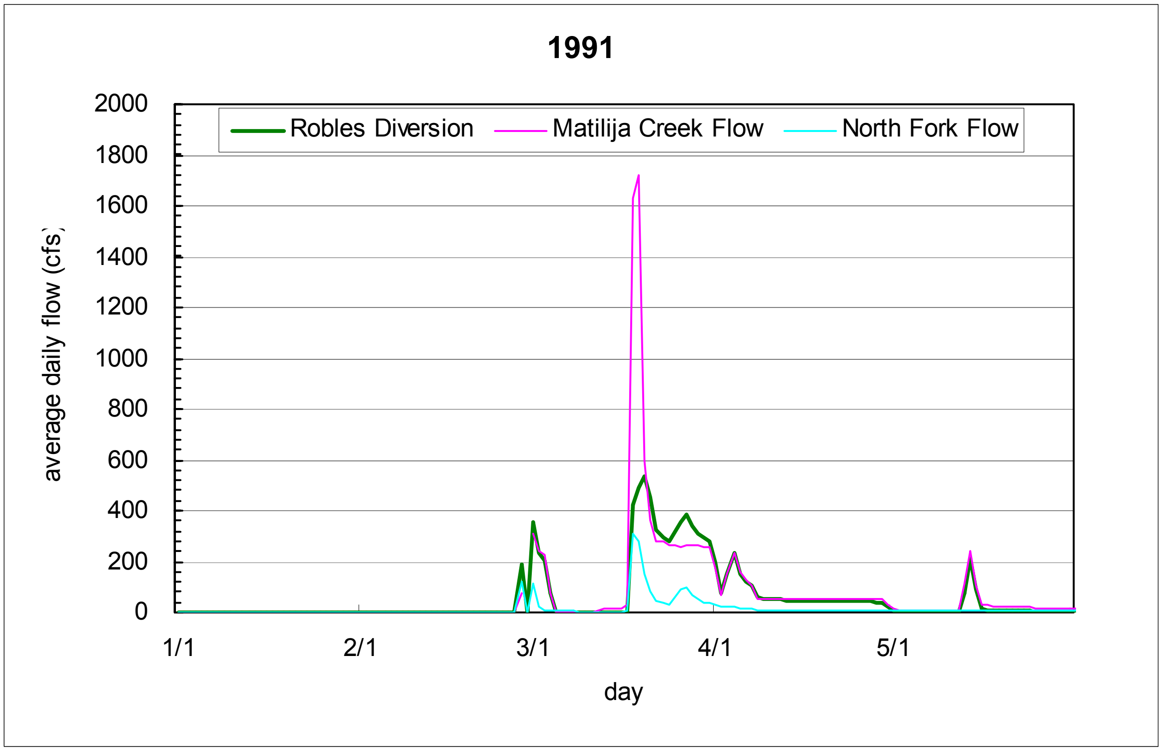
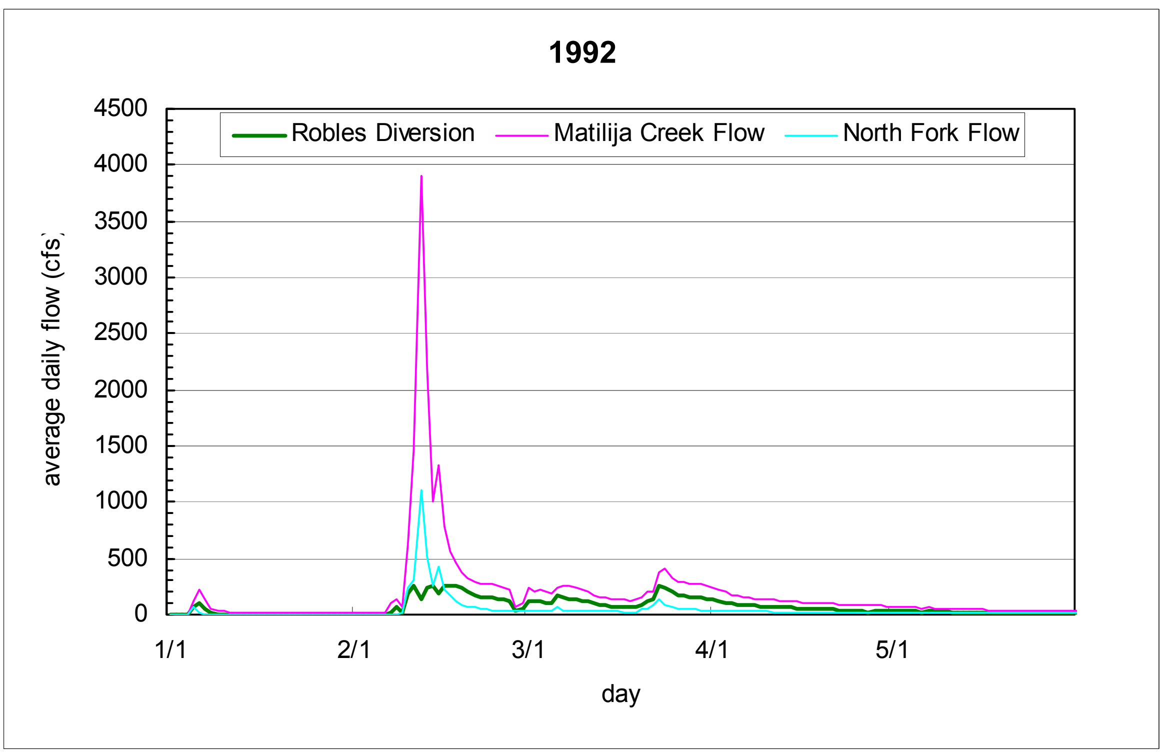
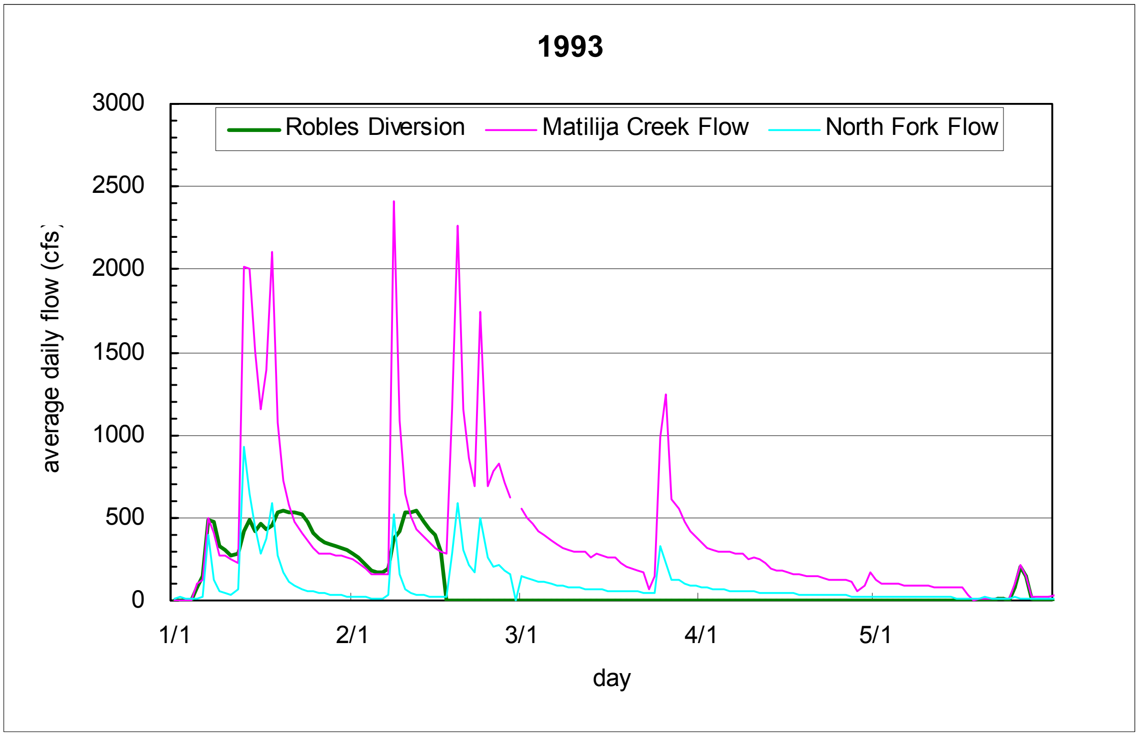


Figure 2.17. Annual Flow and Diversion Volumes for Period 1991 to 2000.

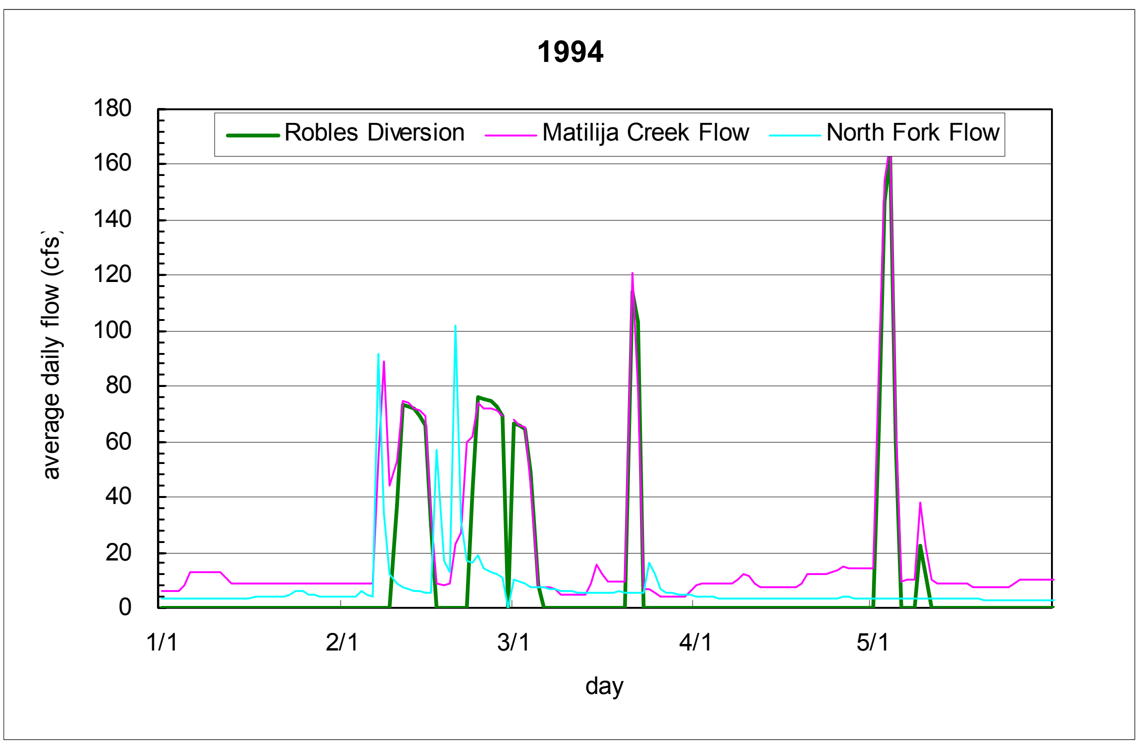
The actual daily average flow diversions for the period of 1991 to 1998 are shown on the following pages. It should be noted that there are large variations in the flow for a given year and as a result, there are large variations in the diversion at Robles. For example, in 1997, 47% of the flow of North Fork and Matilija Creek was diverted into Robles Canal.


1991
Robles Diversion
Matilija Creek Flow
North Fork Flow
average daily flow (cfs)
2000
1800
1600
1400
1200
1000
800
600
400
200
0
1/1
2/1
3/1
4/1
5/1
day

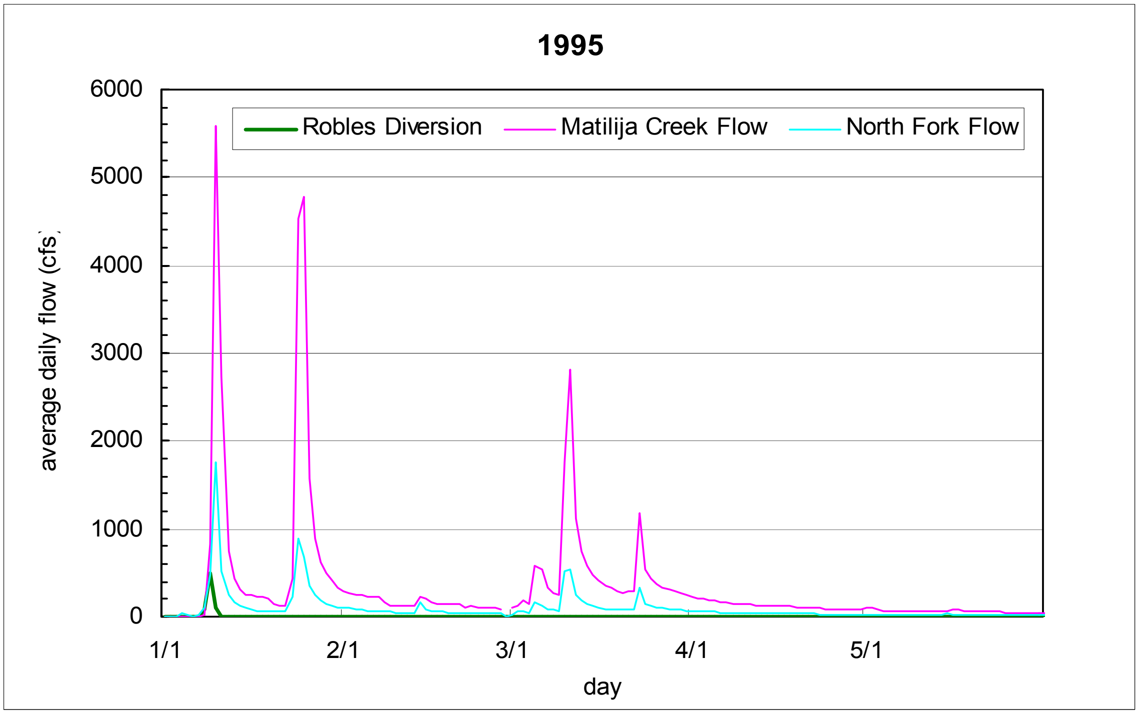

1992
Robles Diversion
Matilija Creek Flow
North Fork Flow
average daily flow (cfs)
4500
4000
3500
3000
2500
2000
1500
1000
500
0
1/1
2/1
3/1
4/1
5/1
day

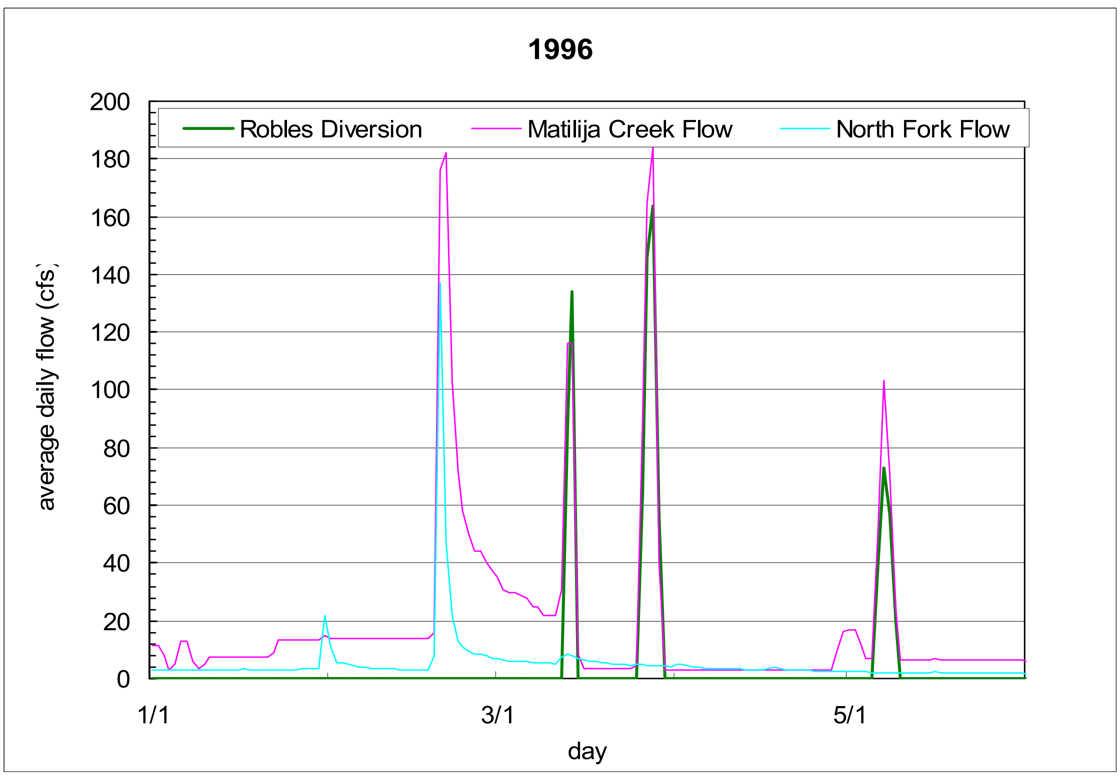


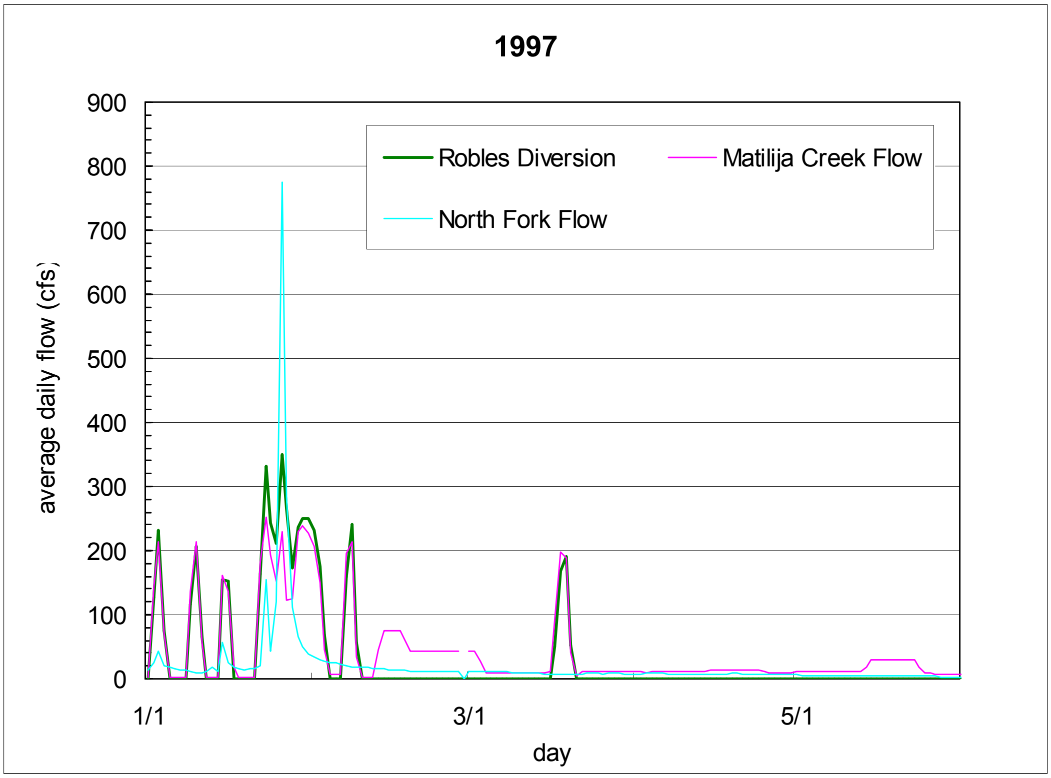
1993
3000
2500
2000
1500
1000
500
0
Robles Diversion
Matilija Creek Flow
North Fork Flow
average daily flow (cfs)
1/1
2/1
3/1
4/1
5/1
day



1994
180
160
140
120
100
80
60
40
20
0
Robles Diversion
Matilija Creek Flow
North Fork Flow
average daily flow (cfs)
1/1
2/1
3/1
4/1
5/1
day


1995
Robles Diversion
Matilija Creek Flow
North Fork Flow
average daily flow (cfs)
6000
5000
4000
3000
2000
1000
0
1/1
2/1
3/1
4/1
5/1
day


1996
Robles Diversion
Matilija Creek Flow
North Fork Flow
average daily flow (cfs)
200
180
160
140
120
100
80
60
40
20
0
1/1
3/1
5/1
day



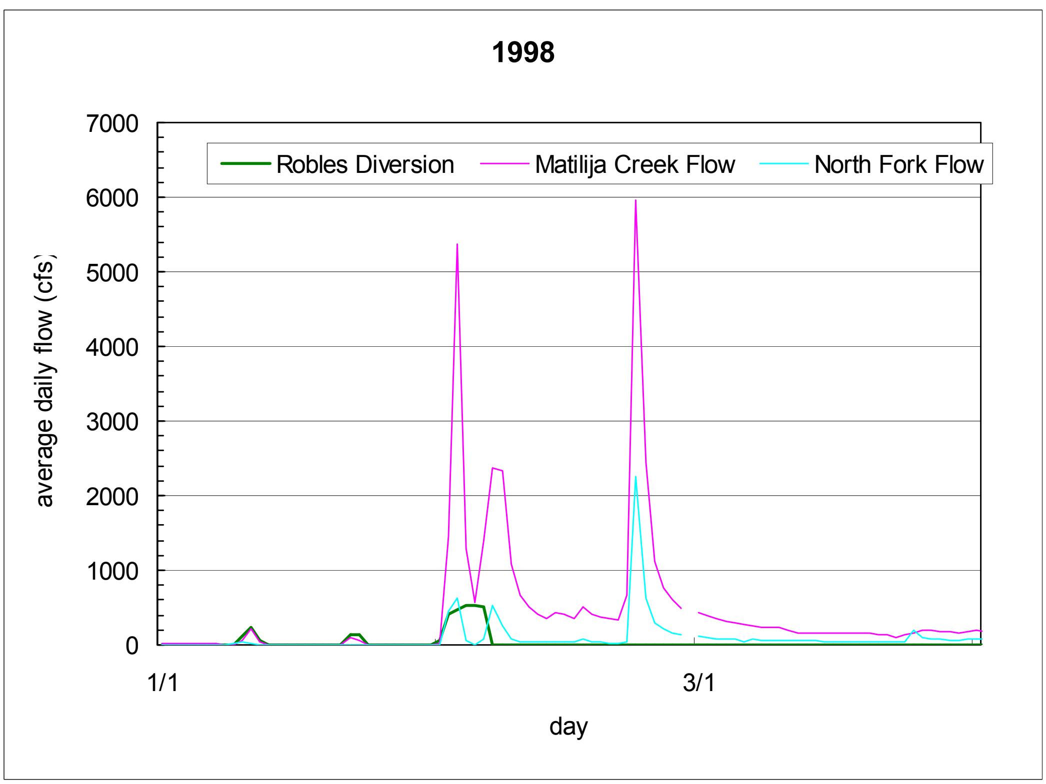


Figure 2.18. Daily average flows at Matilija Creek, North Fork Matilija Creek, and Robles Diversion for the period 1991 to 1998.

2.4. Future Without-Project Conditions Hydrology

Matilija Dam will continue to fill with sediment and the effective storage of the dam will be 230 ac-ft in approximately 10 years and less than 50 ac-ft in 20 years (Table 5.23). This assumes that the current trap efficiency is 45% and the trap efficiency decreases with storage capacity.

As mentioned previously, Matilija Dam does not affect the peak flows and therefore additional sedimentation in Matilija Dam will not affect the peak flows. After the present 500 ac-ft reservoir is gone, however, the current benefit of Matilija Dam to the diversion capacity at Robles will be unavailable. The projection of the cumulative benefit, starting in 2003, of Matilija Dam is shown in Figure 2.19. To generate this graph, it was assumed that the benefit in 2003 was 590 ac-ft/yr and the benefit was assumed to decrease linearly with storage capacity of Matilija Reservoir. The storage capacity was taken from Table 5.23. Based on this analysis, the total benefit of Matilija Dam under the Without-Project Conditions is approximately 5000 ac-ft from 2003 until the reservoir capacity is completely gone, which occurs effectively in 2020.

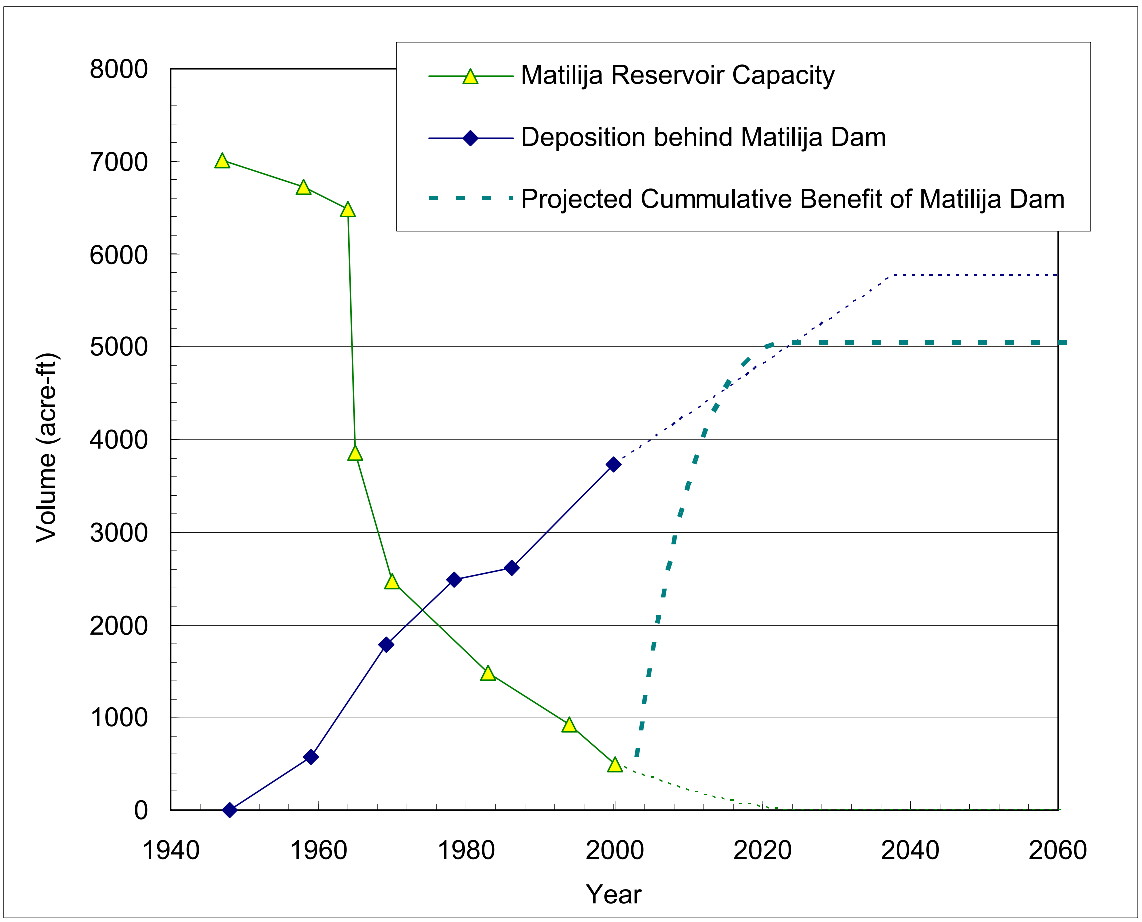


Figure 2.19. Storage Capacity of Matilija Reservoir and Projected Benefit of Matilija Dam to the Amount of Water Diverted at Robles.

Another important factor is that there is some evaporation loss due to the open pool of water of Matilija Reservoir. Matilija Reservoir is approximately 25 acres. Based upon measurements of pan evaporation in the Santa Clara River Basin the evaporation potential is more than 60 inches per year (United Water Conservation District, 2001). Entrix (2002) has also computed the annual evaporation for Lake Casitas. Since 1970, the average evaporation has been average 3.5 ac-ft/ac/yr over the area of the lake (2,700 acres). The average direct precipitation on Lake Casitas was 1.9 ac-ft/ac/yr. The net loss is therefore at least 1.6 ac-ft/ac/yr. Assuming the same rate for Matilija Reservoir, there is 40 ac-ft/yr of water lost due to evaporation from the reservoir. The water loss could be considered more than this because some of the rain falling on the reservoir would enter Matilija Creek regardless if it falls on dry ground or on the reservoir.

There is also water loss through transpiration due to the presence of vegetation on the delta. The delta area is approximately 50 additional acres and is highly vegetated with Arundo and Cottonwood trees. It is estimated the Arundo has a water use of approximately 5.6 ac-ft/ac/yr (Iverson, 1993). Native species were found to have a water use of 1.9 ac-ft/ac/yr in the Santa Ana Basin (Iverson, 1993). Therefore, the water loss due to the Arundo on the delta could be as much as 3.7 ac-ft/ac/yr of water over the area of the delta or a total of 185 ac-ft/yr. Additional work should be done to estimate the vegetation types on the delta. Until further work on the vegetation is done, the transpiration value of 185 ac-ft/yr should be considered an upper estimate.

The total additional evapo-transpiration due to the presence of Matilija Dam could be as large as 225 ac-ft/yr. This water loss could continue even as the reservoir pool disappears because of the high water use of the vegetation on the delta sediments. Over the next 50 years, Matilija Dam would cause up to 11,500 ac-ft of water loss due to evapo-transpiration.

3. Without-Project Groundwater Hydrology

Previous studies of the groundwater hydrology in the Ventura Basin have been conducted by Turner (1971). Reclamation (1981) performed evaluations of various alternatives for water resources development in the Ventura Basin. A map of the Ventura County groundwater basins is given in Figure 3.1.

The Upper Ventura River (upstream of San Antonio Creek) is underlain by alluvial deposits with a maximum thickness of 200 feet with an average thickness of 60 to 100 feet. Just upstream of San Antonio Creek, a groundwater constriction forces water to the surface and causes surface flow below this point (Figure 3.2). Therefore, the groundwater beneath the Ventura River is divided into an upper cell and the lower cell. The water quality in the Upper Ventura River Groundwater is generally good, with total dissolved solids concentrations ranging from 400 to 1000 parts per million (ppm). The groundwater stored in the Lower Ventura River Basin is considered unsuitable for municipal use (Turner, 1971). It is unclear if the degradation of the water quality in the Lower Ventura is due to the oil field operation or natural percolation of contaminated waters from adjacent and underlying marine formations.

Turner estimated that the ground water storage in the Upper Ventura River in the spring of 1970 was 20,410 ac-ft. This is considered approximately full capacity. From 1947 to 1973, Turner states that groundwater use in the Upper Ventura River ranged from 1,458 to 6,268 ac-ft/yr and that production was over 4,000 ac-ft from 1963 to 1973.

Entrix (2001) has prepared a report analyzing the surface-groundwater interactions. In this report, they identify several groundwater users. Meiners Oaks County Water District (MOCWD) operates 2 wells located approximately 1 mile downstream of Matilija Dam and 2 wells near Meiners Oaks adjacent to Rice Road. The MOCWD produces approximately 1,300 ac-ft/yr of water from these wells (Entrix, 2001). Ventura River County Water District (VRCWD) operates three wells located between Meiners Oaks and the Highway 150 crossing. The VRCWD produces approximately 1,200 ac-ft/yr of water. Rancho Matilija Mutual Water Company also operates several groundwater wells along the Ventura River, serving agricultural water to approximately 400 acres. The City of San Buenaventura (City) operates four wells located in the Foster Park area. The City produces approximately 3,900 ac-ft/yr of water from the wells. The amount can vary significantly based on the amount the city extracts from the surface diversion at that location.

There are also several groundwater wells upstream of Matilija Dam. A list of their locations is given in Table 3.1. The well with the lowest elevation is still more than 60 feet above the elevation of the dam crest. No well reaches below the level of the dam crest (1097 ft). The well with the lowest elevation is still more than 60 feet above the elevation of the dam crest.

The infiltration to the Upper Ventura Aquifer occurs through the bed of the Ventura River. The bed of the Ventura River is predominantly composed of gravel and cobbles, with some sand. The median particle diameter in the bed of the Upper Ventura River is over 100 mm (about 4 inches). There is almost no silt or clay in the river bed. This is based on field samples collected at almost 20 sites along the Ventura River (Section 5.3). Because the bed of the Ventura River is composed of coarse material, water is able to seep quickly through the bed. The Upper Ventura River Aquifer is recharged during the wet season as river flows percolate into the aquifer.

Table 3.1. Location and Depth of Wells Upstream of Matilija Dam.

State Well Number	Total Depth (ft)	Water Depth (ft)	Rated Flow (gpm)	R.P. Elevation (ft above msl)
5N/23W-19N01	27	11	5	1171.1
5N/24W-16R01	100	25	40	1726.2
5N/24W-22B01	100	38	45	1546.0
5N/24W-23E01	50	36	?	1475.9
5N/24W-23E02	30	?	?	1470.0
5N/24W-23E04	?	?	?	1503.1
5N/24W-23E05	100	21	15	1463.2
5N/24W-23E06	100	55	15	1464.0
5N/24W-23F01	16	6	?	?
5N/24W-23F02	38	24	13	1490.0
5N/24W-23F03	40	14	15	1444.0
5N/24W-23F04	30	9	5	1502.7
5N/24W-23F05	88	25	?	1438.0
5N/24W-23F06	83	21	20	1482.0
5N/24W-23F07	82	16	30	1503.0
5N/24W-23F08	100	31	22	1497.2
5N/24W-23F09	80	21	30	1442.1
5N/24W-23F10	124	18	30	1492.8
5N/24W-23F11	100	22	25	1442.3
5N/24W-23F12	?	?	?	?
5N/24W-23F13	49	19	10	1442.0
5N/24W-23F14	50	20	10	1441.0
5N/24W-23G01	25	7	20	1451.4
5N/24W-23G02	65	?	?	1450.6
5N/24W-23G03	84	23	22	1503.0
5N/24W-24HS1	(This is an unregulated/non-measured natural spring)			

3.1. Future Without-Project Groundwater Hydrology

If current groundwater use remains the same, there is no change expected to the future Without-Project Groundwater Hydrology. However, additional development in the Upper Ventura River may increase the rate of pumping from the aquifer and this could lower the groundwater levels on average.

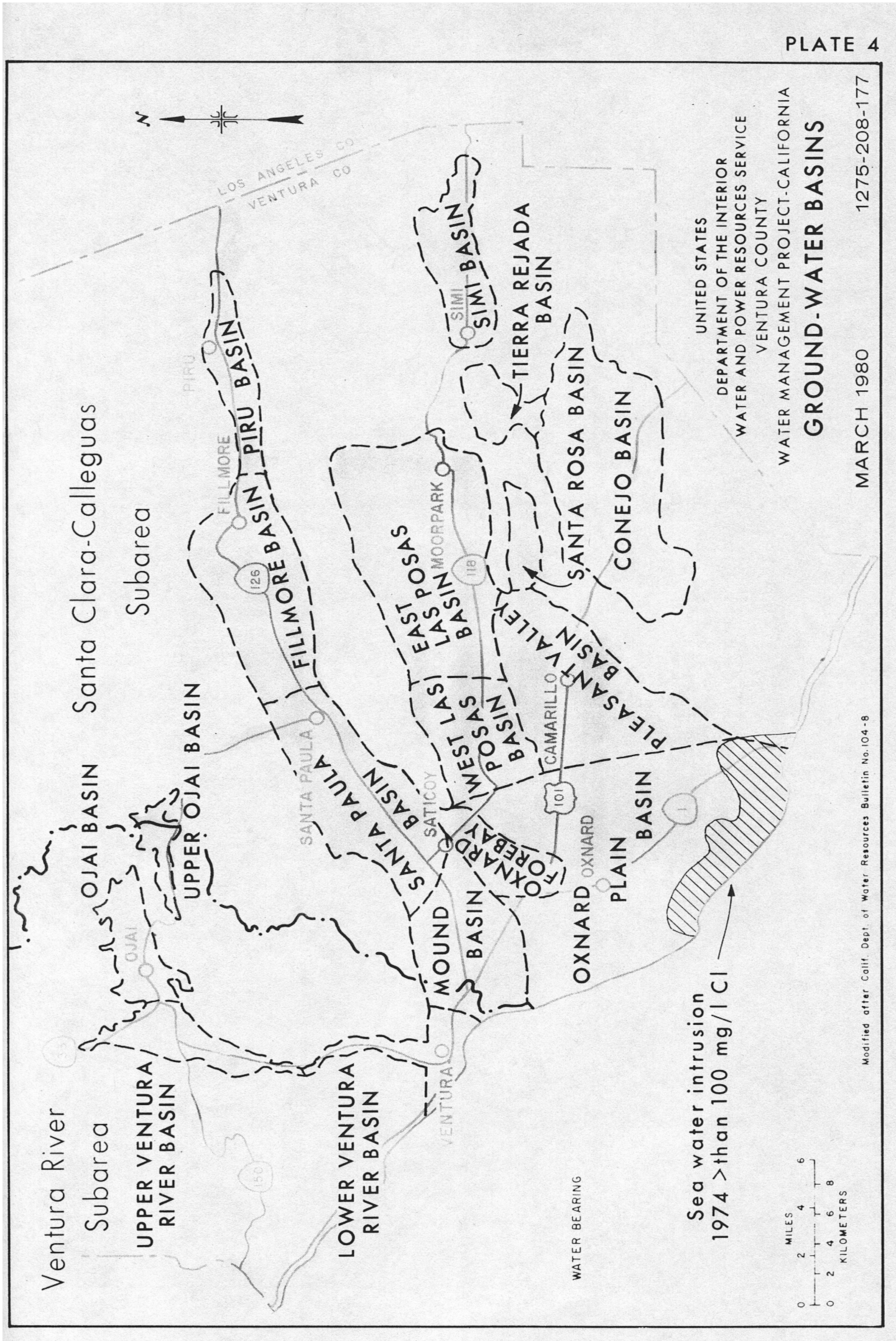


Figure 3.1. Map of groundwater basins in Ventura County. From Reclamation (1981).

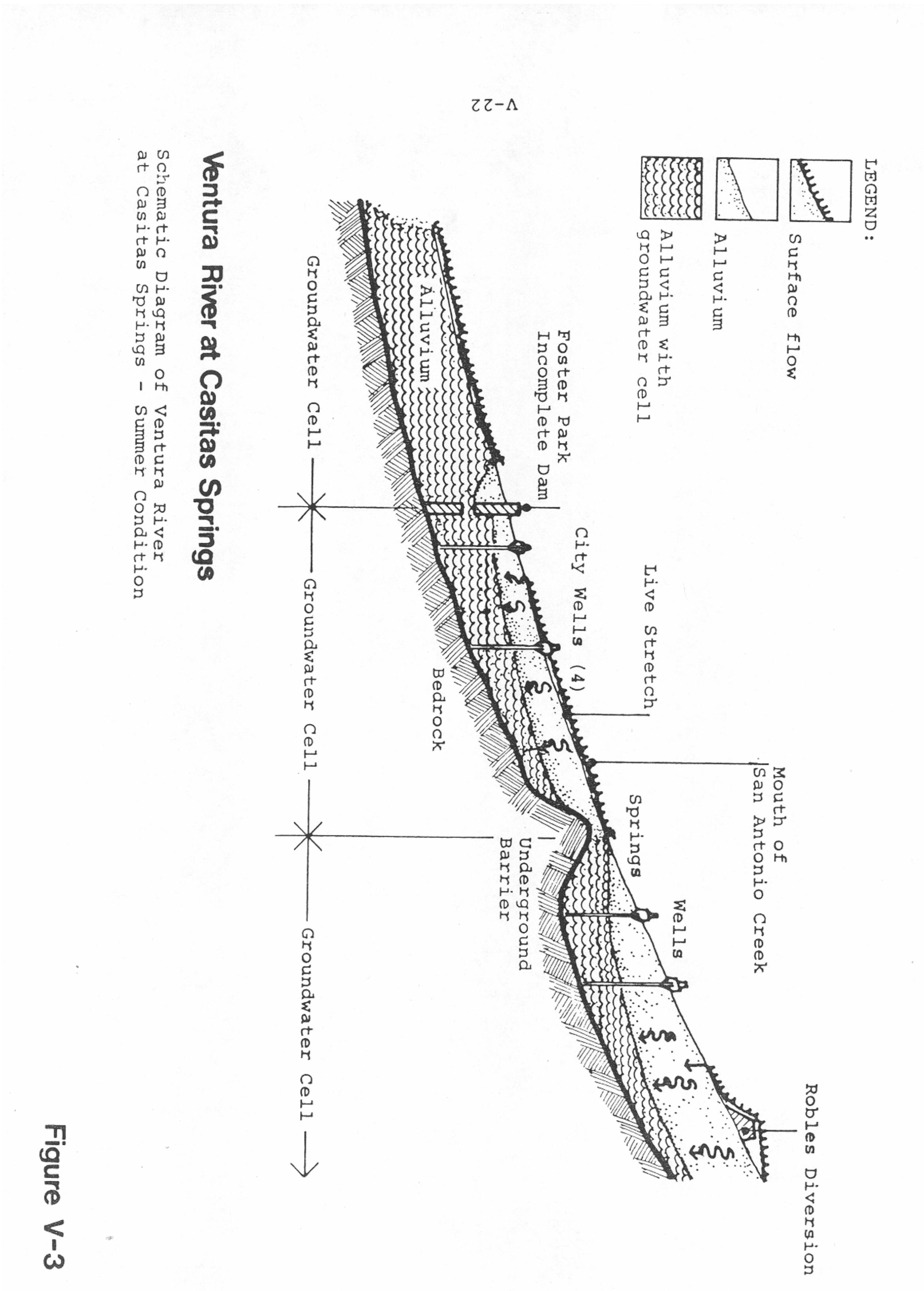


Figure 3.2. Schematic of groundwater basins below Ventura River (Turner, 1971).

4. Without-Project Hydraulics

The Mid-Pacific Region of the Bureau of Reclamation developed digital terrain models (DTMs) and orthorectified photographs for the project reach based on an October 10, 2001 aerial survey flight. Microstation CADD and InRoads software programs were used to develop design surfaces and create geometry for a hydraulic model. Cross sections were constructed at approximately 500 feet intervals along the project reach.

The U.S. Army Corps of Engineers computer program HEC-RAS 3.1.1 simulated the hydraulics for each flood using the cross section data developed in Microstation. Station-elevation coordinates were visually compared to the topographic mapping results and orthophotographs.

Eight bridges were field surveyed to more accurately model bridge geometry throughout the project reach. Simulations of the With Project Conditions assumed a revised bridge geometry at Santa Ana with increased flow conveyance.

4.1. Hydraulic Roughness

Channel roughness coefficients (Manning's n-values) were estimated using photographs of the 1998 flood at Casitas Levee and the rating curve at the Foster Park USGS stream gage in addition to engineering judgment based on published studies of streams in southern California. The 1977 Flood Insurance Model used a value of 0.035 for the main channel. A recent study of Mission Creek in Santa Barbara, California, a coastal stream near the Ventura River, by the LAD Corps of Engineers used a roughness coefficient of 0.035 for the main channel. Mission Creek has a sand, gravel, and cobble bed material. The d_{50} for the upstream sediment gradation on Mission Creek is approximately 50-60 mm. On average, the d_{50} of the bed material along the Ventura River is approximately 100 mm, indicating a courser gradation. Field investigations concluded that the bed material along the Ventura River coarsens from the ocean to Matilija Dam.

A reasonable assumption for the Manning's n-value for the main channel is 0.045. The roughness was increased to 0.065 in Matilija Canyon to reflect the large boulders present. This assumption was based on a selection of Manning's n-values as shown in the widely accepted U.S.G.S. publication from Barnes (1987). A sensitivity analysis evaluated the significance of varying the roughness coefficient along the main channel from a low estimate of 0.035 to a high estimate of 0.055. Table 4.1 shows the results. All simulations used a floodplain roughness coefficient of 0.08.

Table 4.1. Results of Manning's n Sensitivity Analysis.

Reach	**Δ WSEL n = 0.035**	**Δ WSEL n = 0.055**
Ocean to Casitas Vista Bridge	0.0	0.0
Casitas Vista Bridge to Santa Ana bridge	0.32	0.32
Santa Ana to upstream of Robles	0.27	0.27
upstream of Robles to Matilija Dam	0.21	0.21

The analysis indicated 0.2 to 0.3 feet of difference in computed water surface elevations for the 100-year event. The small differences validate using a main channel Manning's n of 0.045.

For much of the river, the flow nears a Froude number of 1. Critical depth controls the water surface more than the roughness coefficients. Interpolating additional cross sections may be necessary to improve the accuracy of the flow modeling. In some cases, interpolating additional sections may also decrease the Froude number and shift more water surface control to the roughness coefficient.

Calibration and Verification

The hydraulic model was calibrated based on observed data at the Foster Park gage. The rating equation developed for the Foster Park gage is the following:

$$WSEL = 0.3\,Q^{0.33} + 207.23$$

where: Q = discharge in ft^3/s.

The hydraulic model matched the lower return periods with reasonable accuracy. However, at discharges above 12,000 – 15,000 ft^3/s, the model does not tend to calibrate well to the rating curve. This is partially because out of the approximately 283 measurements at the gage, only seven are above 15,000 ft^3/s. This implies that there may not be enough data to define the upper portion of the rating curve.

4.2. Overflows

Overflows were mapped for the 10, 50, 100, and 500-year return periods using results from the hydraulic model. The overflow Figures are presented in Exhibit D and show the inundated areas along the Ventura River for the study reach. Mapping assumed constructed levees will not erode or be significantly damaged during flood events. Levees fail to perform only when overtopped. The hydraulic model treated portions of a section inundated because of levee overtopping as ineffective flow areas. Overflow mapping neglected natural levees and expanded the floodplain into areas hydraulically disconnected from the channel under current conditions, but within the historic flow path and below the current water surface elevation. In many cases, this assumption results in similar flood boundaries for events of different magnitudes. This assumption results in a more conservative estimate that accounts for potential changes in planform during large flood events.

4.3. Flood Risk Assessment

The properties at risk are identified in the sections below. They are identified by reach and RM.

Reach 6b – RM 16.5-15.0

Reach 6b begins immediately downstream of Matilija Dam and extends downstream to the canyon mouth. This reach contains little development except the "Matilija Hot Springs" facility.

While events do not inundate the pool itself, flows above the 50-year event inundate the lower grounds.

Reach 6a – RM 15-14.15

Reach 6a begins at the canyon mouth and extends downstream to Robles Diversion Dam. There are approximately 50 structures located near the Ventura River in Reach 6a.

Camino Cielo: There are at least two houses situated along the south bank of the river on the floodplain surface, one upstream, and one downstream of the Camino Cielo Bridge. There are nine structures that appear to be primarily vacation cabins, located upstream of the Camino Cielo Bridge on the north bank of the channel. They are located at a variety of elevations, with the highest being some ten feet above the floodplain surface, and at least five of these being less than one foot above the floodplain surface. The canyon is extremely narrow at this point, with a minimum width of 280 feet, and is only a short distance downstream of Matilija Dam. These structures have a considerable risk of inundation, under both the without- and with-project conditions. Numerous structures are located within 50 feet of the channel bank. All but the structures on the high terrace are within the 100-year floodplain.

Figure 4.1. Downstream side of Camino Cielo.

Meiners Oaks Area: There are approximately 20 structures located along Oso Road and North Rice Road between RM 14.4 and 14.15 within Reach 6a. (There are additional structures within this community downstream of 14.15, but located in Reach 5.) All of these structures are constructed at grade, with no significant first floor elevation above the floodplain. There is no functional levee and all of these structures are above the 100-year floodplain.

Robles Diversion: Robles Diversion Dam is located at the end of Reach 6a. The diversion crosses the Ventura River channel and is within the 100-year floodplain.

Reach 5 – RM 14.15 – 11.27

Reach 5 starts from downstream of Robles Diversion Dam and continues until Baldwin Road Bridge.

Continuation of Meiners Oaks Area: There is a horse stable, a residence, and appurtenant structures located south of Meyer Road within the 100-year floodplain. All of these structures are constructed at grade, with no significant first floor elevation above the floodplain and there is no functional levee. Above RM 13.83, the Meiners Oaks area lies within the Cozy Dale drainage basin with a substantial barrier to potential channel migration into the area. The steep slope of the tributary is expected to prevent backwater influence on the inundation level so the area was excluded from the inundation study. Below RM 13.83 historic photos show active channels in the area. The floodplain was extended to the historic migration extents.

Reach 4 – RM 11.27 – 7.93

Reach 4 starts from downstream of Baldwin Road Bridge and continues until San Antonio Creek.

Live Oak Acres: The Live Oak Levee begins at Ventura River Mile 9.39 on the right bank upstream of the Santa Ana Bridge. It extends along the populated area of Live Oaks to approximately river mile 10.23. The levee itself is joined to the fill of Burnham Road at the upstream side preventing it from being overtopped from the upstream end. This levee contains the 100-yr flood. However, it was necessary to lower the bed elevations at the Santa Ana Bridge based on the maintenance program of the County of Ventura. The Santa Ana Bridge is a severe constriction on the flow. This causes a backwater upstream of the bridge and increases the likelihood that the Live Oak Levee will be over topped. Another repercussion of the bridge constriction is that the scour around the bridge is increased, as evidenced in the photo taken after the 1998 flood (see Figure 4.3). Downstream of Santa Ana road, the floodplain was extended to the limits of historic activity due to uncertainty in the future location of the river.

Reach 3 – RM 7.93-5.95

Casitas Springs: There are at least fifty mobile homes in close proximity to the channel at RM 7.85. The channel at this location is less than 10 feet deep and highly choked with vegetation. The entire mobile home park is at risk of flooding. There is no protective levee at this location. There are numerous structures on Ranch Road, Edison Drive, and Sycamore Drive at Casitas Springs. The protective levee at this location does not provide protection during the 100-year flood.

The Casitas Springs Levee starts on the left bank at approximately Ventura River Mile 6.84 and extends upstream to approximately river mile 7.77. Inundation occurs at the Casitas Levee. Specifically, the 100 and 500-year flood peaks overtop the levee at approximately river mile 7.77. This effectively causes split flow to occur between the channel and the left over bank.

Except for the 500-year flood peak, additional flow from the main channel does not flow over the levee between river mile 7.77 and 7.39. Between river miles 7.39 and 7.29, the 50, 100, and 500-year flood peaks all overtop the levee and can add additional flow to the floodplain. River flow returns to the main channel and is contained again at approximately Ventura River Mile 6.72. Figure 4.2 is photographic evidence of the potential flood risk at Casitas Levee. It is a picture of the river at near peak flood stage during the 1998 flood event, an event with a return period less than 20 years. The water surface elevation for this flood is within 2 feet of the top of the levee.

There are at least three residences located on the south bank of the river downstream of Casitas Vista Bridge (~ RM 6.8). Foster Park is located within the 100-year floodplain and is at risk of flooding.

Further downstream, there are residences, a school, the City of Ventura Water Filtration Plant, and a gasoline refinery located on the south side of the channel. These structures are all located within the 100-year floodplain.

Reach 2 – RM 5.95-0.6

There is a waste treatment facility at RM 5.0 operated by the Oaji Valley Santitation District. The treatment plant is not inundated by the 500-yr flood, but there are the sludge ponds just upstream of the plan are inundated by the 500-yr flood.

The Ventura Levee extends from the Pacific Ocean at Ventura River Mile 0.05 to 2.37. The hydraulic model indicated that all discharges from the 2-year to the 500-year are confined to the main channel by the Ventura Levee.

Figure 4.2. Picture of the Ventura River at the Casitas Levee on 2-24-1998. Picture was taken by William Carey of the Ventura County Watershed Protection District.

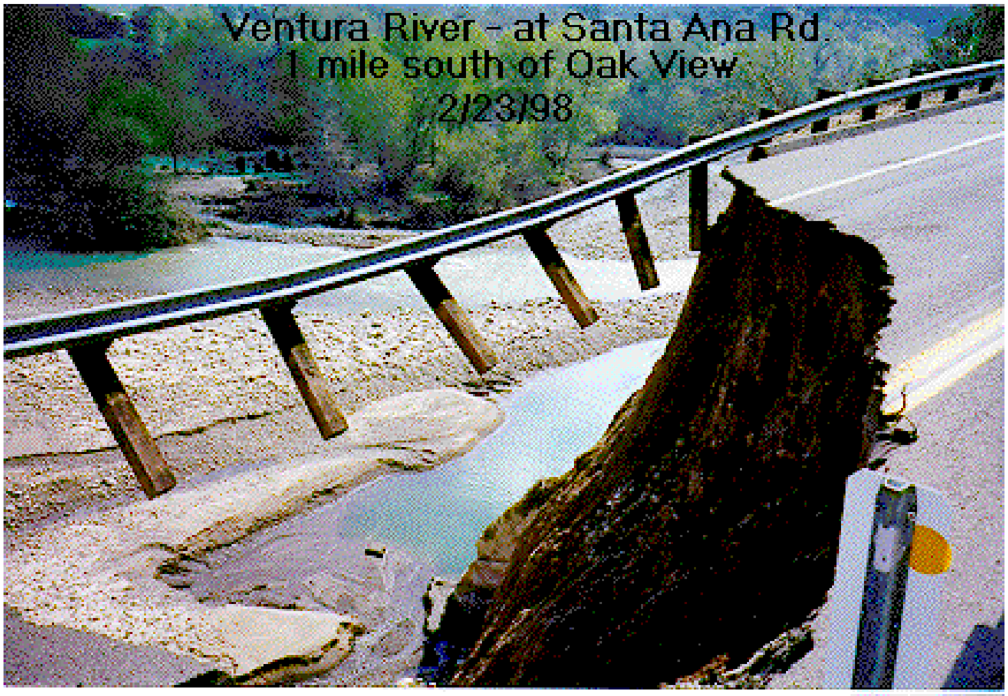


Figure 4.3. Picture of the looking downstream on the Ventura River at Santa Ana Bridge. Picture was taken after the 1998 flood on 2-23-1998.

4.4. Future Without-Project Hydraulics

Erosion is expected in the reach between Foster Park Bridge and Shell Road Bridge (RM 5 to RM 3). This reach has experience erosion in the past and it is expected to continue. The channel is already incised through this reach, so additional degradation of the reach will not alter the hydraulic characteristics significantly. There is no current flood risk in this reach.

Additional sediment transport modeling has shown that 2 to 4 feet of deposition is possible in the next 50 years in the reach protected by Casitas Levee. The deposition will further reduce the levee's level of protection. Currently, it has protection up to approximately the 50-yr flood and it would reduce its protection to approximately the 20-yr flood.

The Santa Ana Bridge and Levee will continue to cause deposition in the future. The County of Ventura has a maintenance plan to excavate sediment at the bridge and therefore the sediment that deposits here will be removed after each significant flood event. It should maintain its current capacity.

After approximately 40 years, coarse sediment will start to spill over the crest of Matilija Dam. After this time, the channel will slowly start to return to an equilibrium condition. The effect of re-supplying Matilija Creek sediment to the Ventura River will be most noticeable in the upper reaches of the Ventura River (i.e. above San Antonio Creek). It is expected that change will occur very slowly with an approximate pre-dam sediment supply being obtained in 100 years.

More description of Future Without Project Conditions is found in Section 9.1 under the description of the No Action Alternative.

5. Without-Project Channel Morphology, Sediment Transport, and Reservoir Sedimentation

5.1. Physiographic Setting

The Ventura River drains about 223 square miles on the southern slope of the Transverse Range. Total relief in the basin is about 6010 feet from Monte Arido on the northwestern margin of the basin to the mouth of the river at the Pacific Ocean. The northern margin of the basin is located less than 25 miles from the ocean. During the late Pleistocene (i.e., the last 100,000 years), the history of the Ventura River has been marked by erosion and incision (Putnum, 1942). The bedrock within the Ventura River basin is comprised exclusively of marine and terrestrial sedimentary rocks. These rocks vary significantly in their composition and relative resistance to erosion. This variability is exhibited as the steep ridges and intervening valleys that somewhat parallel the coastline north of Ventura.

In addition, numerous active faults and folds strongly influence the position of tributary drainages to the Ventura River and control the groundwater hydrology. While the stratum that comprises the bedrock in the Ventura River basin is highly deformed by recent tectonic activity (Dibblee, 1987; 1988; Rockwell and others, 1988), in general the stratum dips steeply to the south with the oldest strata in the mountainous headwaters of the basin actually being completely overturned. The Ventura River downstream of Matilija Dam generally runs normal to this geologic structure with tributary drainages more closely follow or parallel this structure. The geologic structure and the relative resistance of the bedrock to erosion largely control the geomorphology of the Ventura River.

5.2. Previous Studies of Sediment Yield and Transport

5.2.1. Sediment Yield

Scott and Williams (1978) studied small watersheds (less than 10 mi^2) in the Ventura Watershed. One important point noted by this study was the effect of tectonic uplift in the watershed. This effect can generate large amounts of upland sediments for supply to the streambed. Scott and Williams (1978) identified several mechanisms for sediment movement in the Ventura Watershed. Rock falls and landslides are common throughout the area and these events form deposits at the base of steeps hillsides and along the riverbanks. It was determined that rock-fragment flows or dry sliding transport gravel sizes between 2 mm and 64 mm and smaller material. Scott and Williams stated that it is the dominant form of sediment transport on hill slopes in the Ojai area.

Scott and Williams also developed regression equations to estimate sediment yield. The sediment yield resulting from the 1969 flood was measured in 37 debris basins in Los Angeles County. This data was used to develop regression equations that would be applicable to 35 watersheds of Ventura County, including several watersheds in the Ventura River Watershed. Several of these watersheds contribute to San Antonio Creek, and two contribute directly to the Ventura River. Debris flows were found to occur in Cozy Dell Canyon, Stewart Canyon and a tributary to Senior Canyon as the result of the 1969 flood. Cozy Dell Canyon enters the Ventura River just downstream of Robles Diversion. Finally, Fresno Canyon, which joins the Ventura River between San Antonio Creek and Cañada Larga, was also included in the study.

The stream channels in the watershed may experience periods of filling and entrenching. Figure 5.1 (Scott and Williams, 1978) shows a conceptual model of how dry sliding of sediments from the hill slopes can be an upland supply of sediment. This is illustrated by showing sediments depositing in a stream prior to a flood and the degradation that occurs subsequently. However, the periods of filling and entrenchment will be much more pronounced in the upper watershed and smaller tributaries. The main stem of the Ventura River receives relatively little sediment directly from the hill slopes compared to the inputs from the tributaries. Therefore, the main stem of the Ventura River will show smaller elevation changes before and after floods than the upper watershed and small tributaries. Scott and Williams only studied watersheds smaller than 10 mi^2 and therefore their conclusions may not necessarily scale up to the larger watersheds.

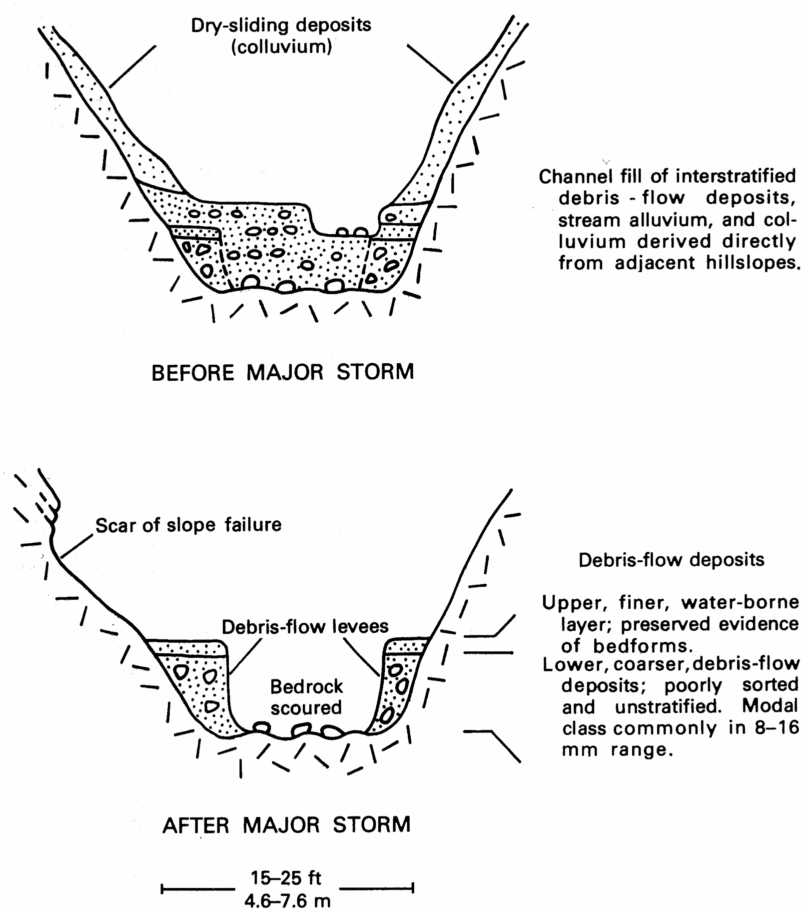


FIGURE 7.—Diagrammatic cross sections of typical bedrock-channel deposits in an area of high erosion rates in the Transverse Ranges.

Figure 5.1. Figure 7 from Scott and Williams. The figure shows cause of sediment transport in small watersheds being dependent upon the previous hydrology.

Several studies have been conducted to estimate the sediment yield for the Ventura watershed. Hill and McConaughy (1988) used sediment discharge measurements from 1969 to 1981 to estimate an annual sediment yield of 2.76 acre-ft/mi^2/yr for the Ventura Watershed. This value was determined with Matilija and Casitas dams in place. The effect of Casitas and Matilija Dams were removed by using the equation:

$$DA_{effective} = (1 - TE/100) \cdot DA_{regulated} \qquad \text{Eq 5.1}$$

where $DA_{effective}$ is the effective drainage area with the dam in place, TE is the trap efficiency, $DA_{regulated}$ is the drainage area regulated by the dam. Removing the effect of the Casitas and Matilija dams and assuming a trap efficiency of 80% for Matilija Reservoir and 100% for Casitas Reservoir, gives a sediment yield of 5.0 acre-ft/mi^2/yr for the entire Ventura River watershed. However, estimates of long-term yield may be high due to the limited dataset of Hill and McConaughy and the inclusion of the floods of 1969. Brownlie and Taylor (1981) estimated that the natural sediment yield (without Casitas and Matilija Dams) to be 2.1 acre-ft/mi^2/yr (1.0 mm/yr) for the Ventura River Watershed for the period between 1933 and 1975. Adding in the effect of Casitas Dam and the current Matilija Dam would give a sediment yield of 1.30 acre-ft/mi^2/yr (0.62 mm/yr). Only adding in the effect of Casitas Dam would give a sediment yield of 1.64 acre-ft/mi^2/yr (0.78 mm/yr).

Studies have also been conducted to estimate the sediment yield of the Matilija Creek Watershed. In a 1954 report, on the feasibility of Water Supply Development, Reclamation estimated the sediment yield to be 1.84 acre-ft/mi^2/yr in the Matilija Creek Watershed. Scott and Williams estimated sediment yields between 1.6 to 6.8 acre-ft/mi^2/yr for headwater basins of the Ventura River. Taylor (1983) used the sediment deposited behind Matilija Dam from 1948-1970 to compute a sediment yield of 1.64 acre-ft/mi^2/yr in the Matilija Watershed.

5.2.2. Sediment Load in Streams

Hill and McConaughy (1988) analyzed the sediment load data from USGS stream gage 11118500 (Ventura River near Ventura) from 1969-1973 and from 1975-1981, and from USGS stream gage 11117500 (San Antonio Creek at Casitas Springs) from October 1976 to September 1978. They coefficients of the sediment rating curves they developed for the suspended load of the Ventura River and San Antonio Creek are given in Table 5.1. The rating curves are of the form:

$$Q_s\ (\text{ton/d}) = aQ\,(\text{ft}^3/\text{s})^b \qquad \text{Eq 5.2}$$

where a and b are constants. They developed rating curves for both the total suspended load and the suspended load with a diameter greater than 0.062 mm.

Table 5.1. Sediment rating curve coefficients derived by Hill and McConaughy (1988).

	Total Suspended Load		**Suspended Load > 0.062 mm**	
River	***a***	***b***	***a***	***b***
San Antonio Creek	2.96E-02	1.92	2.19E-05	2.68
Ventura River	3.55E-02	1.75	3.55E-05	2.38

Relatively infrequent floods dominate the movement of sediment in the Ventura River watershed. Hill and McConaughy concluded that during the period of sediment sampling on the Ventura River, 92% of the total sediment transported in the Ventura River occurred during five floods averaging 10 days each. The dominance of flood events is also shown in Figure 5.1 where the years corresponding to the five floods were the only years to show significant sediment transport.

Hill and McConaughy determined that over 98% of the total sediment load in the Ventura River and San Antonio Creek is suspended. Approximately 96 % of coarse sand load (0.062 mm to 2 mm in diameter) is suspended. While larger particles are moved during large floods, these grain sizes comprise a relatively small portion of the total load. The relative amount of coarse material being transported increases with increasing flow rate. However, these large particle sizes dominate the bed material, and are important in determining the channel geometry. In addition, comparing their data against bed load equations, Hill and McConaughy may have underestimated the bed load transport due to inadequate sampling. The bed load is likely much larger than they measured.

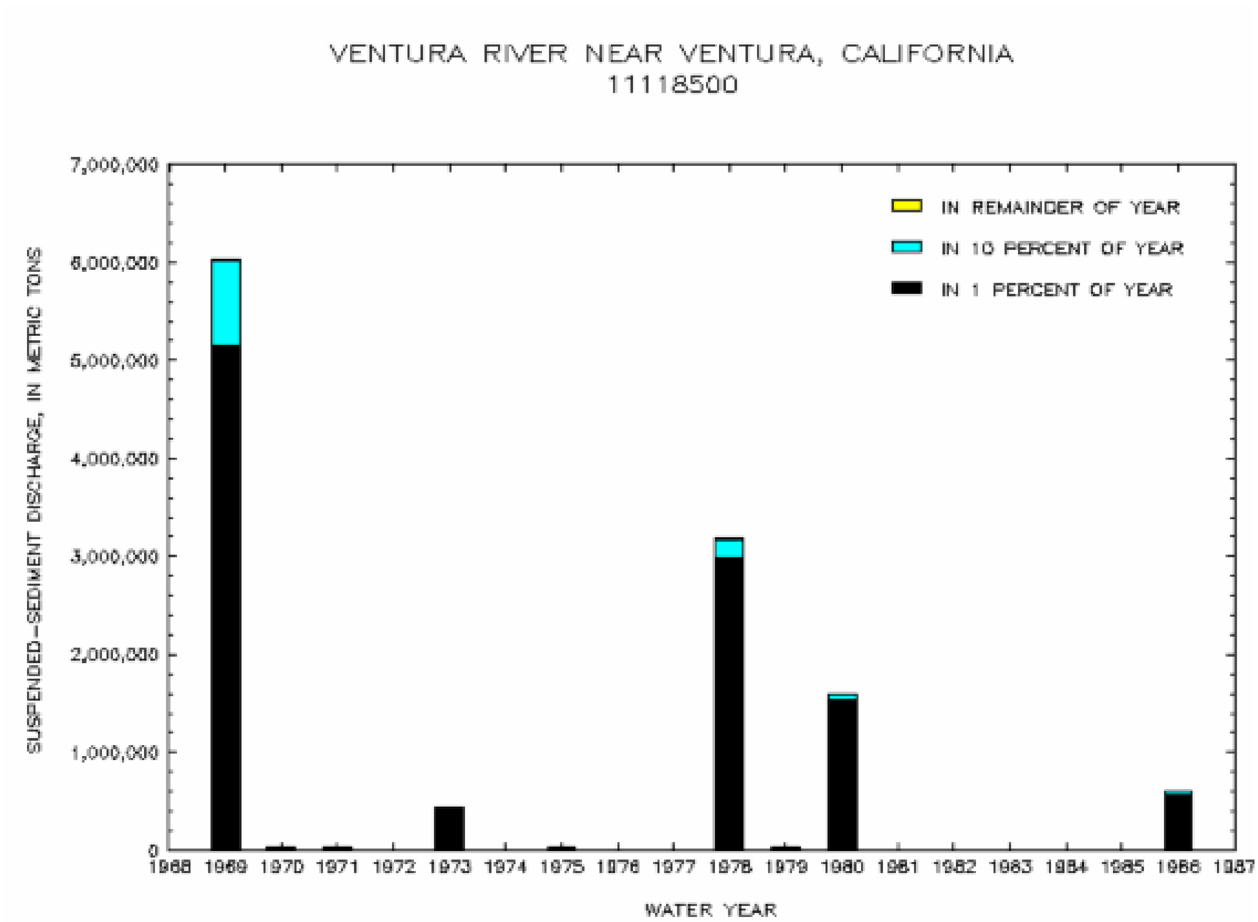


Figure 5.2. Suspended Sediment Loads in Ventura River. There was no data recorded from 10/1/73 to 9/30/74 and from 10/1/82 to 9/30/85 (figure from USGS http://webserver.cr.usgs.gov/sediment/). The year 1983 had substantial flow and sediment transport

As seen in Figure 5.2, the flood of January 18-27, 1969, transported a large amount of sediment. Scott and Williams (1978) estimated the sediment production observed in Cozy Dell and Fresno canyons due to this event. Estimates were also made for other watersheds along the Ventura River by using the developed regression equations as described previously. The characteristics of these watersheds are described in Table 1.3. The results of applying Scott and Williams regression equations are presented in Table 5.2. Hill and McConaughy (1988) estimated the sediment load in the Ventura River for that same period. That analysis concluded the minor drainage basins between Matilija Dam and Foster Park accounted for approximately 16 % of the sediment load in the Ventura River at Foster Park. However, the sediment loads as measured by Scott and Williams were obtained from pre- and post- surveys of debris basins. Debris basins typically allow significant amounts of fine material to pass and therefore the estimates made by Scott and Williams for sediment production could significantly underestimate the amount of fine sediment.

Table 5.2. Sediment Production of Selected Watersheds Resulting from the January 19 – 29, 1969 Flood.

	Drainages east of Ventura River	Drainage Area (mi^2)	1969 Sediment Yield (tons)
E1	1st drainage N. of Cozy Dell Canyon	0.73	19100
E2	Cozy Dell Canyon	1.97	80300
E3	1st drainage S. of Cozy Dell Canyon	0.24	2000
E4	MacDonald Canyon	1.12	37700
E5	Local Drain S. of Meiners Oaks	1.38	2700
E6	Local Drainage in Mira Monte area	1.30	15700
E7	1st drain S. of Mira Monte	1.35	12200
E8	Oakview area local drainage	0.95	9300
E11	Fresno Canyon	1.26	7000
E12	Weldon Canyon	2.19	21900
E13	Manuel Canyon	1.14	2500
	Drainages west of Ventura River		
W1	Kennedy Canyon	1.30	42800
W2	Rice Canyon	0.73	21900
W3	Wills Canyon	1.38	48400
W4	1st drainage S. of Wills Canyon	0.40	7800
W5	Rancho Matilija area drainage	2.32	143800
W6	Live Oak drainage from NW	0.26	4200
W7	Cañada de Rodriguez	1.27	7300
W8	Cañada del Diablo	5.21	83600
	Total of small watersheds		570,200
	Total of small watersheds above Foster Park		454,900
	Ventura River near Foster Park		3,650,000

5.3. Bed Material

Bed Material Sampling Methods

A total of 18 bed material samples were collected in the Ventura River and Matilija Creek. The samples were spaced approximately every mile starting at the mouth and ending 1 mile upstream of Matilija Dam. Two additional samples of beach sand were collected along the shoreline near the mouth of the Ventura River.

Each bed material sample, in the river consisted of a random pebble count of the sediment particles on the surface of the bed. Details on the sampling procedures can be found in Bunte and Abt (2001), but a short description follows. The random pebble count was performed by first delineating an area that was representative of the surface bed material of the river. The area chosen was usually a bar near the main channel of the river that was of similar elevation. The upper portion of the bar was chosen to provide consistency between samples and to be representative of the majority of the surface material in the river. In addition, the upper end of the point bar is where the largest particles entrained have been deposited and is of similar composition to the main channel. Once the area was chosen, two people randomly selected pebbles by averting the eyes from the bed, taking a step, and reaching down with the forefinger. The intermediate axis of the pebble that was first touched was then measured with a metric ruler. Bed material was classified into several classes as presented in Table 13. No less than 100 pebbles were counted at each site. If the particle was less than 4 mm in diameter, it was noted. A bag sample of the material less than 4 mm in diameter was collected at each sample site. The bag samples were later dry sieved in the laboratory. The pebble counts and bag samples were combined by weighting each based on the surface area covered. At three of the sample sites, the three major axes of 25 pebbles were measured. Measuring all three axes gives an estimate of the asymmetry of the particles.

Table 5.3. Definition of Particles Sizes for Sediment Analyses.

Sediment Type	Size Class	Size range (mm)
Fines	clay	0.00024 – 0.004
	silt	0.004 – 0.062
Coarse	sand	0.062 – 2
	gravel	2 – 64
	cobble	64 – 256
	Boulder	256 – 4096

Bed Material Sampling Results

Figure 5.4 contains the representative diameters of the bed material samples as a function of the river mile. The representative diameters are defined as follows: d_{16} is the diameter that 16% of the particles are finer than; d_{50} is the diameter that 50% of the particles are finer than, etc. The representative diameter can be used to characterize each sediment sample. The samples are numbered based on the time at which they were taken and not their location.

The bed is mostly dominated by cobbles, but it contains a large range of sediment sizes. Throughout the entire reach there were sands interspersed between the larger rocks. In the upper reaches near the dam, particles larger than 3 m in diameter were recorded. A top view of typical bed material is shown in Figure 5.3. This photo is near sample site #8, at RM 2.5.

Figure 5.3. Typical Surface Bed Material in Ventura River. Note Large Range of Sizes.

The bed material generally becomes coarser with increasing RM (increasing distance from the ocean). Near the ocean, the d_{50} is approximately 70 – 80 mm, and downstream of Matilija Dam it increases to over 300 mm. In the reach just downstream of the dam, the valley walls are steep and it is possible that some of the large material has its source from the hill slopes in the vicinity. Some of the bed material in this reach may not have been transported by the stream but rather may have been sloughed from the valley walls. Within the study area, the bed material decreases in size upstream of the dam relative to just downstream of the dam.

There are a few notable exceptions to the general trend of increasing particle size with RM. The exceptions are discussed below and can be seen in Figure 5.4.

- Sample #3 (RM 0.6) had a significant amount of sands on the surface. Therefore, the d_{16} was much smaller than the other samples. The large amount of fine material could be because it was closer to the ocean and that the site is immediately upstream of the Main Street Bridge.

- Sample #7 (RM 5.1) is just downstream of the confluence with Coyote Creek and downstream of a more constricted part of the river. Bedrock outcrops control the bed elevation at this location as shown in Figure 5.5 and Figure 5.6. There is only a thin covering of cobbles on top of the bedrock at this site.
- Sample #12 (RM14.4) is just upstream of Robles Diversion. The bed material is finer in this portion of the river because there is an observed decrease in the bed slope in this area.
- Sample #15 is approximately 1.5 miles upstream of Matilija Dam (RM 17.9). The reservoir is approximately 2500 ft in length and the sample site was far enough upstream so that the dam did not affect the bed material size. The bed material upstream of the dam is finer than downstream because the dam traps all the coarse sediment.

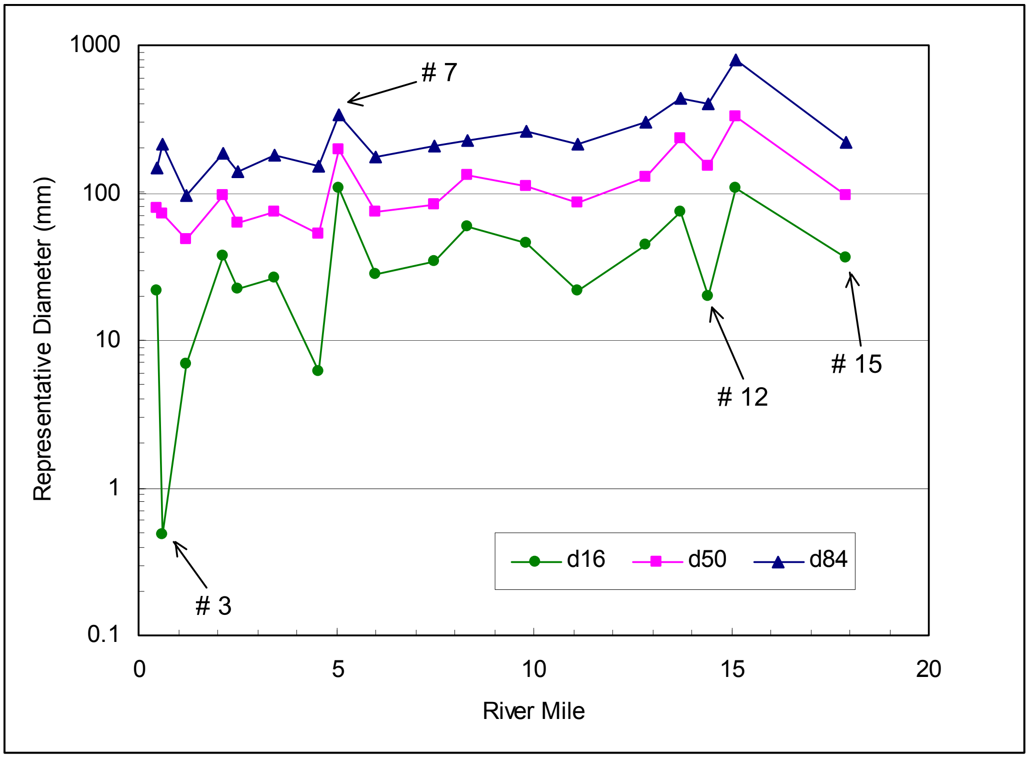


Figure 5.4. Measured representative diameters of surface bed material samples.

Figure 5.5. Bedrock Outcrop at Sample Site #7.

Figure 5.6. Bedrock Outcrop at Sample Site #7.

Downstream of Matilija Dam, the average particle size is directly related to bed slope as shown in Figure 5.7. As the bed slope increases in the upstream direction, so does the average particle size in the bed. The only major exceptions to this correlation between slope and particle size were stated previously.

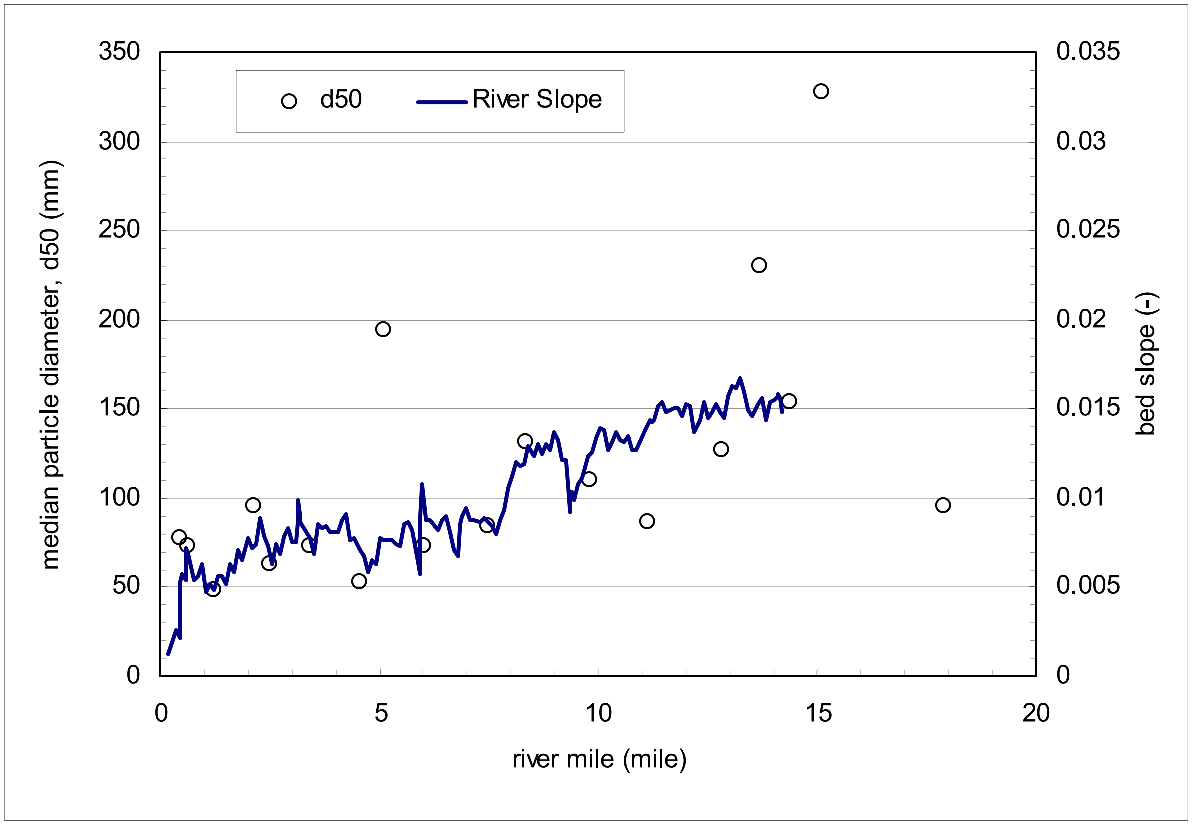


Figure 5.7. Average Bed Slope and d_{50} of Bed Material Samples.

There is gradually less sediment finer than 4 mm found in the bed as river progresses upstream (see Figure 5.8). There are probably two reasons for this. First, most natural river channels become coarser in the upstream direction because the slope is steeper and the river is able to transport coarser material. Second, Matilija Dam traps most coarse sediment and therefore downstream of the dam the bed may have become starved of fine material. Most of the fine material passes through the upper reaches of the Ventura River without depositing on the bed.

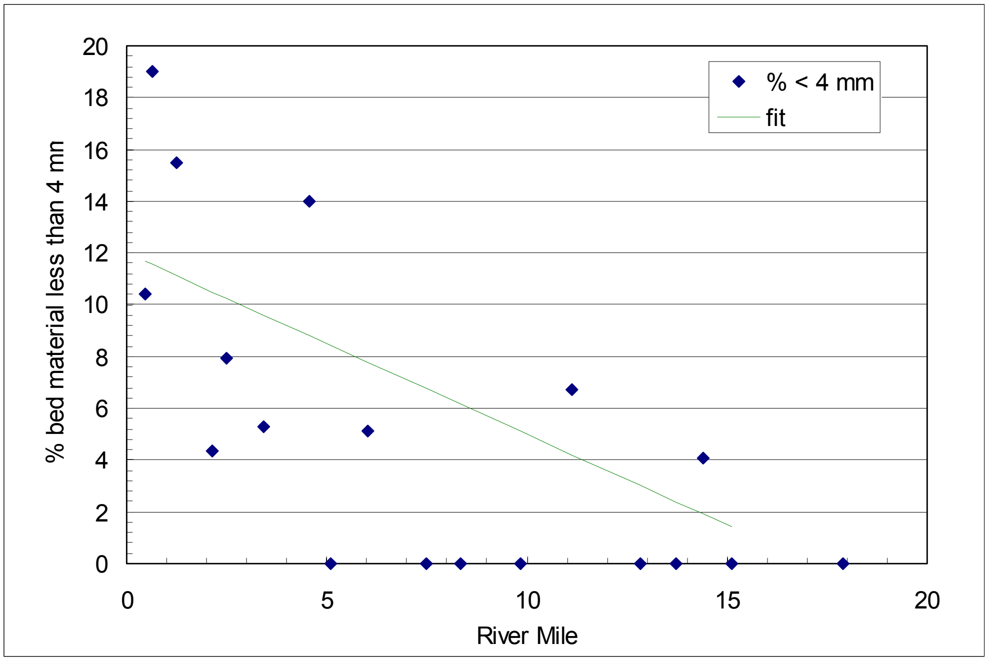


Figure 5.8. Fraction of Bed Material Less than 4 mm.

Hill and McConaughy (1988) reported a bed material gradation that was an average of gradations obtained from sieving field samples and gradations obtained from particle counting and optical methods. Figure 5.9 shows the comparison between the bed gradation measured in Oct 2001 and the one reported in Hill and McConaughy. For material larger than 64 mm (cobbles and larger) the gradations are very similar. However, for material smaller then 64 mm (sand and gravel) the gradations are significantly different. The discrepancy could be caused by one of two things:

1. Different sampling methods. Hill and McConaughy combined three different methods (sieving, particle counting and optical methods) to obtain a composite sample. In the present work, as stated above, only particle counting was used to determine the size gradations. Particle counting procedures are likely to cause underestimation of small particles (Marcus et al., 1995) and therefore, the current gradations may under-represent the quantity of fines in the bed.

2. Erosion of the stream bed since 1988. Erosion would cause the fine material to be selectively removed from the bed.

Based on current estimations of bed degradation, the first reason is the most likely. Even though Matilija Dam has stopped the flow of coarse sediment from Matilija Dam, San Antonio Creek and North Fork Matilija Creek still supply a large amount of coarse sediment to the Ventura River.

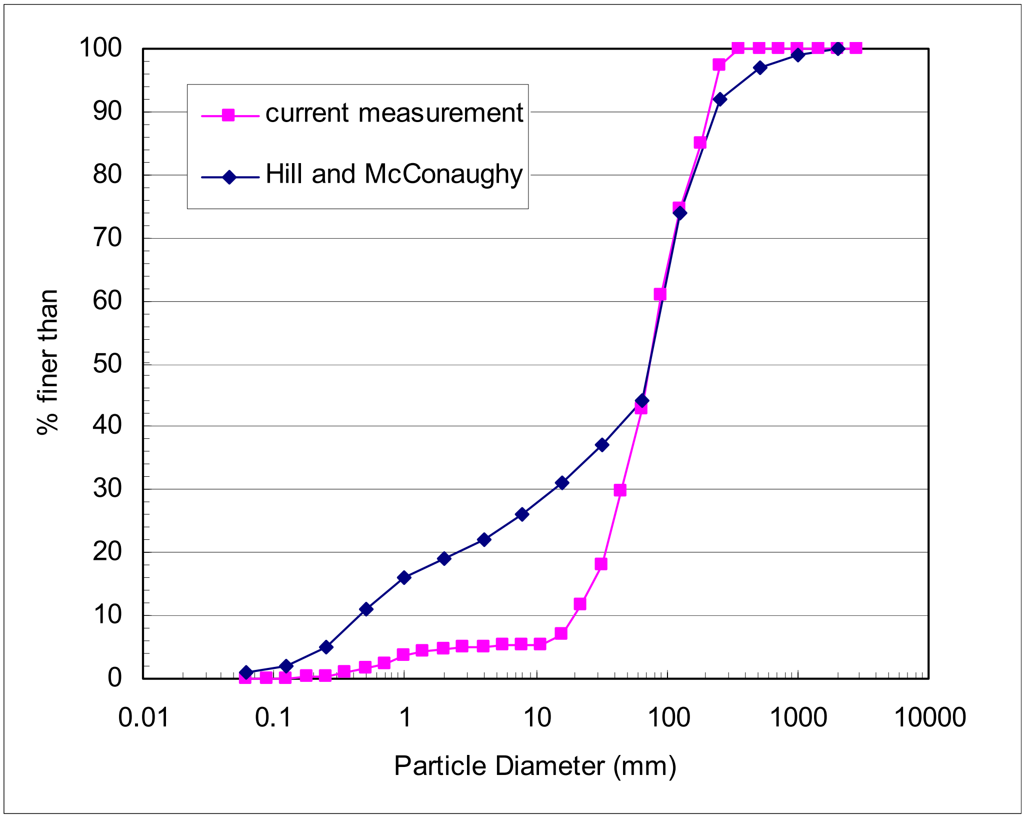


Figure 5.9. Comparison between USGS Composite Sample and Current Measurements of Bed Material near USGS Gage on the Ventura River near Foster Park.

5.4. Deposition in Matilija Reservoir

5.4.1. Historical Deposition

Sedimentation in the Matilija Reservoir has been a concern since its construction (Jamison, 1949; Boyle, 1964). Several surveys have tracked the progression of sedimentation in Matilija Reservoir. In a 1954 report, Reclamation estimated that Matilija was filling in at a rate of 79 acre-ft/yr (Reclamation, 1954). In 1947, a sediment-monitoring program was started to document the sediment deposition occurring in the reservoir. Six silt control lines have been surveyed over a 52 period in the reservoir. These control lines were resurveyed in 1948, 1958, 1964, 1965, 1970, 1986, and 1999. Using CAD technology, the silt control lines were digitized for each year and a volume of sediment trapped in the reservoir was computed using the 1947 silt lines as a baseline. A sediment volume was also calculated for the October 2001 survey. Table 5.4 contains the results of the analysis indicating the deposited sediments in Matilija Reservoir.

The capacity versus elevation relationships are shown in Table 5.5 for the years 1970, 1983, 1994, and 2002. The values for the years 1970, 1983, and 1994 are from CMWD. The values for 2002 were estimated based on a total capacity volume of 500 acre-ft and using the minimum elevation of the reservoir bottom of 1087 ft.

Approximately 2600 ac-ft was lost in 1965 due to the 30-foot notch removed from the dam. The reservoir trap efficiencies were attained from the upper envelop of the Brune curve (Brune, 1953) and are listed in Table 5.4. The Brune methodology defines trap efficiency as the sediment deposited in the reservoir divided by the sediment inflow to the reservoir multiplied by 100 percent. A 15-year moving average was used to calculate the average inflow to the reservoir. The trap efficiencies are subject to uncertainty. This is because the Burne curve does not take into account the extreme hydrological variability that exists in Matilija Creek. Further numerical modeling and comparisons with similar reservoirs would be necessary to develop better models of the trap efficiency. Hill and McConaughy (1988) assumed a trap efficiency of 80% for Matilija Reservoir during the period 1969 to 1981, which is approximately the average trap efficiency assumed for the same period in Table 5.4.

Based on the analysis, it is estimated that Matilija Dam traps approximately 45% of the total sediment that enters it from Matilija Creek. It is estimated that the trap efficiency for sand sizes and greater is still practically 100%. This is evidenced by the small amount of sand located in the downstream portion of the reservoir. Field verification and analysis of borehole samples within this section of the reservoir indicates that these coarser grain sizes are being deposited in the delta or the upstream end of the reservoir. Using this hypothesis would indicate that a large percentage of the fine material (silt size and smaller) passes over the top of Matilija Dam.

The analysis developed using the silt control lines was used to create a depositional history in the reservoir as shown in Figure 5.11. The earliest deposits in the reservoir developed mainly at the upstream end and in the channel region immediately upstream of the dam. Then the 1969 flood deposited approximately 1,000 acre-ft of sediment uniformly over the entire length of the reservoir. Between 1969 and 1978, deposition occurred in the area directly upstream of the dam face. This was in part because the dam height was reduced in 1965. The previous delta, which had formed, when the reservoir water surface elevation was higher, was partially eroded and a new delta developed further downstream. From 1978 to 1986, there was only a small amount of deposition. Deposition increased from 1986 to 1999 and the deposition layer increased slightly over the length of the reservoir. The layer was uniform from 1986 to 1999 because the delta corresponding to the lower spillway elevation was already formed. In general, the grain size is expected to decrease in the downstream direction toward the dam. The transport capacity of the stream decreases toward the dam and therefore Matilija Creek is only able to transport finer material closer to the dam. Matilija reservoir exhibits a traditional reservoir depositional scheme. The upper portion of the reservoir contains gravel size or larger material while near the dam, the sediment deposits are mostly silts and clays.

Figure 5.10. Picture of Sediment Trapped behind Matilija Dam While the Reservoir was Drawn Down. Picture was taken in July 2003 by Paul Jenkin of the Surfrider Foundation.

Table 5.4. Historical Reservoir deposition.

Year	Dam Crest Elevation (NAVD 88)	Reservoir Storage (ac-ft)	Est. Trap Efficiency (%)	Est. Deposited Volume (yd3)	Est. Deposited Volume (ac-ft)
1947	1127.6	7018	95	0.00	0
1958	1127.6	6718	95	920,000	569
1964	1127.6	6488	94	1,200,000	745
1965	1097.6	3856	89	--	--
1970	1097.6	2473	84	2,880,000	1782
1978	1097.6	--	--	4,010,000	2482
1983	1097.6	1480	73	--	--
1986	1097.6	--	--	4,210,000	2606
1994	1097.6	930	56	--	--
1999	1097.6	500	45	5,900,000	3720
2001	1097.6		45		
Trap efficiencies estimated using the medium Brune Curve (1953). Deposited Volume estimated from Silt Control lines, except for 1999 when complete survey was done. -- indicates that no data was available or was not computed.					

Table 5.5. Matilija Reservoir Elevation versus Storage Tables (from CMWD).

	Active Storage Volume (ac-ft)			
Elevation (NAVD 88)	1970	1983	1994	2002 est.*
1042.6	14.2	0	0	0
1047.6	93	0	0	0
1052.6	219	0	0	0
1057.6	367	0	0	0
1062.6	533	57	0	0
1067.6	724	172	0	0
1072.6	947	305	39	0
1077.6	1199	468	153	0
1082.6	1479	662	283	0
1087.6	1789	906	447	24
1092.6	2121	1190	666	250
1097.6	2473	1480	930	500
* 2002 was estimated based on a 500 acre-ft total capacity and zero capacity at elevation of 1087 feet				

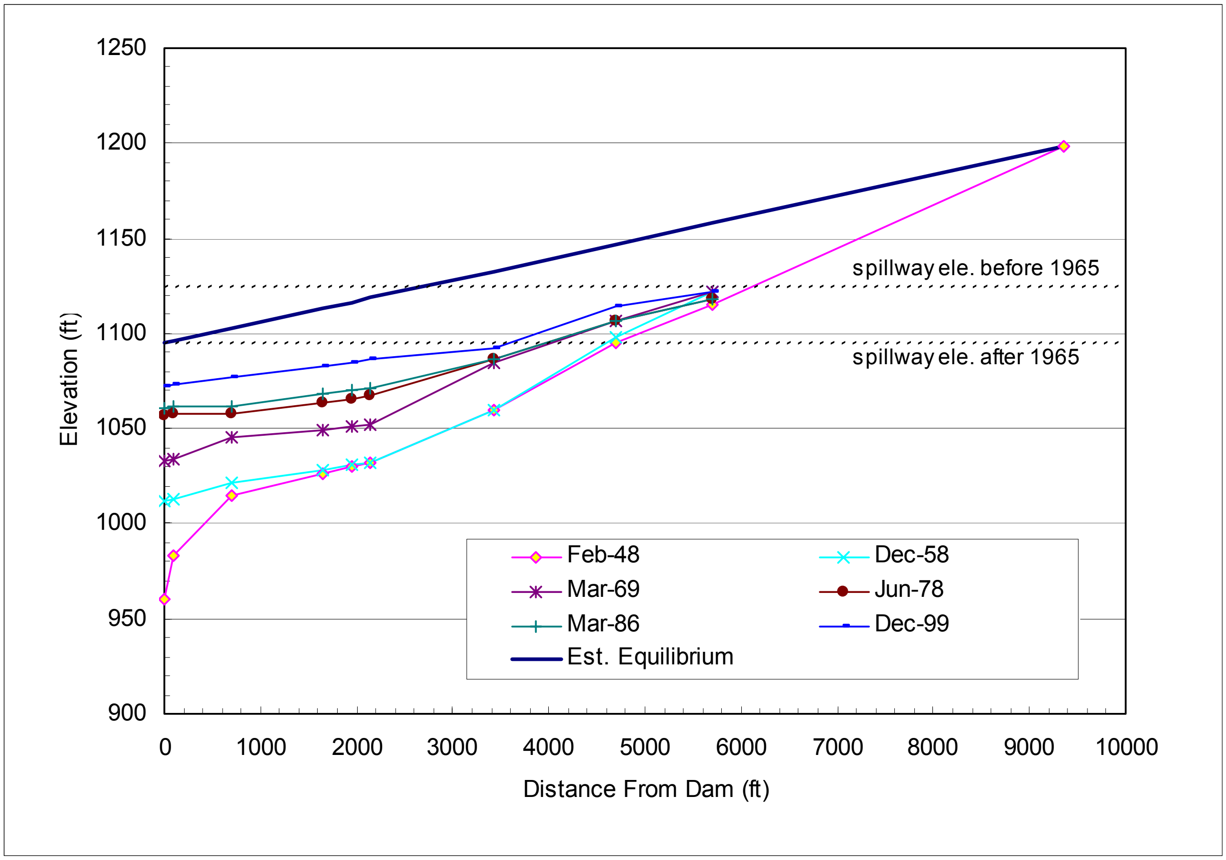


Figure 5.11. Profile plot of depositional history.

5.4.2. SEDIMENT SAMPLING OF TRAPPED SEDIMENT

Based on the core sampling, the Corps determined average gradations for the three different regions of the sediments behind Matilija Dam (Table 5.6). The total volume of the reservoir sediment was determined as well.

Table 5.6. Gradations and Sediment Volume Determined from Drill Data by COE.

	% finer than		
Grain Diameter (mm)	**Reservoir**	**Delta**	**Upstream Channel**
512	100.0	100.0	100.0
256	100.0	100.0	87.9
128	100.0	100.0	75.9
64	100.0	99.8	60.9
32	100.0	98.4	48.9
16	99.9	95.1	36.9
8	99.8	92.5	29.9
4	99.7	89.9	24.9
2	99.7	87.3	21.9
1	99.5	83.7	18.4

0.5	99.0	77.5	15.0
0.25	97.2	66.5	12.0
0.125	92.2	50.8	9.0
0.0625	82.8	33.2	6.0
0.031	70.9	21.9	4.0
0.016	57.3	14.5	2.0
0.008	43.1	9.7	1.0
0.004	30.1	5.3	0.0
0.002	18.0	0.0	0.0
Total Volume (yd^3)	2,100,000	2,800,000	1,000,000

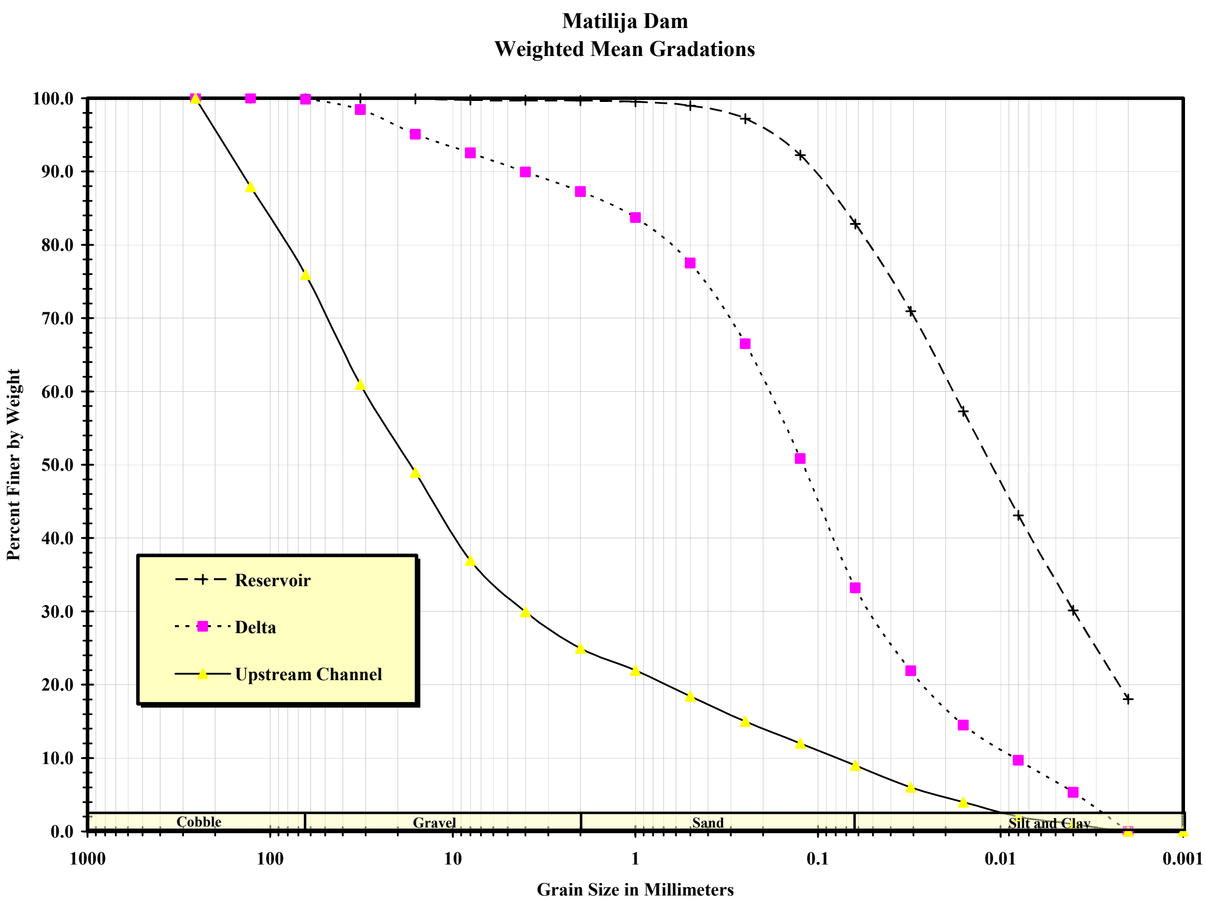


Figure 5.12. Average size gradations of reservoir deposits.

In 1947, a sediment-monitoring program was started to document the sediment deposition occurring in the reservoir. Six silt control lines have been surveyed over a 52 period in the reservoir. These control lines were resurveyed in 1948, 1958, 1964, 1965, 1970, 1986, and 1999. The average elevations from the silt lines are shown in Table 5.7.

Table 5.7. Average elevations of silt control lines (NAD27).

Silt Line	Approximate Distance from Dam (ft)	Feb-48 (ft)	Dec-58 (ft)	Mar-69 (ft)	Jun-78 (ft)	Mar-86 (ft)	Dec-99 (ft)
0	0	960	1012	1033	1057	1061	1072
1	100	983	1013	1034	1058	1062	1073
2	700	1015	1021	1045	1058	1062	1077
3	2140	1032	1032	1052	1067	1071	1086
4	3400	1060	1060	1085	1086	1086	1092
5	4700	1095	1098	1107	1107	1107	1114
6	5700	1115	1122	1122	1118	1118	1122

The *in situ* bulk density of sediment is often difficult to measure because sampling methods tend to compact the sample. Lane and Koezlers (1943) method of calculating the bulk density can be used in such cases. The method accounts for particle-size distribution and the age of the sediment deposit to estimate density. The combined initial bulk density of the sediment can be computed using the equation:

$$W_0 = W_c P_c + W_m P_m + W_s P_s \qquad \text{Eq 5.3}$$

where W_0 is the initial bulk density of the total mass of stored sediment. W_c, W_m, W_s are the bulk densities of clay, silt and sand. Estimates of these values can be found in Lara-Pemberton. P_c, P_m, P_s are the fraction of clay, silt and sand, respectively, as measured by the sampling scheme. To predict the current bulk density of sediments deposited at a given time the following equation can be used (Miller, 1953):

$$W_T = W_0 + K \log T \qquad \text{Eq 5.4}$$

where W_T is the present bulk density, T is the time in years and K is the compaction coefficient, which should be taken from reservoirs with similar operational characteristics. The compaction coefficient can also be estimate in a similar manner to the initial bulk density, W_0:

$$K = K_c P_c + K_m P_m + K_s P_s \qquad \text{Eq 5.5}$$

where K_c, K_m, K_s are the compaction coefficients of clay, silt and sand.

Table 5.8. Reservoir Composition and consolidation parameters.

	Clay	Silt	Sand and Gravel	Total
Percent in Reservoir	30	56	14	100
Weight of Initial Deposit (lb/ft^3)	26	71	97	61
Consolidation Parameter, *K*	16.0	5.7	0	8

Using Table 5.7 it is possible to estimate the ages of the reservoir deposits at various depths. Then, using Table 5.8, the bulk density of the reservoir sediments was estimated.

Table 5.9. Average depths relative to present surface and corresponding bulk densities of reservoir deposits.

Year	Current Depth to Sediments in Reservoir (ft)	Total Volume Deposited (yd^3)	Volume Deposited in Time Interval (yd^3)	Weight Deposited in Time Interval (tons)	Current Bulk Density of Deposited Increment (lb/ft3)
1947	88	0	0	0	
1958	59	920,000	920,000	920,000	75
1969	37	2,880,000	1,960,000	1,940,000	74
1978	16	4,010,000	1,130,000	1,100,000	73
1986	12	4,210,000	200,000	190,000	71
1999	0	5,900,000	1,690,000	1,490,000	68

Averaging over the depth of reservoir sediments gives a computed average bulk density of reservoir sediments of 71 lb/ft^3. Based on the information from the Corps, the measured current average bulk density of the entire reservoir area is 73 lb/ft^3. The measurements of the bulk density indicate that there is no significant stratification of bulk density in the reservoir.

5.5. Sediment Loads and Sediment Yield from Watershed

This section discusses the sediment loads that occur in the Ventura River and its tributaries. The sediment loads information along with the results of previous studies and the depositional history of the Matilija Reservoir is used to compute the sediment yield from the watershed. The effect of forest fires is also discussed.

Sediment Loads in Streams from 1989 to 2002

Sediment rating curves of the form of Eq. 5.2 were developed at three gage locations: 1) North Fork Matilija Creek, 2) San Antonio Creek, and 3) Ventura River near Foster Park. The coefficients are given in Table 5.10. The coefficients are similar to those developed by Hill and McConaughy (Table 5.1). This is a result of the fact that Hill and McConaughy used much of the same data to determine their coefficients. The bed load coefficients for San Antonio Creek and Ventura River were obtained by fitting the USGS bed load measurements with Eq. 5.2. Table 5.11 presents the developed coefficients for floods from 1990 to the present.

Table 5.10. Suspended Sediment Rating Curve Coefficients Derived for the Floods from 1990 Until Present.

	Total Suspended Load		**Suspended Load > 0.062 mm**		**Suspended Load > 0.125 mm**		**Suspended Load > 1 mm**	
River	***a***	***b***	***a***	***b***	***a***	***b***	***a***	***b***
North Fork Matilija Creek	5.70E-02	1.98	8.47E-05[1]	2.76[1]				
San Antonio Creek	2.95E-02	1.91	2.19E-05	2.68				
Ventura River	4.85E-02	1.70	3.56E-05	2.35	2.58E-05	2.35	3.10E-03	1.25

[1]These values are not from measurements. They were inferred from the San Antonio Creek coefficient values.

Table 5.11. Bed Load Sediment Rating Curve Coefficients Derived for the Floods from 1990 Until Present.

	Bed Load		**Bed Load > 1 mm**	
River	***a***	***b***	***a***	***b***
North Fork Matilija Creek	2.76E-01[1]	1.25[1]	2.76E-01[1]	1.25[1]
San Antonio Creek	1.43E-01	1.25	1.43E-01	1.25
Ventura River	9.10E-03	1.25	9.10E-03	1.25

[1]These values are not from measurements. They were inferred from the San Antonio Creek coefficient values.

Uncertainty exists in the coefficients for the coarse suspended and bed load of North Fork Matilija Creek. These coefficients were adjusted to give a similar ratio of coarse sediment to total sediment loads as found in the San Antonio Creek data. Their values were not based on direct measurement of loads. Future work should include direct measurements of the sediment load of North Fork Matilija Creek.

The fraction of the load from the tributaries relative to the load in the Ventura River is shown in Figure 5.13 and Table 5.15. The drainage area of North Fork Creek is approximately 9% of the total drainage at the Ventura Stream gage at Foster Park, and it contributes approximately 17% of the total load and 26% of the sand load. The drainage area of San Antonio Creek is approximately 27% of the total drainage at the Ventura Stream gage at Foster Park, and it contributes approximately 29% of the total load and 44% of the sand load.

The drainage area of Matilija Creek is only 29% of the drainage area at Ventura Stream gage at Foster Park. However, Matilija Creek supplies 49% of the flow to the Ventura River as shown in Table 5.12.

Table 5.12. Fraction of Flow Contributed by Tributaries Relative to Ventura Gage near Foster Park for Selected Floods.

		Fraction of flow contributed		
Date Begin	Date End	Matilija Creek	North Fork Matilija Creek	San Antonio Creek
2/26/91	3/6/91	1.49	0.47	0.71
3/16/91	3/20/91	0.60	0.11	0.27
2/6/92	2/25/92	0.67	0.17	0.28
1/13/93	2/5/93	0.66	0.18	0.36
2/5/93	2/15/93	0.74	0.11	0.49
2/15/93	3/24/93	0.48	0.12	0.23
3/24/93	4/10/93	0.57	0.13	0.27
1/6/95	4/20/95	0.41	0.11	0.17
2/16/96	2/29/96	0.28	0.09	0.33
2/21/98	3/11/98	0.17	0.05	0.11
3/1/01	3/25/01	0.57	0.09	0.20
Average		**0.42**	**0.10**	**0.20**

Table 5.13. Sediment Loads of Selected Floods.

		Total Tons			Total Tons > .062 mm		
Date Begin	Date End	North Fork	San Antonio	Ventura	North Fork	San Antonio	Ventura
2/26/91	3/6/91	2.57E+03	4.39E+03	3.18E+03	5.02E+02	8.73E+02	2.48E+02
3/16/91	3/20/91	1.18E+04	3.91E+04	1.13E+05	2.55E+03	1.23E+04	2.72E+04
2/6/92	2/25/92	1.74E+05	1.59E+05	5.91E+05	1.20E+05	7.39E+04	2.84E+05
1/13/93	2/5/93	1.41E+05	2.75E+05	3.62E+05	4.93E+04	1.39E+05	6.99E+04
2/5/93	2/15/93	2.00E+04	1.06E+05	1.24E+05	5.62E+03	3.82E+04	2.45E+04
2/15/93	3/24/93	7.65E+04	9.47E+04	3.82E+05	1.85E+04	1.96E+04	4.89E+04
3/24/93	4/10/93	2.07E+04	3.48E+04	1.05E+05	5.15E+03	6.63E+03	1.29E+04
1/6/95	4/20/95	4.44E+05	7.88E+05	2.64E+06	2.39E+05	5.85E+05	1.08E+06
2/16/96	2/29/96	1.64E+03	1.26E+04	1.88E+04	3.39E+02	2.77E+03	1.82E+03
2/21/98	3/11/98	4.11E+05	6.25E+05	2.71E+06	3.67E+05	4.48E+05	9.92E+05
3/1/2001	3/25/01	6.19E+04	1.70E+05	7.87E+05	1.97E+04	6.27E+04	2.68E+05
Sum		**1.36E+06**	**2.31E+06**	**7.83E+06**	**8.27E+05**	**1.39E+06**	**2.81E+06**

Table 5.14. Sediment Loads of Selected Floods. Values for North Fork and San Antonio Creeks are not considered reliable and are not reported here.

		Total Tons > 0.125 mm			Total Tons > 1 mm		
Date Begin	Date End	North Fork	San Antonio	Ventura	North Fork	San Antonio	Ventura
2/26/91	3/6/91	–	–	1.87E+02	–	–	3.82E+01
3/16/91	3/20/91	–	–	1.99E+04	–	–	5.55E+02
2/6/92	2/25/92	–	–	2.06E+05	–	–	2.29E+03
1/13/93	2/5/93	–	–	5.11E+04	–	–	2.22E+03
2/5/93	2/15/93	–	–	1.79E+04	–	–	7.63E+02
2/15/93	3/24/93	–	–	3.61E+04	–	–	3.15E+03
3/24/93	4/10/93	–	–	9.52E+03	–	–	9.74E+02
1/6/95	4/20/95	–	–	7.84E+05	–	–	1.20E+04
2/16/96	2/29/96	–	–	1.36E+03	–	–	2.01E+02
2/21/98	3/11/98	–	–	7.21E+05	–	–	1.10E+04
3/1/2001	3/25/01	–	–	1.95E+05	–	–	3.08E+03
Sum				**2.04E+06**			**3.63E+04**

Table 5.15. Fraction of sediment loads from various tributaries relative to sediment load at Ventura River near Foster Park.

		Total Load Fraction		Sand Load Fraction	
Date Begin	Date End	North Fork	San Antonio	North Fork	San Antonio
2/26/91	3/6/91	0.807	1.379	2.026	3.526
3/16/91	3/20/91	0.105	0.347	0.093	0.452
2/6/92	2/25/92	0.294	0.268	0.422	0.260
1/13/93	2/5/93	0.389	0.761	0.705	1.990
2/5/93	2/15/93	0.161	0.855	0.229	1.556
2/15/93	3/24/93	0.200	0.248	0.378	0.401
3/24/93	4/10/93	0.197	0.332	0.400	0.515
1/6/95	4/20/95	0.168	0.299	0.222	0.543
2/16/96	2/29/96	0.087	0.670	0.186	1.519
2/21/98	3/11/98	0.152	0.231	0.370	0.452
3/1/01	3/25/01	0.079	0.216	0.074	0.234
Average		**0.174**	**0.295**	**0.295**	**0.495**

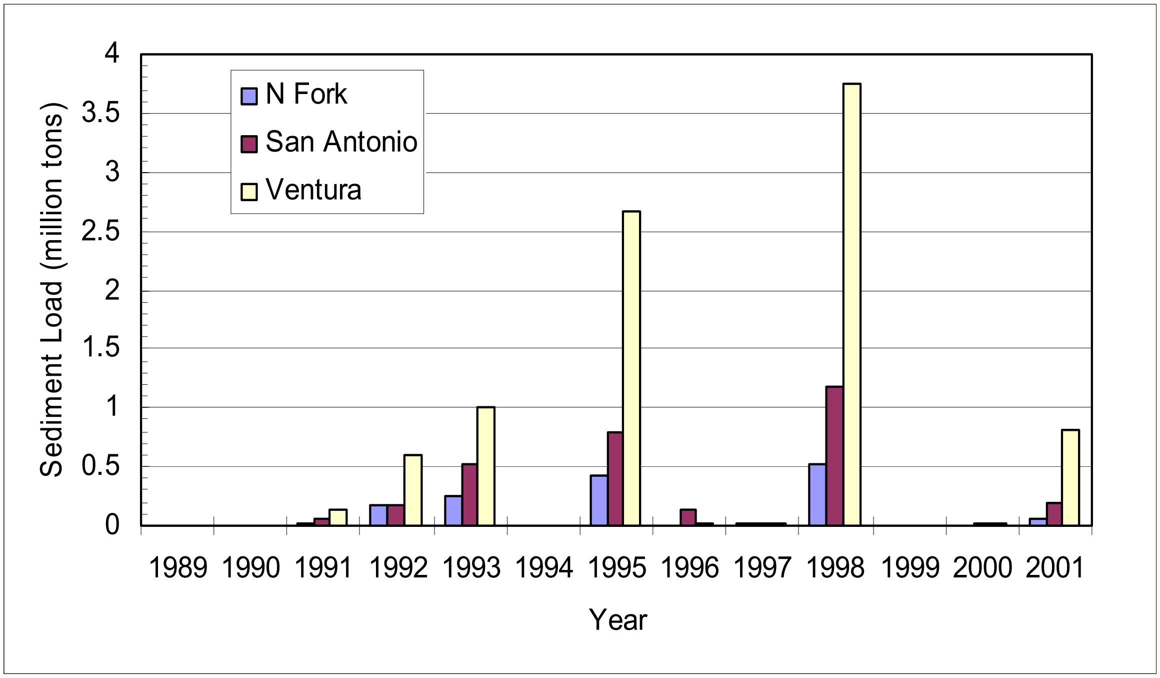


Figure 5.13. Annual sediment loads of rivers in Ventura Watershed from 1991 to 2001.

Long Term Sediment Yields

The previous section analyzed the floods from 1989 until the present. However, this was a relatively wet period and therefore was a period of greater sediment loads. Using this period alone to estimate sediment yield would cause it to be overestimated.

To estimates based on longer records, data from previous studies and the depositional history of the Matilija dam can be used. Based on the estimated 6.0 million cubic yards (3,719 acre-ft) of sediment deposited behind Matilija dam since its construction, and using trap efficiencies as presented in Table 5.4, the sediment yield is estimated to be 1.92 acre-ft/mi^2/yr (0.79 mm/yr) or 105 acre-ft/yr upstream of Matilija Dam. This estimate is similar to the estimate of Reclamation (1954). Downstream of the dam, the sediment yield needs to be modified by the trapping efficiency of the dam, which is currently estimated to be 45%. The best estimate of the long-term sediment yield of the Ventura Watershed without any dams in place is defined by Brownlie and Taylor (1981), who computed it as 2.10 acre-ft/mi^2/yr (1.0 mm/yr). With Casitas and Matilija Dam in place, and assuming the trap efficiency of Casitas is 100% and the trap efficiency of Matilija Dam is 45%, the sediment yield of the Ventura River Watershed is 1.36 acre-ft/mi^2/yr.

Currently, the Matilija Creek Watershed contributes 24% of the total sediment load of the Ventura River at Foster Park. As the reservoir fills, the Matilija Creek Watershed will contribute more sediment until its contribution stabilizes at approximately 37% of the total sediment load at Foster Park. After the reservoir has reached equilibrium and the trap efficiency of Matilija Reservoir is practically zero, the sediment yield of the Ventura Watershed will be 1.64 acre-ft/mi^2/yr.

An estimate of the how much sediment is being eroded from the stream channel can be made by comparing a stream survey in 1971 to the 2001 survey. From 1971 to 2001, 1.9 million yd^3 of sediment was eroded from the streambed from the beginning of the Ventura River until Foster Park. During that same time, approximately 12.1 million yd^3 was transported by the river through Foster Park. Therefore, approximately 16% of the total load originated from the streambed.

The minor drainages between the start of the Ventura River and Foster Park comprise a drainage area of 25.3 mi^2 and contribute sediment. If the 1969 flood is a representative sediment-transporting event, those drainages contribute at least 12.4% of the total load at Foster Park.

The estimated current contributions of watersheds upstream of Foster Park are presented in Table 5.16. The fraction of the total load originating from the minor drainages was increased to 0.13 so that the sum of the total fractions equaled one. The minor drainage fraction was increased because it was considered to have the largest degree of uncertainty. In addition, the sand load fraction of the minor drainages was set to 0.04 so that the sum of the total sand load fractions equaled one.

The fractions listed in Table 5.16 are for the present condition. Because the trap efficiency of Matilija Dam is continually decreasing, the relative contribution of the Matilija Creek Watershed will continue to increase. As the contribution of the Matilija Creek Watershed increases, the relative contribution of the floodplain and channel to the sediment load will decrease.

The current sediment yields are listed in Table 5.17. Based on the current Ventura sediment yield of 1.36 acre-ft/mi^2/yr, 303 acre-ft/yr of sediment is delivered to the ocean. Based on the floods from 1991 until now, the ratio of coarse sediment (> 0.062 mm) to total sediment is 0.36. Therefore, approximately 109 acre-ft/yr of coarse sediment is delivered to the ocean (Table 5.18). The ratio of sediment coarser than 0.125 mm to total sediment is 0.26, means that approximately 79 acre-ft/yr of sediment coarser than 0.125 mm is delivered to the ocean on an annual basis.

Assuming that Matilija Dam remains in place, the sediment yield of the watershed will approach that as if Matilija Dam was never there. The sediment delivery estimates for 50 years from now are listed in Table 5.18.

Table 5.16. Estimated current contributions of sediment load from watersheds upstream of Foster Park.

Watershed	**Fractions contributed Total Load**	**Fractions contributed Total Load (> 0.062 mm)**
Matilija Creek	0.24	0.00
North Fork Matilija Creek	0.17	0.30
Minor Drainages between start of Ventura River and Foster Park	0.13	0.04
San Antonio Creek	0.30	0.50
Floodplain and Channel	0.16	0.16

Total	1.00	1.00

Table 5.17. Average sediment yield in the Ventura River Watershed.

Watershed	Sediment Yield per mi^2 (acre-ft/mi^2/yr)
Ventura Watershed without Casitas Dam and Matilija Dam	2.10
Ventura Watershed with Casitas Dam and Matilija Dam in place (current conditions)	1.36
Ventura Watershed with Casitas Dam in place	1.64
Matilija Creek Watershed	1.92

Table 5.18. Average annual sediment delivery to the ocean.

	yd^3/yr of sediment delivered				
type	**fines**	**sand**	**gravel**	**cobbles**	**total**
Current	311,000	136,000	9,400	530	457,000
Equilibrium Estimation	373,000	164,000	11,300	630	548,000

Forest Fires

The occurrence of wildfire plays a significant role in the augmentation of erosion rates from Southern California watersheds. Highly flammable chaparral species, steep slopes, loose sediments, hydrophobic soil conditions created by the intense heat generated by wildfire, and the aggravating influence of dry offshore “Santa Ana” winds provide Southern California with one of the most volatile fire/erosion complexes in the world (LAD, USACE, 2000). Generally, smaller watersheds are more sensitive to the effects of wildfire. Smaller watersheds have less storage areas for sediment and an increase in supply may quickly be seen by an increase in load downstream. However, the larger watersheds may have a significant time delay between an increase in supply and the corresponding increase in downstream transport. In addition, if the stream is already capacity limited, an increase in supply may not increase the sediment loads.

Since traditionally there is a relationship between suspended sediment and wildfire occurrences, an investigation was conducted, at the Foster Park gage, to analyze this correlation in the Ventura Watershed. Based on the available suspended sediment data, a significant correlation between suspended sediment load and the last significant fire could not be observed. While wildfires generally increase the suspended load, there have only been two fires since the time suspended sediment samples have been collected at the Foster Park gage. These occurrences were in 1979 and 1985.

Table 5.19 contains information on fires that have burned over 5% of the Ventura River watershed as well as their frequency of exceedence. The average recurrence interval for a fire that burns over 5% of the watershed is 13 years. The exceedence probability of a certain percent watershed burn is defined as the probability that a fire will occur within one year that burns equal to or more than that percentage of the watershed.

Table 5.20 contains information on all the fires that have burned in the Matilija Creek watershed. Two fires (1932 and 1985) burned almost the entire Matilija Creek watershed. The next largest fire in that watershed burned only 16.7% of the watershed.

Table 5.19. Fires that have burned over 5% of the Ventura River watershed.

Fire Name	**Date**	**Area (mi^2)**	**Percent of Ventura River Watershed burned**	**Exceedence Probability**
Coyote Creek	7/1/1910	14.85	6.6%	0.0738
29 Sulpher Mountain	9/16/1929	25.30	11.2%	0.0642
Los Padres	9/1/1898	30.77	13.6%	0.0546
Wheeler Springs	9/12/1948	31.90	14.1%	0.0450
Creek Road	9/18/1979	33.96	15.0%	0.0354
Thatcher	6/1/1917	46.33	20.5%	0.0259
Matilija	9/7/1932	85.94	38.0%	0.0163
Wheeler #2	7/1/1985	122.81	54.4%	0.0067

Table 5.20. Fires located in the Matilija Creek watershed.

Fire Name	**Date**	**Area (mi^2)**	**Percent of Matilija Creek Watershed burned**	**Frequency Exceeded (yr^{-1})**
WHEEL	10/27/1993	0.13	0.2%	0.0833
R. COLLA	7/5/1985	0.15	0.3%	0.0738
WHEELER SPRINGS	9/12/1948	0.95	1.8%	0.0642
LOS PADRES	9/1/1898	1.07	2.0%	0.0546
MATILIJA	4/1/1898	1.13	2.1%	0.0450
MATILIJA	7/7/1983	4.65	8.6%	0.0354
THATCHER	6/1/1917	9.07	16.7%	0.0259
WHEELER #2	7/1/1985	53.84	99.1%	0.0163
MATILIJA	9/7/1932	54.05	99.5%	0.0067

Using the data from Table 5.19 and Table 5.20, fire frequency curves were developed for the Matilija and Ventura watersheds. Based on the analysis, there is approximately a 1% chance that a 50% burn will occur in the entire Ventura Watershed in any given year. There is approximately a 2.2% chance that a 50% burn will occur in the Matilija Watershed in any given year. Figure 5.14 presents the fire frequency in the Matilija Creek and Ventura River Watersheds.

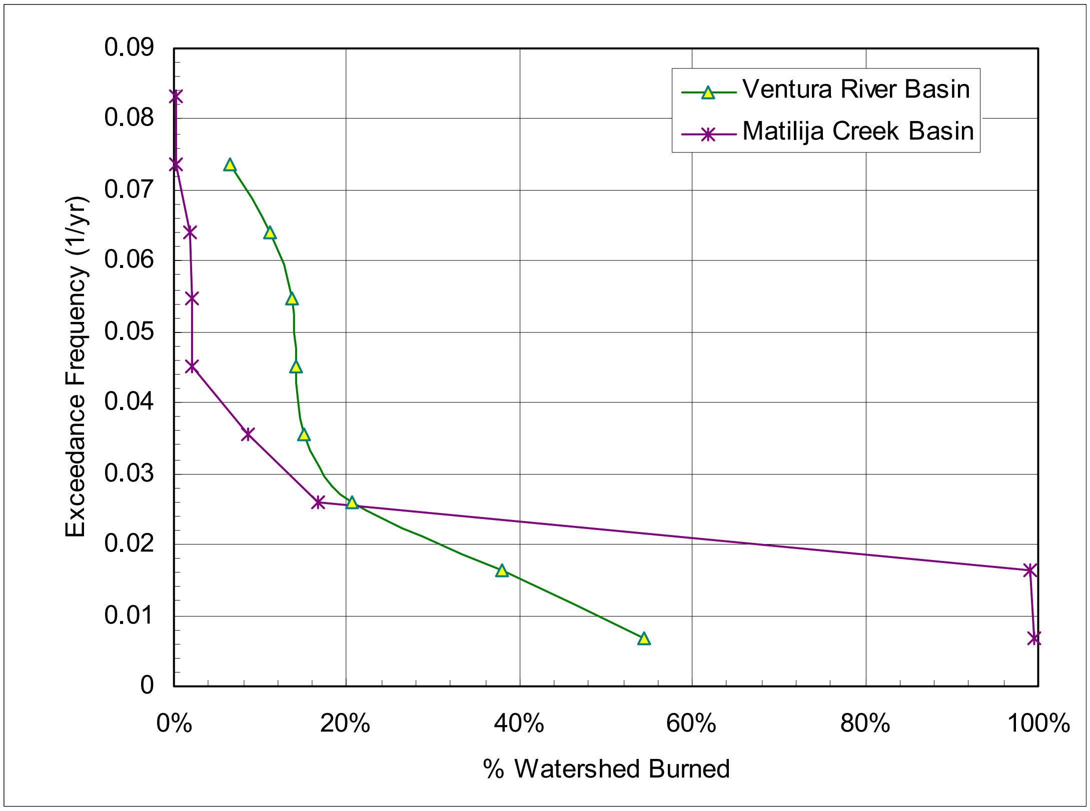


Figure 5.14. Fire frequency in the Matilija Creek and Ventura River Watersheds.

5.6. Static Analysis of Sediment Transport

Critical Shear Stress for Motion and Bed Armoring

This section identifies the flow rates at which sediment particles begin to move in the Ventura River. Incipient motion is defined as the condition under which particles just start to move. Using the results from the hydraulic analysis, the critical diameter for incipient motion is computed for all sub-reaches (see Figure 5.15) using Shield's criteria:

$$\theta_{cr} = \frac{\tau_b}{(\gamma_s - \gamma) d_{cr}} \qquad \text{Eq 5.6}$$

where: θ_{cr} is the non-dimensional critical shear stress, τ_b is the average bed shear stress, g is the acceleration of gravity, γ_s is the specific weight of sediment, γ is the specific weight of water and d_{cr} is the critical sediment diameter.

For a given flow rate, particles larger than the critical diameter are not expected to move in significant amounts. It was assumed that $\theta_{cr} = 0.04$, which is a typical value assumed for gravel bed rivers (Buffington and Montgomery, 1997). The results from the calculations for the critical diameter for the Ventura River are presented in Figure 5.15.

Throughout almost the entire river, the d_{50} is mobilized for floods equal to and larger than the 2-yr flood. This indicates that the average flood will move most of the particles on the bed. The only exceptions to this are areas where there is exposed bedrock or there has been armoring of the bed. Bedrock controls occur from RM 6 to RM 5 where the river is constricted through a narrow canyon. Armoring has occurred downstream of Robles Diversion from RM 14 to approximately RM 13. In the canyon immediately downstream of Matilija Dam, from RM 16 to RM 15 there is also armoring of the bed.

In the lower part of reach 3 (from Foster Park to the Estuary), the 5-yr flood mobilizes the d_{84} of the bed. This indicates that at least 84% of the bed is mobilized. In reach 4 (from San Antonio Creek to Robles Diversion), a 10-yr to 100-yr flood would need to occur to mobilize the d_{84} of the bed. This indicates that the material in the lower part of the river is mobilized more frequently than the material in the upper portion of the river near the dam. It is likely that because Matilija Dam has blocked a large amount of coarse sediment from entering the Ventura River, the upper portion of the Ventura River is more armored. This impact is minimized by the fact that San Antonio Creek is a large sediment supply and offsets the impact of Matilija Dam on the sediment supply.

It is possible to estimate the depth required to armor the bed against motion by using the following equation (Reclamation, 1984):

$$y_d = y_a \left(\frac{1}{\Delta p} - 1 \right) \qquad \text{Eq 5.7}$$

where y_d is the depth from the original stream bed to the top of the armoring layer (i.e. y_d is the depth of degradation), y_a is the thickness of the armoring layer, Δp is the fraction of the original bed material larger than the armor size. The armor size can be found by using Shields criteria (Eq. 5.5).

The depths to an armored layer for the 2-, 10-, and 100-yr flows are found in Figure 5.17. If no line is shown, the armor size was larger than the d_{95} at that section and no armoring may occur for the current bed material gradation. In reach 6 (the canyon immediately downstream of Matilija Dam), the bed is relatively well armored and not subject to large degradation. Reach 5 (the reach immediately upstream of Robles Diversion), is shown to be subject to over 6 feet of erosion during the 100-yr flood. However, this reach is controlled by the diversion structure and is in a natural depositional zone and therefore significant erosion is not expected. Reach 4 is relatively well armored, even for the 100-yr flood. Large events may not scour the bed more than 2 feet throughout most of this reach. Reach 3 is not as well armored, but there are several controls in this reach that may prevent further erosion. The Foster Park Diversion at RM 6.31 will not limit the river bed degradation upstream of this location. Bedrock outcrops between RM 5 and 6 also limit erosion in this reach. There is also some evidence of bedrock outcrops near Shell Road Bridge (Figure 5.29). The reach near the ocean and upstream of the Estuary (RM 1 – 2) is also not well armored and it is uncertain if any bedrock exists there. However, the ocean level may exhibit some control on limiting the erosion in this reach.

Critical Shear Stress for Suspension

Sediment load can loosely be categorized into three types of motion: Suspended load, saltating load and bed load. The following equations define the three categories as described by Raudkivi (1998):

$$\frac{w_f}{u_*} < 0.6 \qquad \text{suspended load}$$

$$0.6 < \frac{w_f}{u_*} < 2 \qquad \text{saltating load} \qquad \text{Eq 5.8}$$

$$2 < \frac{w_f}{u_*} \qquad \text{bed load}$$

where w_f is the fall velocity of the sediment particle and u_* is the shear velocity ($u_* = \sqrt{\tau_b/\rho}$). The critical diameter for suspended load is defined as the diameter below which the sediment moves purely as suspended load. Suspended sediment transport rates are much greater than bed load sediment transport rates.

The critical diameter for suspended load along the Ventura River was calculated using the above equation and is presented in Figure 5.18. The suspended critical diameter is greater than 1 mm for almost the entire river for every flood larger than the 2-yr. The 100-yr flood suspends all particles finer than 2 mm for almost the entire length of the river. This indicates much of the sand load can behave as wash load during large floods. Fine sands are transported into the Ventura River by tributaries or directly from the hill slopes. Then, the fine sand is transported directly into the ocean with little interaction from bedload particles.

Sediment Transport Capacities

Meyer-Peter-Müller sediment transport equations were chosen to compute transport capacity based on engineering practice. Only sizes greater than 1 mm were included in the transport calculation. Meyer-Peter-Müller equation is a bed load equation and does not reliably predict the transport capacity for suspended material. Most sediment transport equations are developed using single size or well sorted bed material. In the Ventura River, the bed material is composed of particles ranging from fine sands to boulders that move during large flood events. A comparison between the computed concentrations and measured sediment load is shown in Figure 5.19. The Meyer-Peter-Müller equation accurately predicts the measured capacities. The measured capacities were taken from the sum of the bed-load and suspended load rating curves for the particle sizes larger than 1 mm.

The capacity concentrations are relatively constant throughout most of the Ventura River with a few notable exceptions. In the canyon immediately downstream of the Matilija Dam, the capacity concentration are quite low, this is largely due to the large bed material at this location. The river is unable to move the bed material in this reach in large concentrations. The sediment concentration decrease again from RM 14 to RM 13, downstream of Robles Diversion because of bed material size increases in this reach. There is a smaller decrease in sediment transport capacity at RM 9 because of the constriction of the Santa Ana Bridge. Because the sediment carrying capacity decreases upstream of the bridge, the area upstream of the Santa Ana Bridge would be an area of deposition.

The sediment capacity has a sharp spike at Casitas Vista Road Bridge for the 100-yr flood. The 100-yr flood is severely constricted by this bridge and the topography and therefore there is a large backwater pool formed upstream of the bridge. This creates a much lower sediment transport capacity and causes the sharp downward spike seen in Figure 5.19.

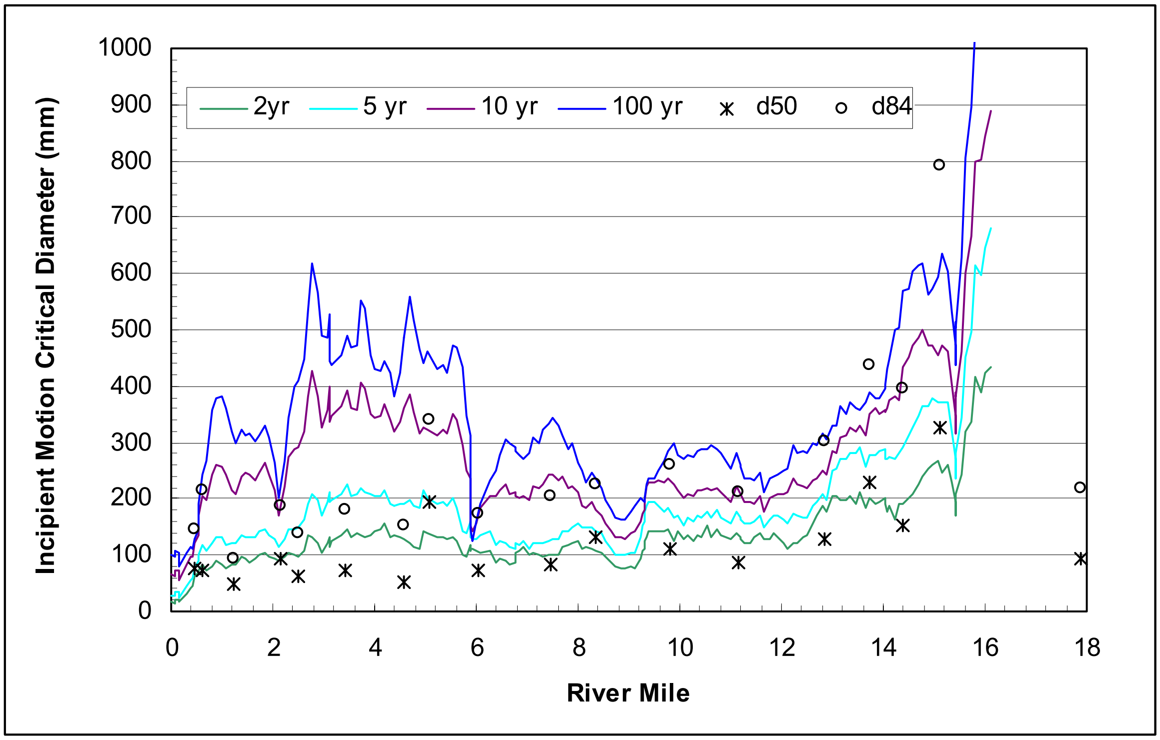


Figure 5.15. Incipient motion critical diameter for the Ventura River and comparison with the d_{50} and d_{84} of the bed material.

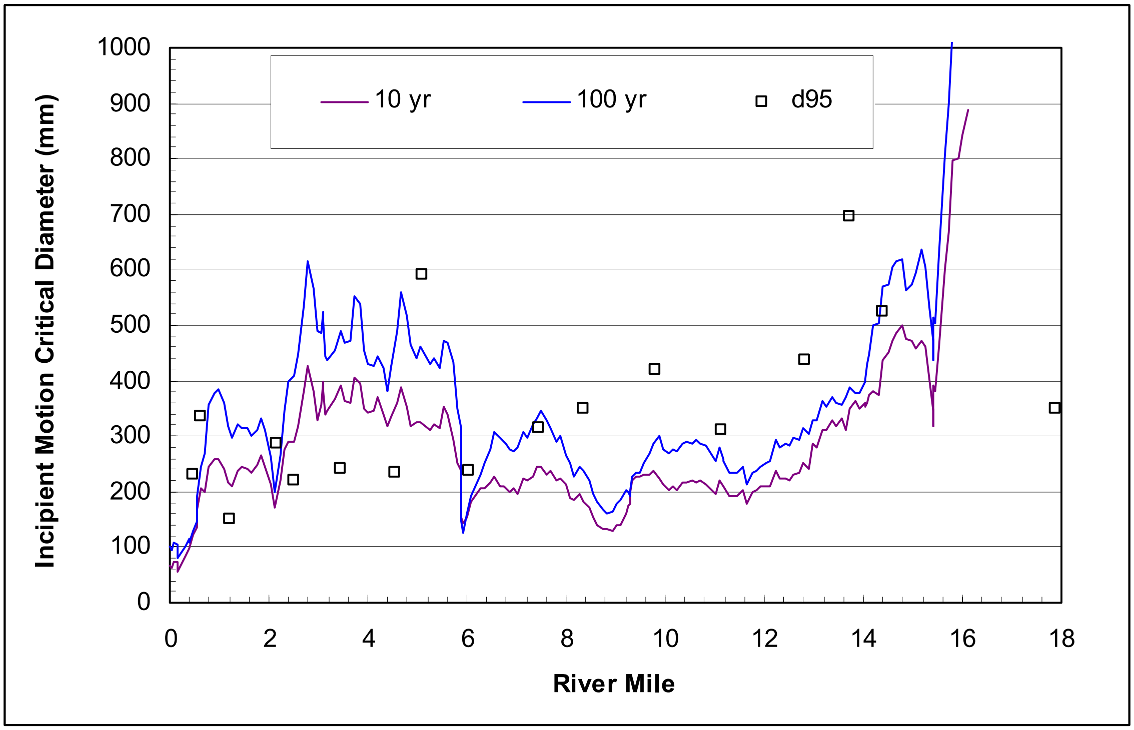


Figure 5.16. Incipient Motion Critical Diameter for 10- and 100-yr Floods, Plotted with d_{95}.

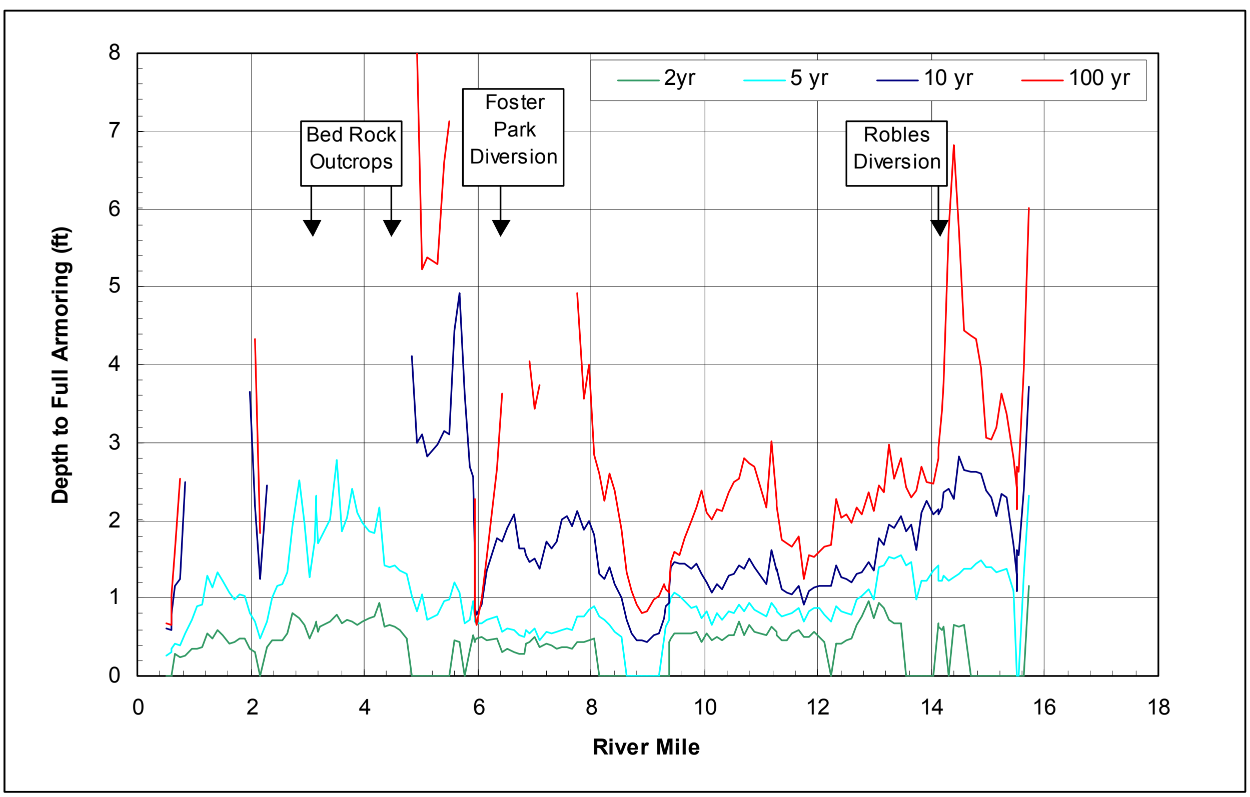


Figure 5.17. Estimated Depth to Full Armoring.

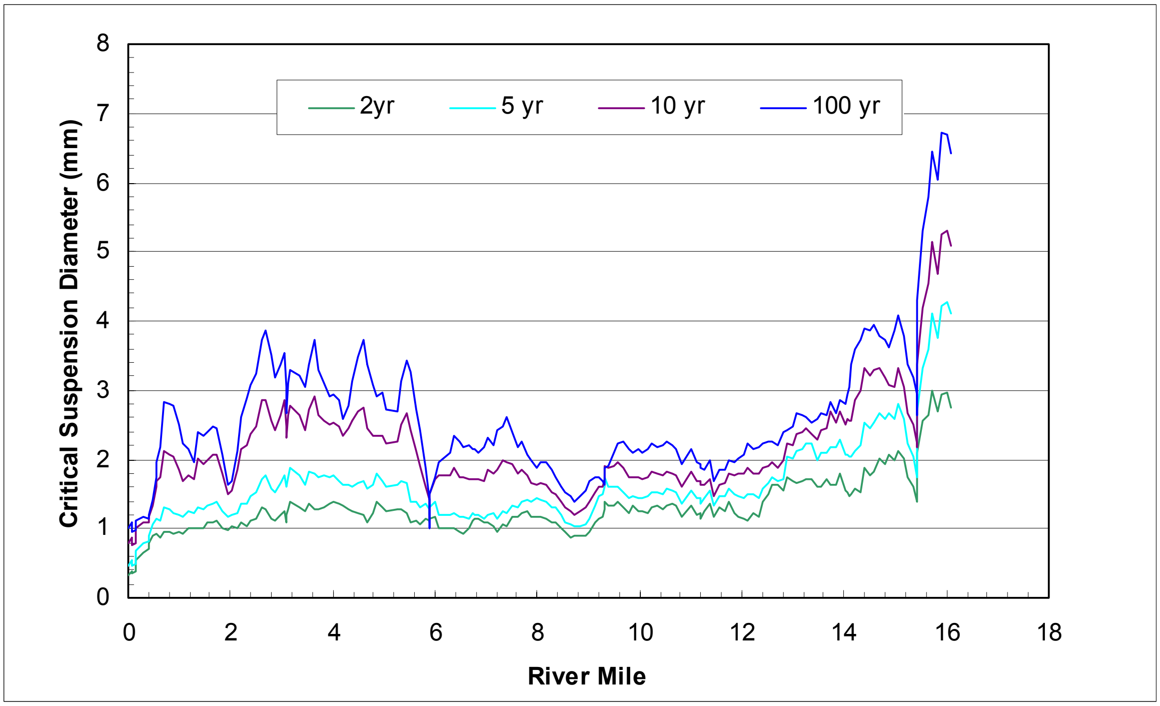


Figure 5.18. Critical suspended diameter along Ventura River for selected floods.

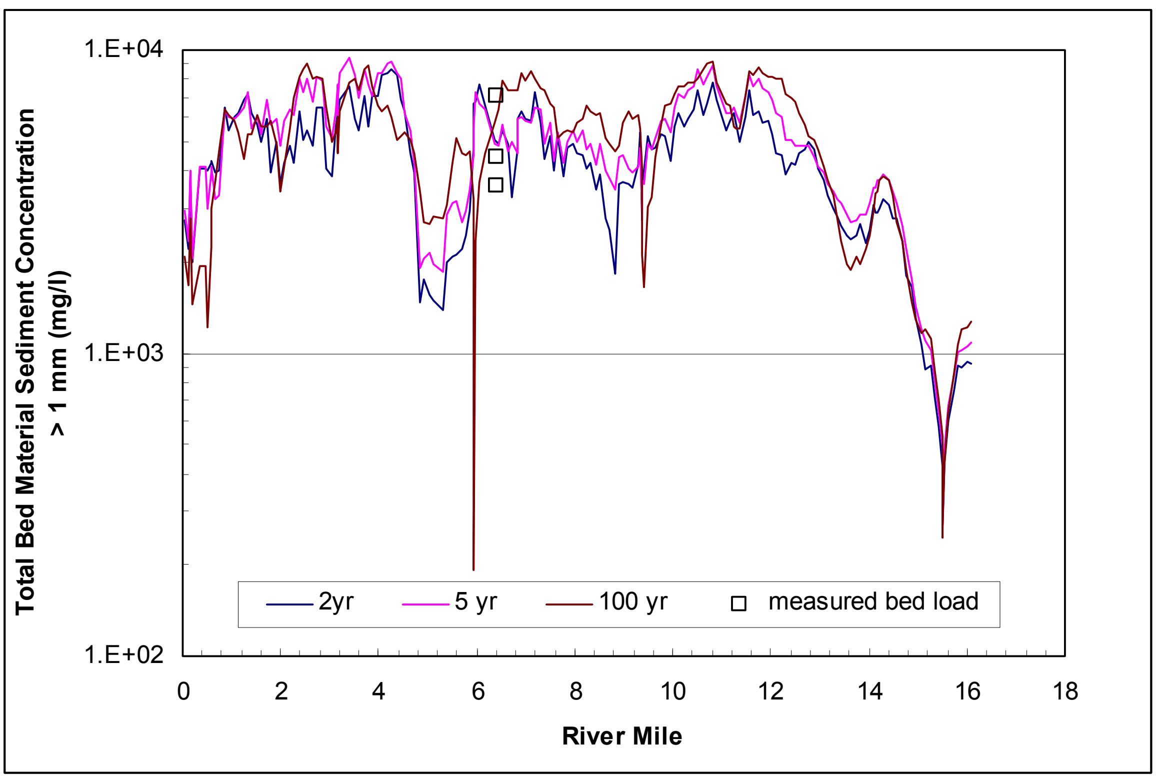


Figure 5.19. Bed material sediment capacity concentration of sediment sizes greater than 1 mm, for the Ventura River using Meyer-Peter-Müller sediment transport equation.

5.7. River Morphology

5.7.1. SUMMARY OF CURRENT VENTURA RIVER GEOMORPHOLOGY

Table 1.1 contains the project reaches that have been defined. These major reaches were further subdivided based on geomorphic analyses. Several criteria were used to subdivide the previously defined reaches included bedrock or other geologic control, overall channel morphology, the presence of alluvial terraces within the reach, and position of large tributary drainages, which represent significant increases in the overall basin area on the Ventura River.

From the confluence of Matilija Creek with the North Fork of Matilija Creek, the course of the Ventura River flows north to south in a direct path to the Pacific Ocean. Based on the overall character of the river channel, the river upstream of the confluence appears to be largely controlled by bedrock. Morphologically, this is supported by the width of the channel, its sinuous character, and the large boulders present in the channel through the entire reach.

Downstream of the confluence with the North Fork of Matilija Creek, Matilija Creek becomes the Ventura River. In this reach (the North Fork Matilija Creek to Kennedy Canyon reach; see Table 5.21), the narrow, sinuous character of the channel widens into a linear valley flanked by alluvial fans and low river terraces. The gradient of the alluvial fans (tributary canyons) remain steep relative to that of the Ventura River. The distal margins of the alluvial fans have been truncated by the river forming steep banks along both sides of the channel. The downstream end of this reach is also controlled by bedrock (Coldwater Formation; Dibblee, 1987) in the channel.

Bedrock control, that is locations where bedrock in the channel bed forms natural grade control, are reported downstream of Foster Park (Putnum, 1942, p. 728). Based on aerial photography interpretation, it appears that addition sites where the gradient or channel position of the Ventura River is controlled by rock are 1) near the mouth of Kennedy Canyon (about 0.50 miles upstream of Robles Diversion Dam), 2) near the confluence of San Antonio Creek with the Ventura River, and 3) much of the length of Reach 3B.

At the Kennedy Canyon site, the Cozy Dell Formation forms the ridge immediately to the west of the river. Steeply dipping bedrock crops out in the right channel bank near the confluence of Kennedy Canyon with the Ventura River. The influence of bedrock on the morphology of the river channel at this location is also displayed by the marked narrowing of the channel. Upstream of Kennedy Canyon, the river flows in a single, relatively straight channel that is flanked by high alluvial terraces. Downstream of Kennedy Canyon the valley and river channel widen dramatically, from about 400 feet to more than 2400 feet near the mouth of Rice Canyon.

At the San Antonio Creek site, the bedrock in the channel is somewhat hidden on the photography as at the Kennedy Canyon site as it is masked by thin alluvial deposits. However, at both sites the overall width of the valley and the channel pattern immediately upstream of the confluence are similar as the channel narrows dramatically relative to the channel further upstream and downstream.

Live Oak Acres is constructed on flood plain deposits that are believed to be between 100 and 500 years old. Rockwell and others (1984; p.1470) previously mapped these as Q2 deposits (<250 years old).

From San Antonio Creek until the estuary, the river is relatively more confined and has fewer channels. The river enters the estuary at approximately RM 0.6. The estuary is a sometimes protected from tidal action by a sand bar. The sand bar is removed when high flows pass through the estuary and then is created again by the supply of sand from littoral transport (Wetlands Research Associates, 1992).

Table 5.21. Geomorphic Descriptions of Reaches of Matilija Creek and Ventura River. The reach numbers correspond to those found in Table 1.1 and Figure 1.1.

Reach #	Land Marks	River Miles	General Geomorphic Characteristics
7a	Matilija Dam and reservoir	16.8 - 16.47	Reach covered by Matilija Dam and reservoir.
6b	Matilija Dam – North Fork Matilija Creek	16.47 - 16.0	Narrow, steep and sinuous bedrock controlled canyon reach; channel characterized by very coarse bedload and a single very narrow (<300 feet) alluvial terrace (e.g., Matilija Hot Springs).
6b	North Fork Matilija Creek – Kennedy Canyon	16.0 - 15.0	Narrow canyon reach opens into narrow linear valley; alluvial fans and low alluvial terraces flank channel; distal margin of alluvial fan deposits truncated by the river; lower end of the reach is controlled by bedrock (Coldwater Formation).
6a	Kennedy Canyon – Robles Dam	15.0 - 14.15	The average valley and river channel widen (400’ to more than 1650’) and the channel slope (0.020 to 0.013) changes significantly relative to the upstream reach.
5	Robles Dam – Meiners Oaks	14.15 - 12.3	Similar characteristics to upstream reach with exception that the valley continues to widen to roughly 2-3 times width of reach 5A. River channel takes on braided pattern. The downstream end of the reach constricted between bedrock and older alluvial terrace; controlled by geologic structure (Arroyo Parida-Santa Ana fault).
4/5	Meiners Oaks – Santa Ana Blvd.	12.3 – 9.5	Channel again widens into alluvial valley flanked by high terraces. The channel retains braided character but narrows slightly near Live Oak Acres. Natural constriction created by Devils Gulch and Oak View faults. The Live Oak Acres levee that flanks the channel for almost a mile to the bridge at Santa Ana Blvd.
4	Santa Ana Blvd. – San Antonio Creek	9.5 – 7.93	Similar characteristics to upstream reach; wide alluvial valley flanked by high alluvial terraces. Channel pattern begins to shift from braided to multi-tread with vegetated bars. Downstream end of the reach is controlled by bedrock and geologic structure near the confluence of San Antonio Creek (Ayers Creek syncline).
3	San Antonio Creek – Foster	7.93 - 6.1	River channel and valley narrow slightly from upstream reaches. Large portion of the reach is flanked by the

Reach #	Land Marks	River Miles	General Geomorphic Characteristics
	Park		Casitas Springs levee. Downstream end of the reach is controlled by bedrock and geologic structure (Cañada Larga syncline).
2	Foster Park – Shell Road	6.1 - 3.0	Narrow canyon reach opens into wide valley flanked by broad flat alluvial terraces. River channel width remains narrow and becomes deeply incised in alluvium in the lower portion of the reach. Bedrock is exposed in the channel bank at several locations in the upper part of the reach (northern flank of the Ventura Avenue Anticline).
2	Shell Road - Estuary	3.0 - 0.6	Similar characteristics to Reach 3B with exception that valley and active channel continue to widen in a downstream direction and no bedrock was observed in the reach.
1	Mouth of the Ventura River/Estuary	0.6 - 0.0	Morphology of the reach formed primarily in response to large floods, tidal influence, and coastal processes. Affected by channelization and three bridge crossings.

5.7.2. HISTORICAL MORPHOLOGY OF THE VENTURA RIVER

Cross Section Analysis

Channel cross sections were generated for the study reach from a digital terrain model created from 2001 aerial photography. There is a high level of confidence associated with the 2001 topographic data. The 2001 aerial photography was flown at a low elevation (1:6000) when the river was relatively dry so the majority of ground was exposed. The maximum possible elevation error for the 2001 data is +/- 1 foot, but in most areas is estimated to be much less. The only other set of channel survey data available throughout the study reach is from 1970. The cross section data was generated using 2-foot contour data created from January 1970 aerial photographs using photogrammetric methods and has a lower level of confidence than the 2001 data. The 1970 contour data is noted as having a maximum potential error of +/- 2.5 feet (USCOE, 1971). The original coordinates of the 1970 data were not found so their locations had to be determined from plan view drawings in the 1971 flood report. Based on these drawings, the 1970 sections are generally within a few tens of feet in longitudinal distance from the 2001 cross section locations they are compared to in this report. The 1970 data represents the river channel 23 years after the completion of Matilija Dam and about one year following the large flood events that occurred in January and February of 1969.

Having an additional set of cross section data that represents the pre-dam river channel downstream of Matilija Dam would be very useful to evaluate further the impacts of Matilija Dam. A new photogrammetric technology is being explored that allows generation of historical topography using placement of high quality control points from recent aerial photography onto historical aerial photography. If this process could be completed with a high level of confidence, the data would be very useful for this project. A set of historical aerial photography from 1947 and 1970 was chosen to try to generate historical topography using the new 2001 topographic data. The 1947 data would represent pre-dam channel topography. The 1970 data should serve as verification of the data results because it should match well with the 1970 cross section data generated using contour data created directly from the 1970 aerial photography (USCOE, 1971).

Several approaches were tried to accomplish this process, but the most promising seems to be using the 2001 control data to generate the 1970 topography, and then using the generated 1970 topography to create new control points and generate the 1947 topography. While significant improvements were made during this effort to regenerate the historical data, a high level of confidence could not be assigned to the generated data because the new 1970 data did not match well with the 1970 survey data generated from the 2-foot contours (Figure 5.20). Additionally, areas such as terraces that should be stable in elevation did not match well to the 2001 cross section elevations. A research report is being done by the GIS department at Reclamation to document this effort. It was noted that a major obstacle in the process was finding enough control points (areas of identical topography that have not been altered over time) that matched between the 1947, 1970, and 2001 aerial photographs. The 1947 and 1970 aerial photographs were flown at a higher elevation (relative to the ground) than the 2001 aerial photographs that can also introduce error. The creation of the historical topography could possibly be improved in future research efforts by using additional control from other references or trying to find other candidates for control points in the existing images. Based on the above discussion, the 1947 topographic data could not reliably be used at this time. However, the 1947 aerial photography were used to measure geomorphic features that can be readily identified in plan view.

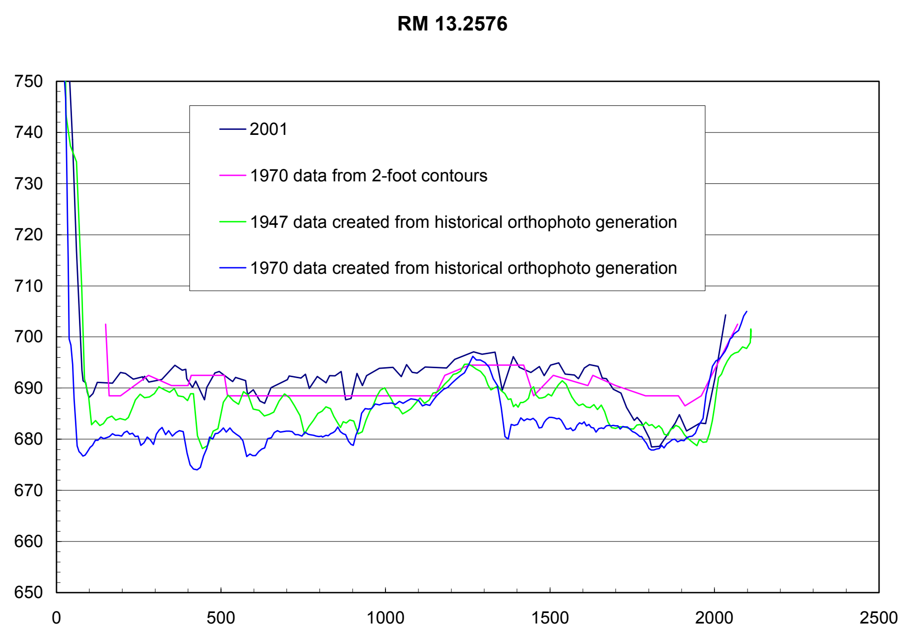


Figure 5.20. Comparison of historical cross section data generated from two different photogrammetric methods (Reach 5, RM 13.2576).

Cross section data was also collected at the USGS gaging station at Foster Park Bridge at RM # 6. A comparison of these cross section measurements shows the changes in the channel bed because of the 1958 flood (Figure 5.21). The figure shows that the river is relatively dynamic during a flood and the riverbed elevations can change several feet during a single event.

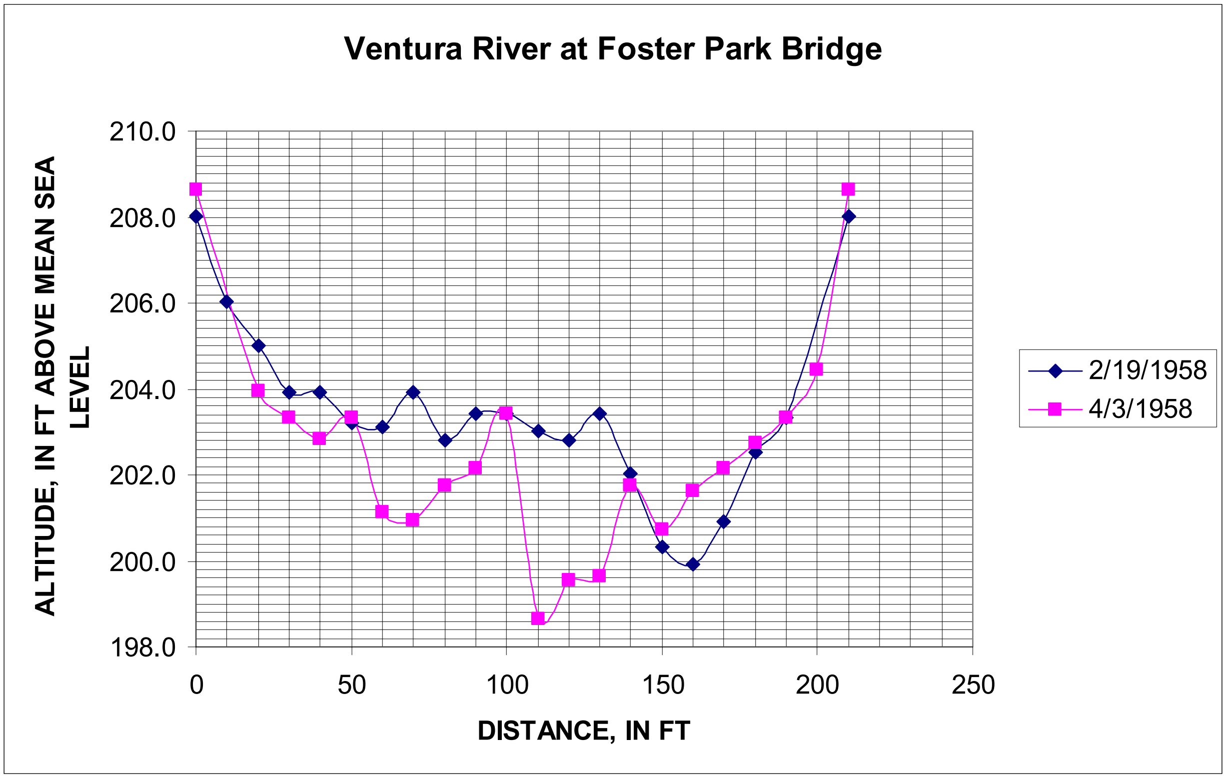


Figure 5.21. Change of Cross Section at Foster Park Bridge due to 1958 flood.

There is some uncertainty in the location and elevation accuracy (+/-2.5 feet) of the 1970 cross section data and the riverbed elevation naturally fluctuates within a range of a few d_{50} particle diameters as seen in the USGS gaging station location. Therefore, any changes between the 1970 and 2001 thalweg elevations within a range of +/- 2.5 feet may only be a reflection of short-duration channel dynamics and error within the data, particularly if it is only at one location. Changes beyond 2.5 feet over a group of cross sections would more likely indicate long-term changes in the channel bed. A 3-point moving average of the change in channel bed elevation between 2001 and 1970 was computed. A thalweg value was used rather than an average channel bed elevation because the Ventura River is wide and often has multiple bars between channels that would make it difficult to compute the average channel bed. Based on the comparison, the Ventura River has experienced significant erosion since 1970 at three locations (Figure 5.22). The first two locations are in the upstream portions of Reach 6a immediately downstream from Matilija Dam and in Reach 5 immediately downstream of Robles Diversion (RM 13 – 14). Reaches 3, 4, and the downstream half of Reach 5 appear to fluctuate and be dynamic in channel bed elevation, but no consistent trend of aggradation or degradation beyond the 2.5 feet criteria can be seen from the thalweg comparison. At RM 6.5, upstream of Foster Park Diversion, the channel has remained relatively stable because of the concrete diversion structure located in the river and because of natural channel controls. Reach 2, however, has had the largest channel changes and widespread degradation since 1970.

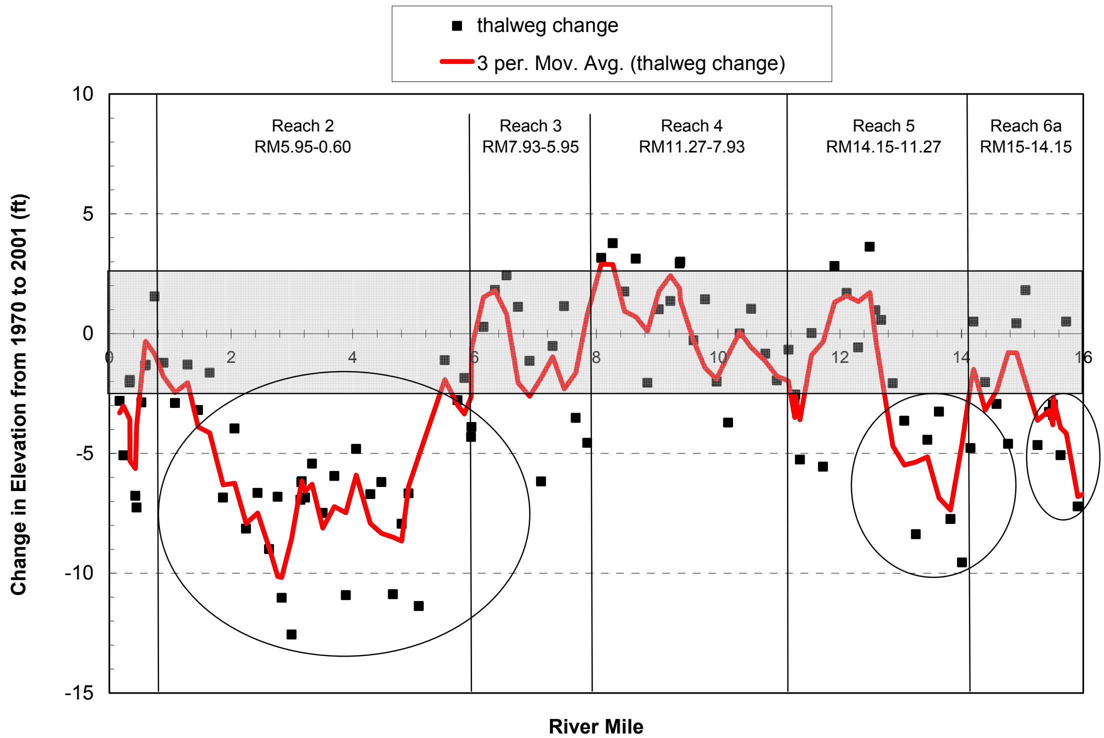


Figure 5.22. Comparison of change in thalweg elevation between 2001 and 1970. Negative changes indicate areas of degradation in the channel bed. Positive changes indicate areas that have aggraded. Areas within 2.5 feet of change are considered to be within the error range of the 1970 data.

As a check on the thalweg comparison, the computed 100-year floodwater surface elevations were also compared based on the 2001 and 1970 cross section data. The limitation on this comparison is that the 1970 data has less detail than the 2001 data and does not contain any of the existing bridges that often cause noticeable backwater during floods in the 2001 data, particularly from the bridges at RM 9.4 and RM 15.8. However, the general comparison is consistent with the thalweg comparison showing a drop in flood stage at the three areas that have experience degradation (Figure 5.23).

Plots of the cross sections in 1970 and 2001 are given in Figure 5.28. Based on the cross section comparison, the channel has become more entrenched in the reach downstream of Robles Diversion from RM 14 to 13. However, from RM 13 downstream to RM 9, the river has remained relatively stable in the past 30 years. The channel is active, but the average bed elevation and the channel properties have been maintained.

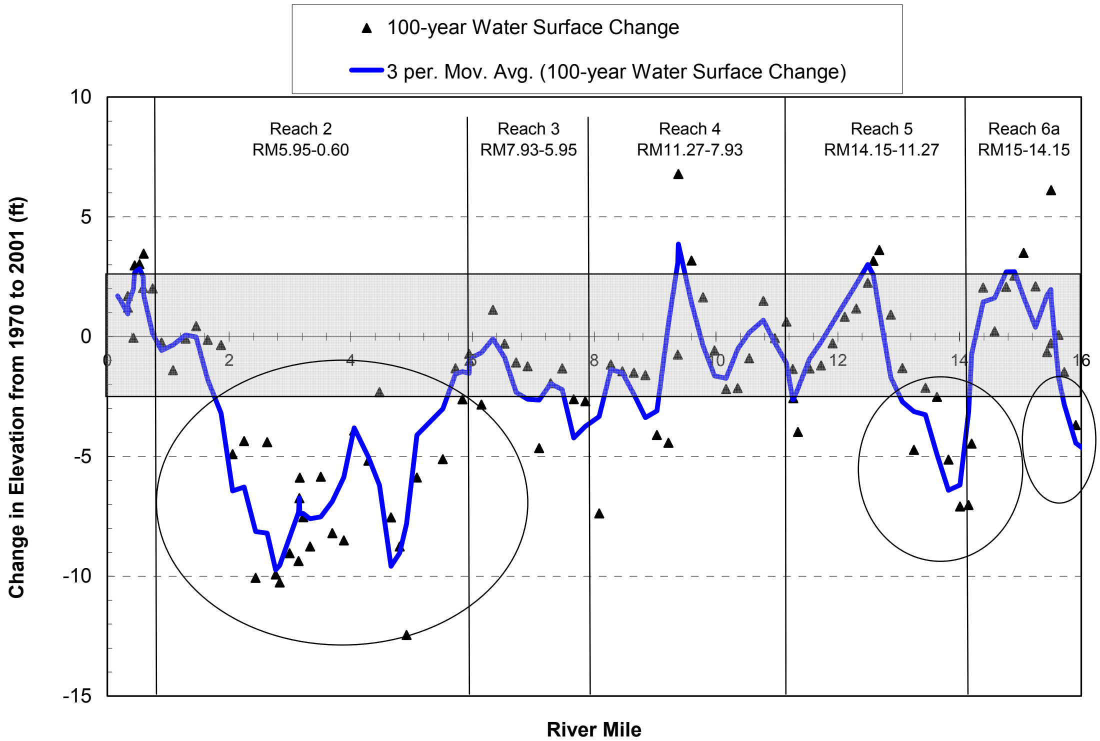


Figure 5.23. Comparison of change in 100-year flood stage between 2001 and 1970. Negative changes indicate areas where the flood stage has lowered. Positive changes indicate areas where the flood stage has increased. Areas within 2.5 feet of change are considered to be within the error range of the 1970 data.

In addition to the 1970 cross sections described above, there were repeat cross section surveys performed at Shell Road Bridge from 1975 to 1994 (Figure 5.29). There was almost 10 feet of erosion and the cross section has narrowed and become deeper. The trend of narrowing and deepening may not continue, however, because since 1994 the bed elevation has shown slight aggradation. In 1994, the bed elevation was 97.5 ft NAD88 and the current bed elevation is 99.8 ft.

There were also historical cross sections surveyed on 9-23-1993 just on the upstream side of the bridge of the Baldwin Road Bridge (RM11.27). The thalweg elevation for this cross section in 1993 was 520.2 ft. The thalweg elevation from the 1971 survey was 530.0 ft and from the 2001 survey was 521.8 ft. These surveys suggest that there was significant erosion from 1971 to 1993 and since then this reach has been relatively stable in terms of thalweg elevation.

River Plan Form Analysis Using Aerial Photography

The morphology of the Ventura River was analyzed using available survey information and aerial photography. The primary sources for historical aerial photographs of the Ventura River and Matilija Creek were the Ventura County Flood Control District, the Ventura County Mapping Department, the U.S. Forest Service, and the U.S. Geological Survey. A list of the relevant photographs found at these agencies was compiled along with information on type of photograph, date, scale, and coverage (Exhibit M. Table 26.1). Additional sources, researched but not used in this study, include private companies, and the University of California at Santa Barbara, at Berkeley, and at Los Angeles.

A set of criteria were developed for prioritization of the sets of photographs to be considered for analysis of channel changes resulting from historical floods. In addition to the September 13, 1947 set representing "pre-dam" conditions and the September 9, 2001 set representing present conditions, top priority was assigned to photo sets taken soon after the five largest post-dam flow events (January-February 1969 counted as one). Other criteria included completeness of coverage of the full length of the Ventura River, and the size of any flows between the major event and the date of the photos. If a flood had two large peaks (for example, 1969), photos taken after the second peak were given higher priority.

Photograph sets taken on three dates were selected for this phase of the study: September 13, 1947; January 30, 1970; and, September 9, 2001. In preparation for inclusion in a Geographic Information System (GIS) and subsequent analysis, the photos were scanned, orthorectified, and combined into a mosaic. Each set of combined photos was brought into the GIS as a layer and was projected using a single coordinate system (State Plane, Zone 5, NAD83).

The GIS was used to describe, measure, and analyze basic channel geomorphology at each of the section locations for each of the three selected post-flood dates. The information collected at each section included widths of the active channel, the bank-to-bank channel, and the individual active-channel segments; and descriptions of channel form and right- and left-bank material and vegetation.

The Historical Aerial Photograph GIS was constructed by first importing the 1947, 1970, and 2001 sets of combined photos into a GIS map using ESRI ArcGIS. Channel cross-section locations were previously chosen using previous FEMA maps and topographic information developed from the 2001 photos. The cross sections are specified in river miles (miles upstream from the mouth of the Ventura River, measured along the 2001 thalweg). Beginning at the estuary, the sections are labeled in an upstream direction A through Z, then Aa through Zz, Aaa through Zzz, and Aaaa through Yyyy.

For each set of photos, beginning with the 2001 set, a GIS layer was made consisting of active-channel section lines each drawn approximately perpendicular to the channel banks and at the previously chosen measured distance upstream from the river mouth (river-mile distance of known landmarks, such as bridges and tributary streams, also was used as a reference). The section lines for the 1947 and 1970 layers were drawn at the same locations as the 2001 sections. At some locations, this resulted in section lines that were not quite perpendicular to the 1947 or 1970 active channels, thereby causing channel widths measured along the oblique section line to be somewhat greater than actual. At a few locations, where the 1947 or 1970 active channel had a significantly different orientation than the 2001 active channel, the 1947 or 1970 section lines were drawn perpendicular to the banks rather than parallel to the 2001 section.

Each section has one or more segments, depending on the number of separate active channels along the section. If there is more than one segment, the segments are numbered, starting from the right bank (for example, Ccc1 and Ccc2). Each segment follows the straight section line (no breaks in section).

The primary guide used to determine the active-channel boundaries at each section was the amount and density of vegetation, or lack thereof, and the characteristics of the vegetation where present. Channel areas without vegetation were considered likely to be active. Areas with dense vegetation were considered not part of the active channel, even though in some locations, especially in the reaches downstream from Casitas Road Bridge, dense vegetation may be hiding part of the active channel, including the actual location of the banks. Most of the dense vegetation appears to be riparian (trees and tall shrubs), as opposed to lower, drier-appearing, brush. In these same reaches, low, dense, bright, or dark green vegetation that appeared to be growing in a narrow, incised central low-flow channel was considered part of the active channel. In general, it was assumed that the floods remove channel vegetation; however, in the lower reaches of the Ventura River, high water may have flowed under and through dense, established riparian vegetation without removing it.

The definition used for ‘active channel’ was those parts of the Ventura River channel that most likely experienced flow in the last large flow before the date of the set of photographs. None of the three set of photos were taken immediately after a flood. The September 2001 aerial photographs reflect the relict active-channel morphology from the most recent major flow, 1998 (38,800 cfs peak average daily flow), as well as modifications resulting from subsequent smaller flows between 1998 and 2001. Similarly, the 1947 photos probably show the effects on the active-channel morphology of a combination of the 1943, 1944, and 1945 medium-large peak flows (35,000 cfs, 20,000 cfs, and 17,000 cfs), as well as possibly some remnant effects of the 1939 flood (39,200 cfs). The 1970 photos were taken approximately one year after the January-February 1969 floods, though the peak average daily flow in the intervening period was only about 100 cfs. However, by the time the 1970 photos were taken, some of the channel had been scraped and modified using bulldozers; the assumption was made that the modifications did not extend beyond the active-channel boundaries.

Several factors cause difficulty in mapping active channels using aerial photographs taken varying amounts of time after a flood. First, the characteristics of an ‘active channel’, as represented on aerial photographs, are not easy to define, other than clues presented by vegetation or lack thereof. Second, the rate at which vegetation grows back after a sizable flow was not precisely known. Third, not all parts of a channel that experience flow are necessarily stripped of vegetation. Fourth, at some locations, three or more ‘ages’ of channel or bar surfaces (based primarily on density and relative maturity of brush or shrubs) are visible on the aerial photos. These factors combine to create difficulty in determining whether areas of light to moderate riparian shrub growth, especially on channel bars, experienced flow during the last flood (though possibly not during subsequent smaller flows), and therefore are considered part of the active channel. (Generally, the location of the most recent moderate-to-low-flow active channel is the easier to interpret using the aerial photographs.) Because the process of identifying the historical active channel using aerial photographs is interpretive, a confidence rating was assigned for each section and segment and was entered in the GIS attributes tables, as well as remarks on possible alternative interpretations.

Discussion of Plan Form Changes

A plot of the active channel widths in 1947, 1970, and 2001 is shown in Figure 5.24. The most striking conclusion from the graph is the similarity of 1947 and 2001, and the large widths in 1970. The major cause of the large widths in 1970 was the extreme nature of the 1969 flood. The flood peak and duration was large enough to remove large amounts of vegetation from the flood plains and to rework the channel significantly. After the large floods, the channel gradually returns to narrow width and vegetation grows on the flood plain. An example of the channel changes in shown in Figure 5.25. This reach is immediate below Robles Diversion, where some of the largest changes in width have occurred. The large widths in 1970 are easy to identify.

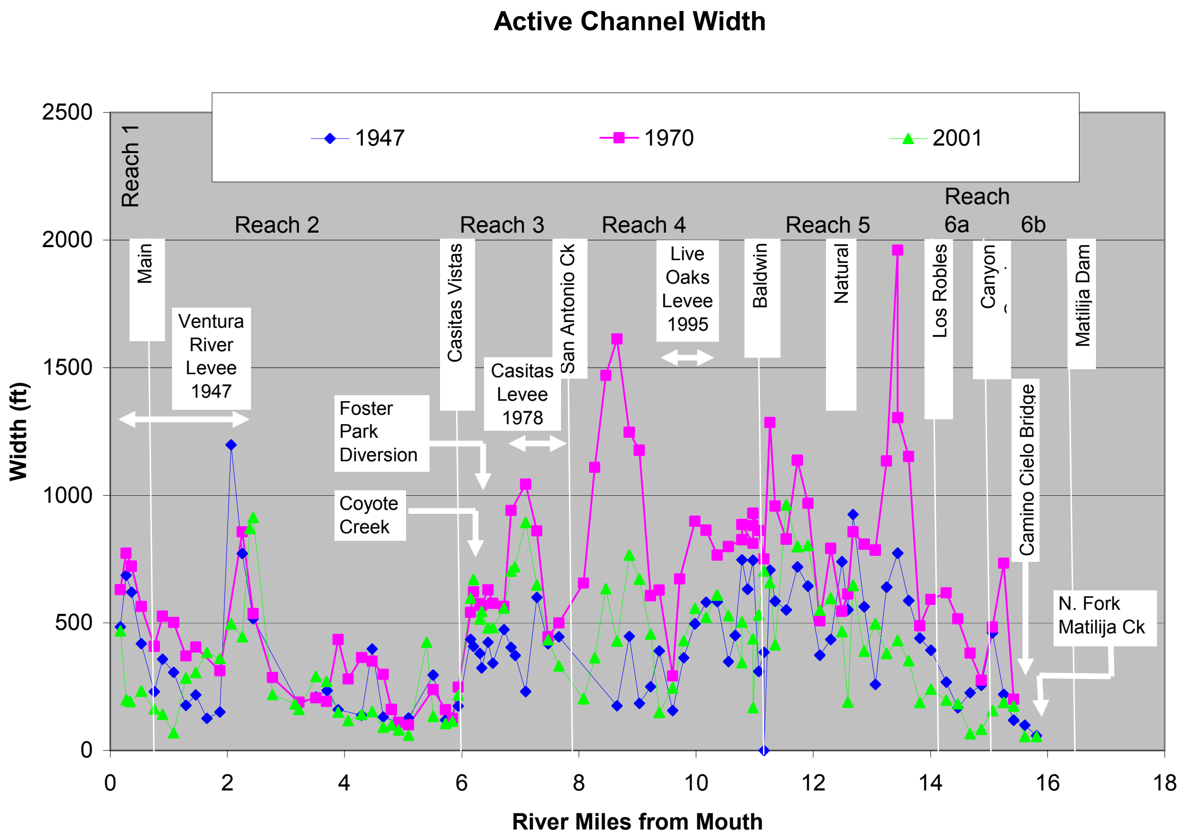


Figure 5.24. Historical Active Channel Widths of the Ventura River in 1947, 1970, and 2001.

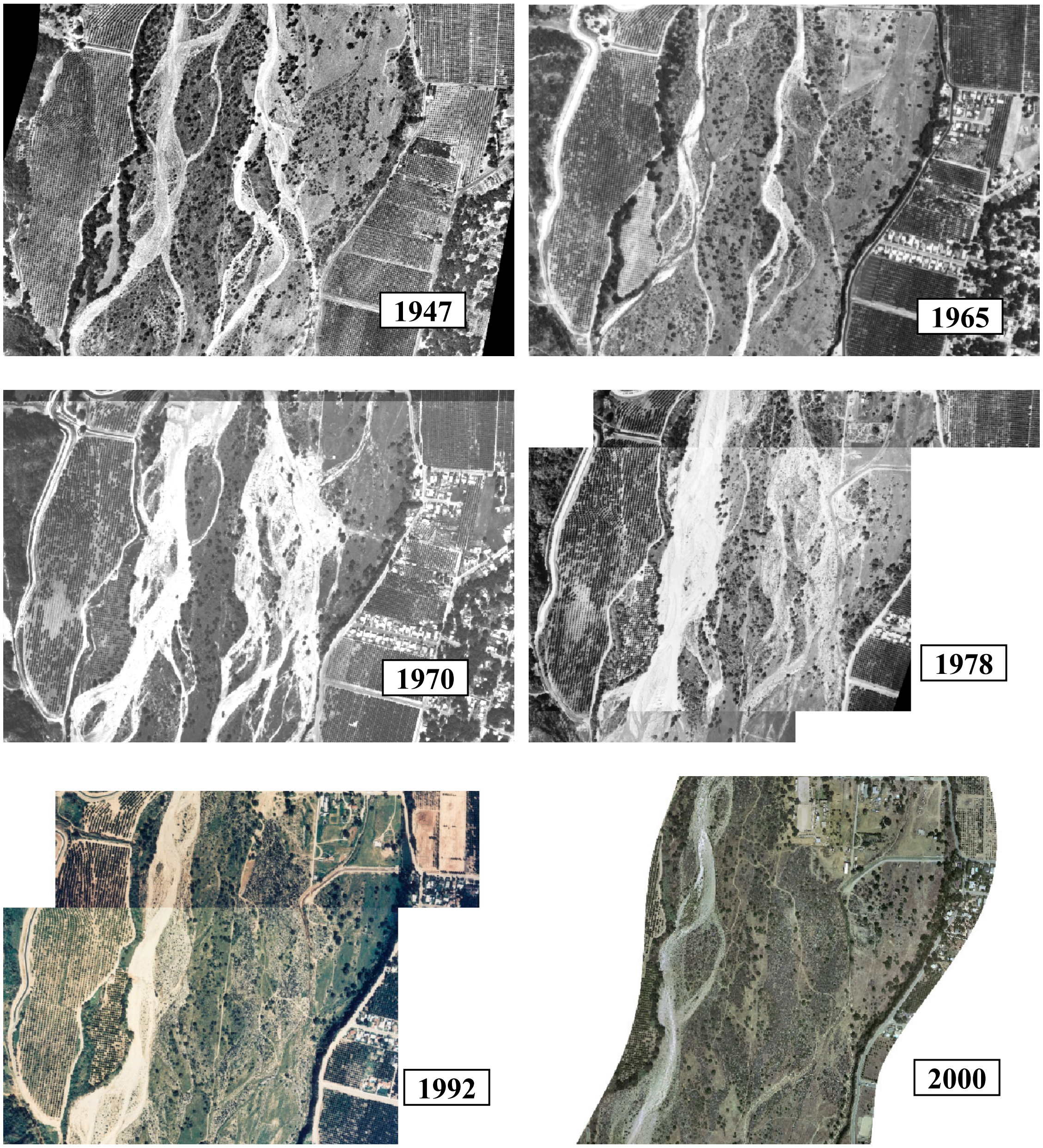


Figure 5.25. Historical Aerial Photograph Comparison at RM 13.5 Downstream of Robles Diversion.

Discussion of Sediment Supply and Causes of Erosion

Based on aerial photograph interpretation, it appears that the coarse sediment supply along the Ventura River is almost unlimited. In addition to the sediment yield from the basin, a tremendous amount of sediment is currently stored in flood plain and terraces along the river. Despite the general feeling that the largest proportion of the total sediment load in a river is transported by flows that are in the range of the mean annual flood (Wolman and Miller, 1954) a variety of data from the western U.S. seems to indicate that the largest proportion of sediment is actually transported by the infrequent, large magnitude floods. For example, during the 1969 flood season, the suspended sediment flux on the Ventura River was greater than the preceding 25 years (Inman and Jenkins, 1999). The record of sedimentation at Matilija Dam supports this conclusion. The total storage capacity of reservoir was reduced by about 1000 ac-ft or about 14% of its total design storage capacity during this flood year. This is about three times the volume of the preceding 22 years. It appears that the most effective mode of sediment transport on the Ventura River basin is the larger magnitude floods. This idea is also supported by comparisons of historical aerial photography that indicate dramatic changes in the river channel morphology following large magnitude floods.

Of the 10 largest floods during the period of record at the stream gaging station on the Ventura River near Ventura (USGS stream gaging station #11115500), the floods of 1969 rank number 2 and 5. In addition, 8 of the 10 largest recorded floods on the Ventura River have been since the closure of Matilija Dam in 1947 (see Table 1-4). Of these eight floods following the closure of Matilija Dam, only the flood of 1952 occurred prior to the floods of 1969.

Table 5.22. Ten Largest Floods at USGS Gage 11118500 since 1927.

Rank	Date	Flow at Foster Park
1	February 10, 1978	63,600
2	January 25, 1969	58,000
3	February 12, 1992	45,800
4	January 10, 1995	43,700
5	February 25, 1969	40,000
6	March 2, 1938	39,200
7	February 23, 1998	38,800
8	February 16, 1980	37,900
9	January 22, 1943	35,000
10	January 15, 1952	29,500

While the geologic setting primarily controls the current morphology of the river, the current climate conditions (during the last 35 years) and the associated hydrology strongly influence the movement of sediment within the river system, and thus the channel form. Based on the climate regime and the geomorphology, it is apparent that the sediment that is coarser than approximately 10 mm is transport-limited. That is, more coarse sediment is available within the drainage basin than can be transported by the Ventura River. This is largely a reflection of the physiography, in particular the semi-arid climate, nature of the bedrock, and active tectonics responsible for high uplift rates and steep slopes.

As mentioned previously, there has been degradation documented in the Ventura River since 1971 in three reaches. Causes for the degradation could be:

1. A shift from a relatively dry period to a wet period.

2. Trapping of sediment behind Matilija Dam and associated downstream degradation.

3. Trapping and removal of sediment at Robles Diversion.

4. Trapping of sediment and water behind Casitas Dam.

The hydrological record and the large sediment supply in the Ventura River floodplain supports reason 1. Because the coarse sediment sizes are transport limited, increasing the volume of water will cause degradation. The degradation of the Ventura River may be the result of the rivers increased ability to move sediment and in this particular case, the movement of sediment stored in the channel and adjacent flood plain. An analysis of the stream gaging records in the Ventura River basin suggests that the 40-year period beginning with the 1969 floods has been a relatively wet period when compared to the previous 40-year period.

The impact of Matilija Dam and Robles Diversion Dam will be most important in the reaches immediately below Matilija Dam. Because Matilija Creek provides the majority of the sediment in the reaches above San Antonio Creek, the termination of its sediment load at Matilija Dam has a larger affect in the upper reaches. Therefore, while reason 1 is probably the largest factor for the degradation of the river system as a whole, a combination of reasons 1, 2 and 3 are likely significant causes of degradation in the reaches immediately below Matilija Dam and Robles Diversion.

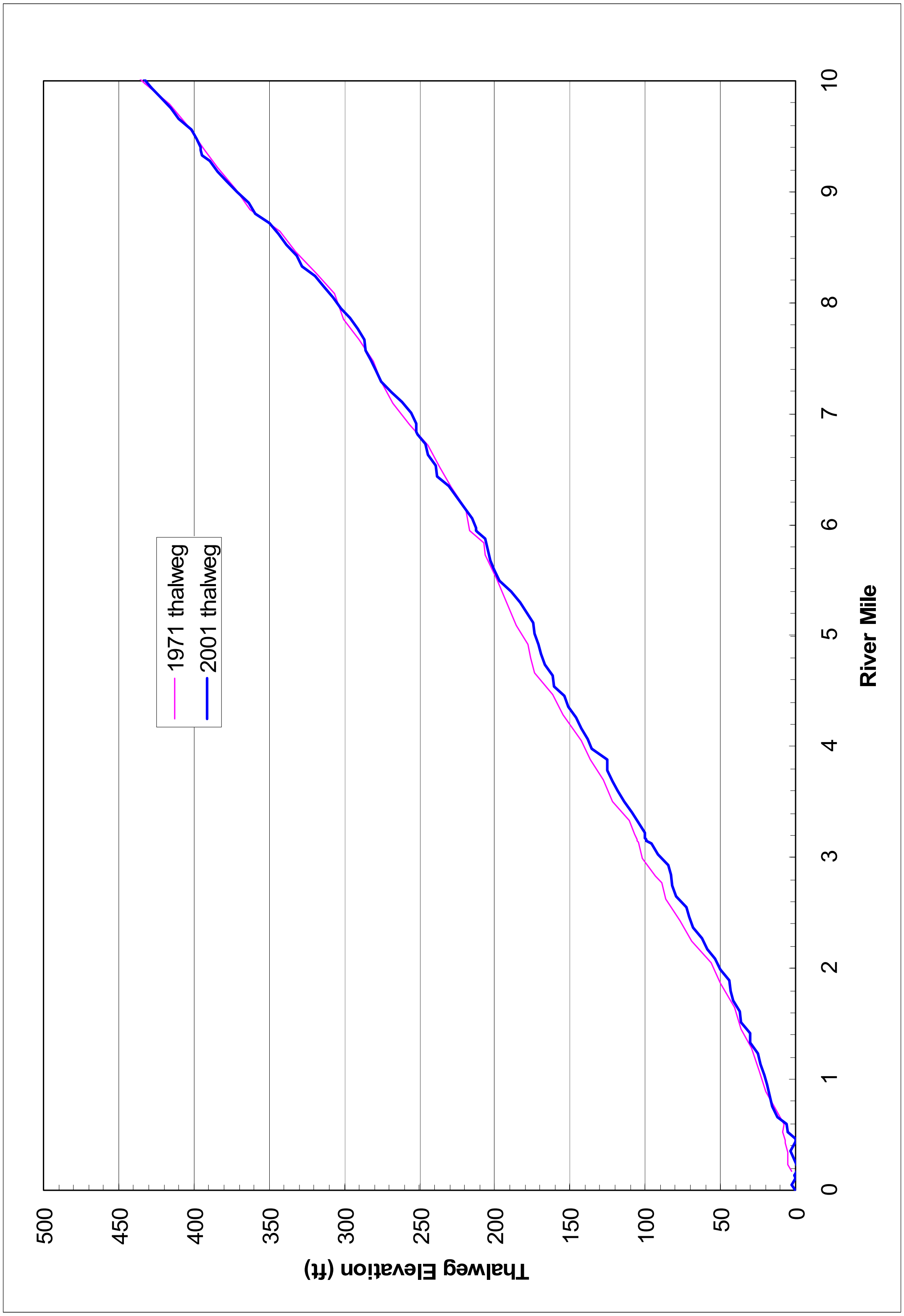


Figure 5.26. Measured thalweg profile for 1971 and 2001, RM 0 – 10.

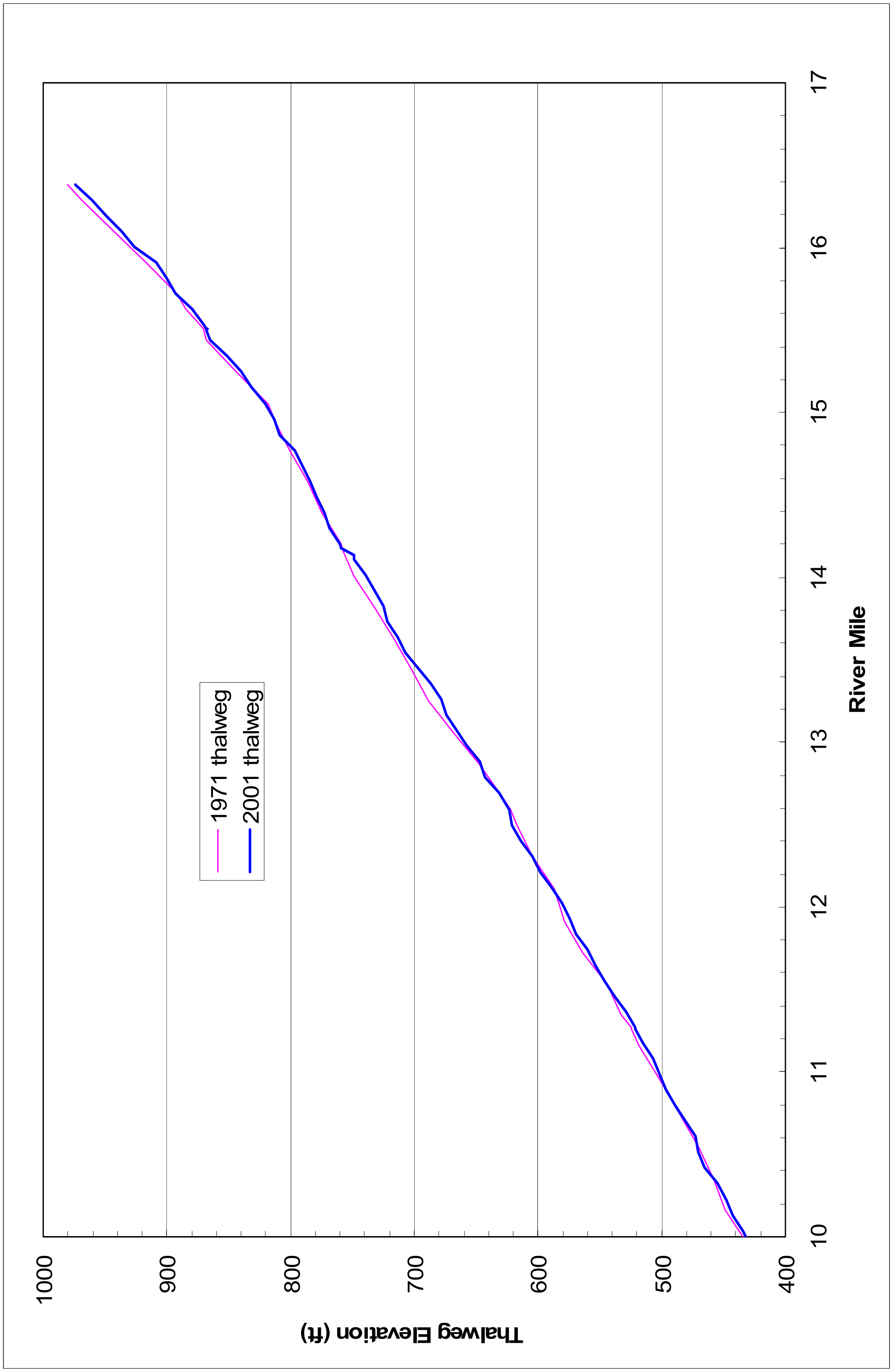
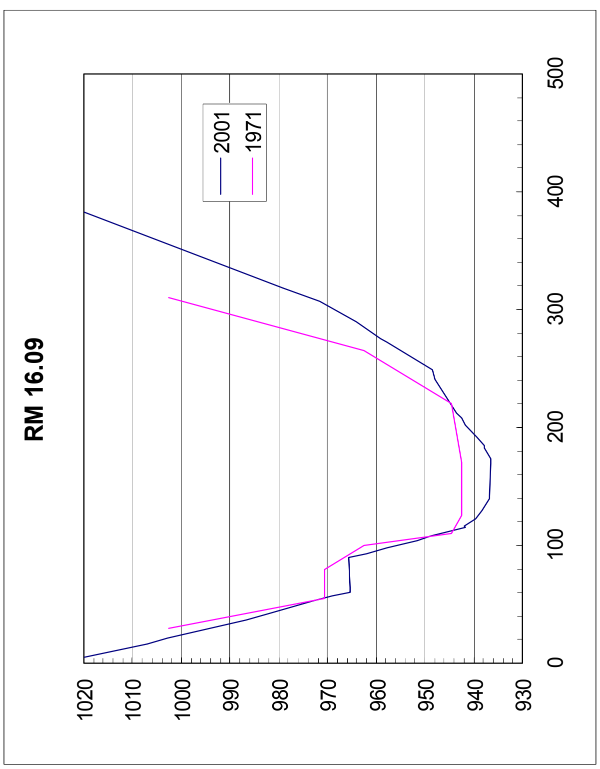
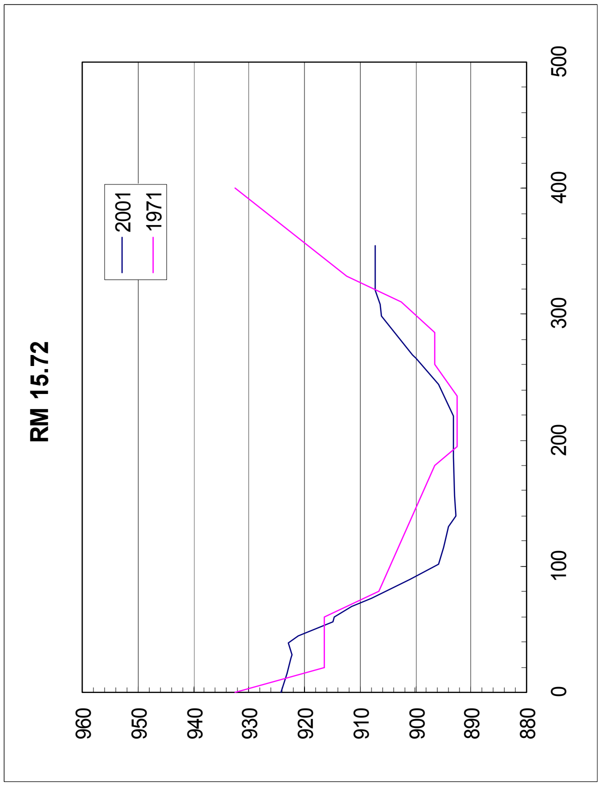
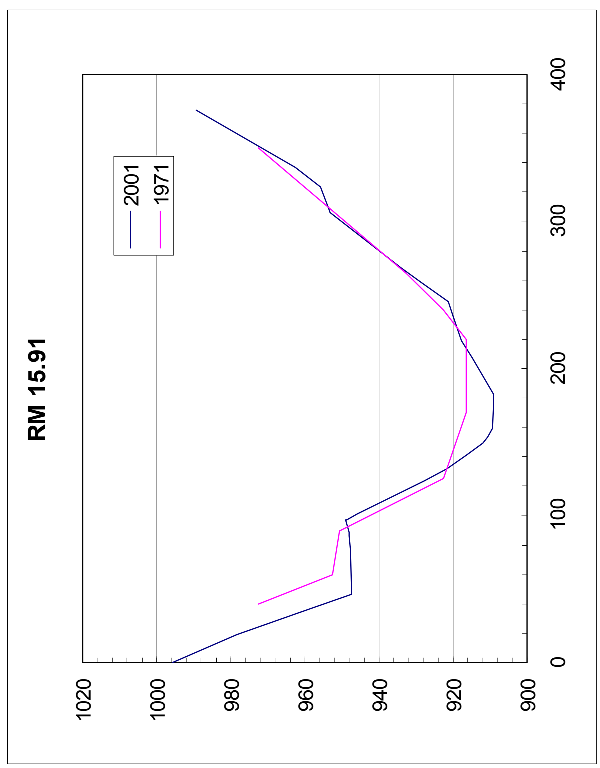
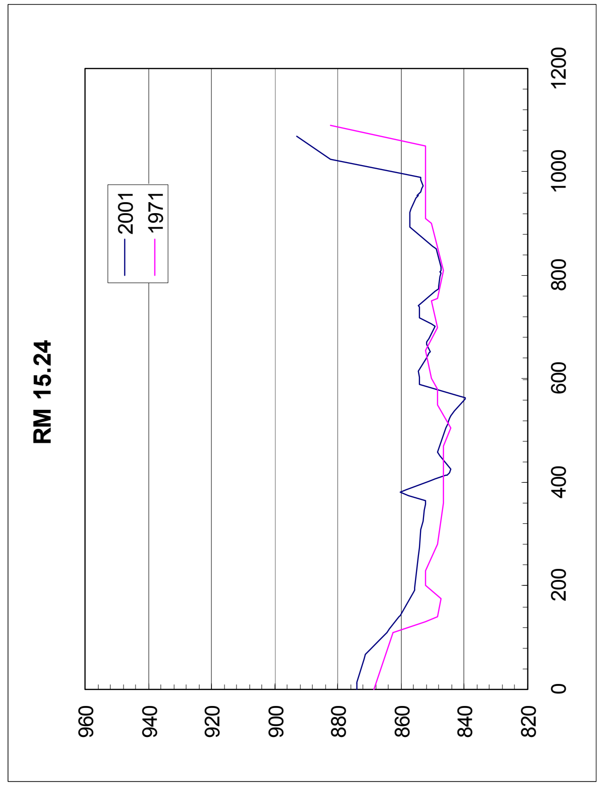


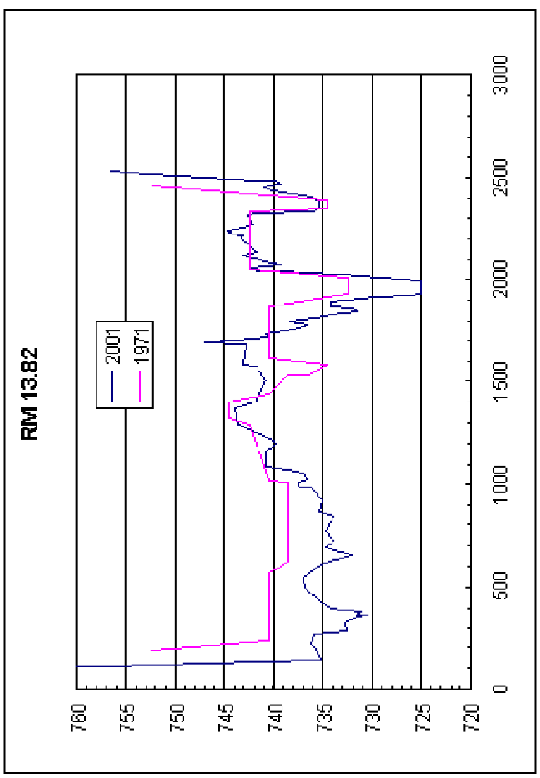
Figure 5.27. Measured thalweg profile for 1971 and 2001, RM 10 – 17.

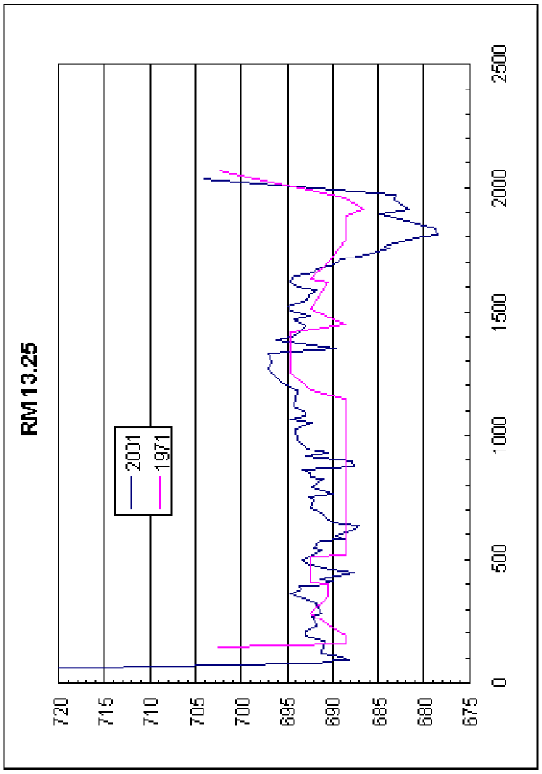

RM 16.09
2001
1971

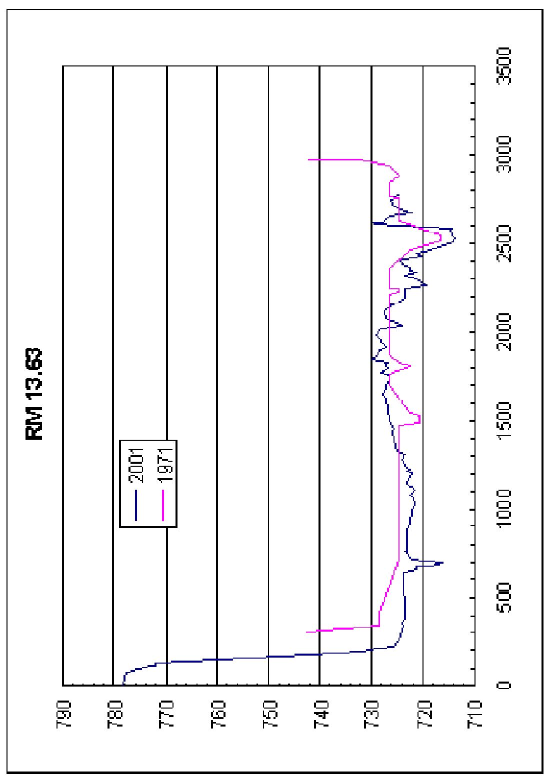

RM 15.72
2001
1971

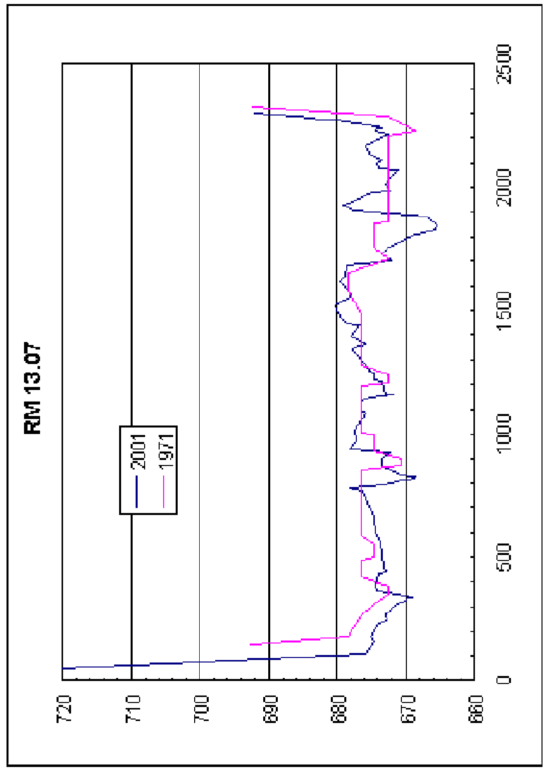

RM 15.91
2001
1971


RM 15.24
2001
1971


RM 13.82
2001
1971
760
755
750
745
740
735
730
725
720
0
500
1000
1500
2000
2500
3000


RM 13.25
2001
1971
720
715
710
705
700
695
690
685
680
675
0
500
1000
1500
2000
2500


RM 13.63
2001
1971
790
780
770
760
750
740
730
720
710
0
500
1000
1500
2000
2500
3000
3500


RM 13.07
2001
1971
720
710
700
690
680
670
660
0
500
1000
1500
2000
2500

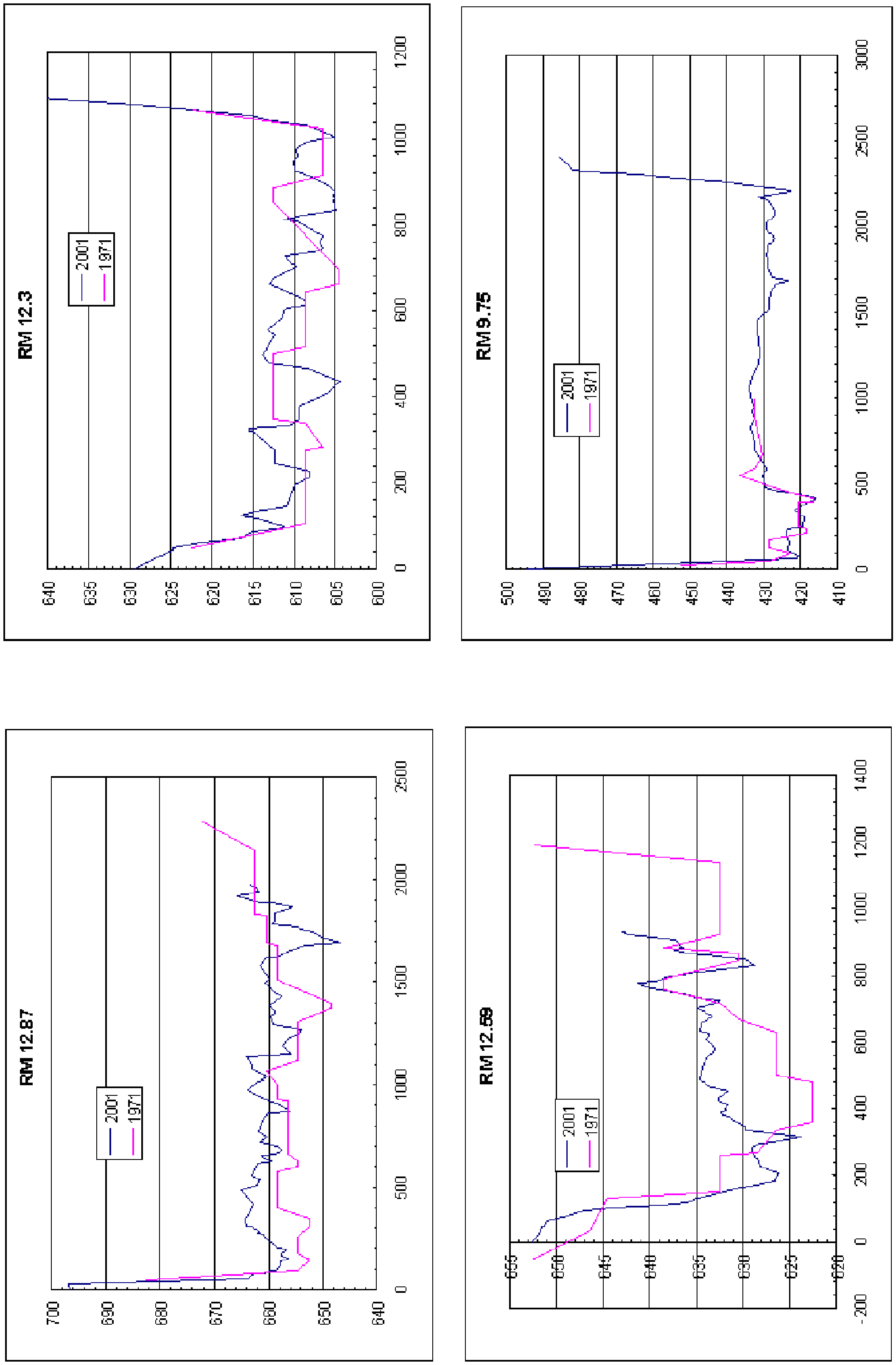


Figure 5.28. Cross section comparison between 1971 and 2001 surveys.

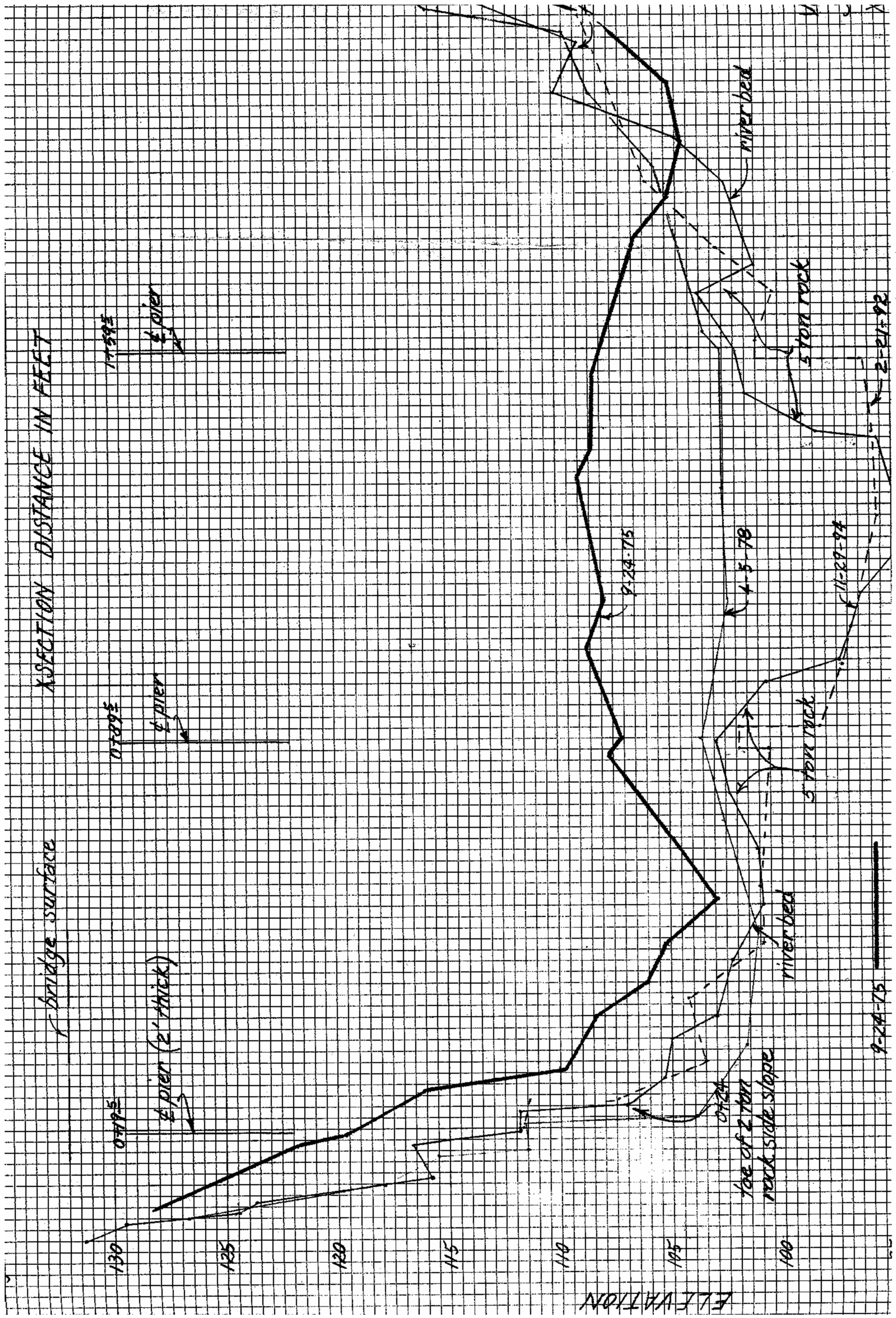


Figure 5.29. Bed elevation changes all Shell Road Bridge (from Ventura County Records of William Carey).

5.7.3. Historical Morphology of the Pre-Dam Matilija Creek Upstream of Matilija Dam

The 1947 Matilija Creek channel is shown in Figure 5.30, along with the location of the current reservoir area. The pre-dam channel was relatively wide in area upstream of Matilija Dam and was on the left side of the current reservoir area. The pre-dam stream centerline is the best estimate for the most stable stream centerline.

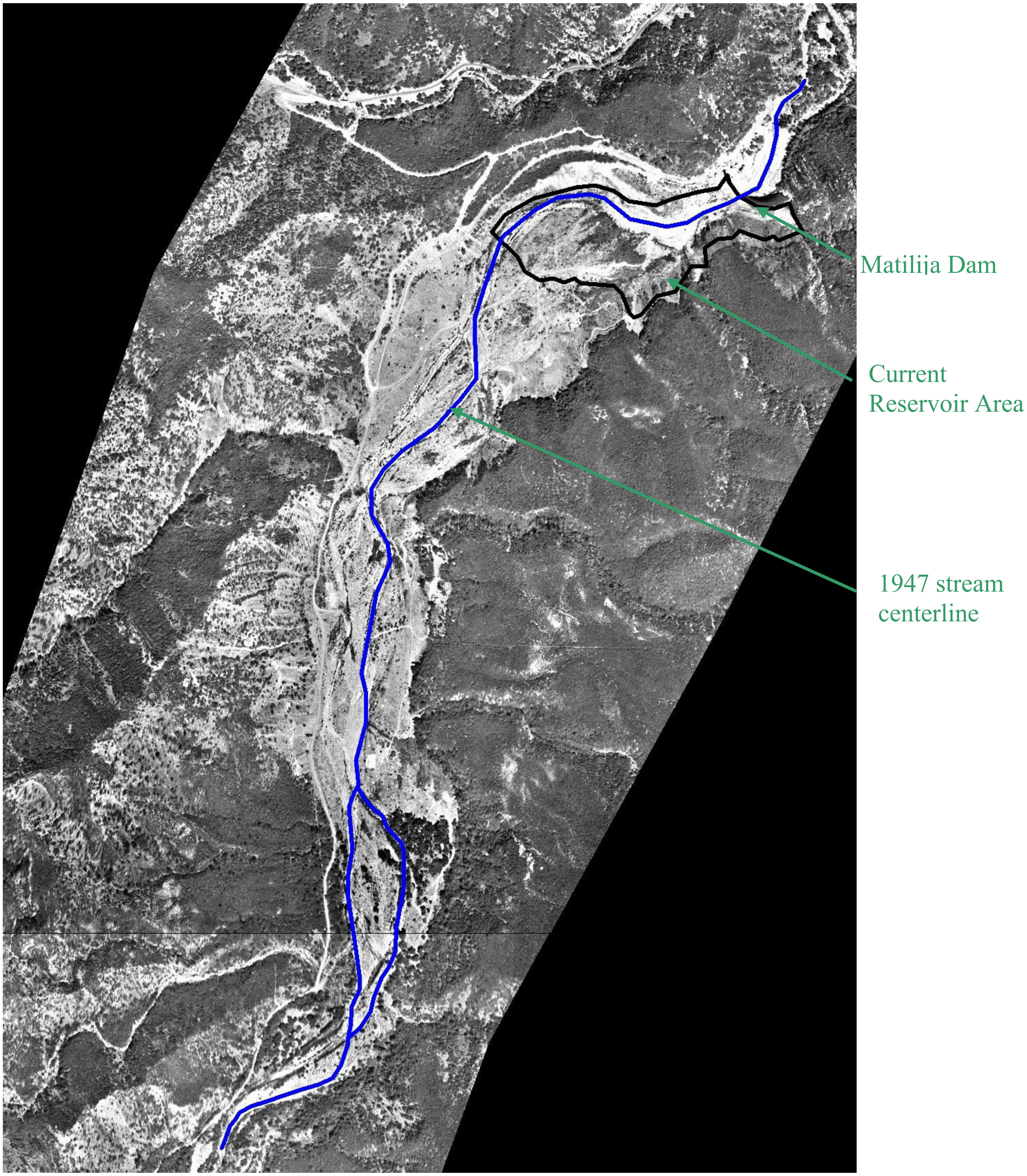


Figure 5.30. Aerial Photograph Taken in 1947 of Matilija Creek Upstream of Matilija Dam.

5.8. Historical Coastline Changes at Mouth of Ventura River

The 1947, 1970, and 2001 aerial photographs were used to analyze changes to the coastline during that period. The coastline was digitized using the waterline as the estimate of the coastline. The results are shown in Figure 5.31. The 1970 coastline protruded into the ocean the furthest. The 1947 and 2001 coastlines were relatively similar, with the 1947 coastline being slightly further into the ocean. The 1970 photo was taken soon after the 1969 flood which carried over 6 million tons of sediment to the ocean (Figure 5.2) and was the largest annual sediment load since 1929. Based on these photos, the natural annual variations of the coastline are quite large and therefore, it is difficult to quantify the effect of Matilija dam on the coastline.

Even though the variability of the coastline seems to dominate any trends that are occurring, there has been significant erosion since 1970. Moffatt and Nichol (2003) stated that over the period of available photography (1947 to 2002) that the beach at Emma Wood has receded approximately 150 feet. Most of that erosion was attributed to the extreme waves during the winter floods of 1981 and 1983.

Figure 5.31. Aerial Photograph of Coastline at Mouth of Ventura River

5.9. Future Without-Project Channel Morphology, Sediment Transport, and Reservoir Sedimentation

5.9.1. FUTURE WITHOUT-PROJECT CONDITIONS IN MATILIJA CREEK AND VENTURA RIVER

Based on the analysis of the channel morphology (Section 5.7) and numerical simulations (see Section 9.1), the Ventura River, as a whole, will remain in a dynamic equilibrium state under without-project conditions for the 50 year project life. Dynamic equilibrium means that the average properties of the river, its average slope, width, and depth will not change significantly, but at any given location in the river, there may be significant and largely unpredictable changes. For example, the channel may shift from one side of the river valley to the other during a large flood event. There are significant exceptions to this general dynamic stability, however, that are listed below.

Robles Diversion will continue to trap sediment and that sediment will continue to be mechanically removed. The removal of sediment at Robles Diversion has been largely responsible for the channel erosion that has occurred from RM 14 to RM 13, immediately downstream of Robles Diversion. Up to 10 feet of channel elevation change has occurred since 1970. The erosion is expected to continue, but at a decreased rate. The numerical modeling with GSTARS-1D shows that the erosion will progress downstream in the next 50 years (Figure 19.136, p. 449).

Another location of instability is at Santa Ana Bridge (RM 9.3). The bridge and levee are a severe constriction on the flow and therefore it causes deposition upstream of the constriction that may progress downstream through the levee region. Numerical modeling predicts that some deposition will continue to occur in this region. The County of Ventura currently has a program to excavate sediment at Santa Ana Bridge and therefore the present river channel will be maintained, but only through continued maintenance.

Some deposition along the reach of Casitas Levee is expected over the next 50 years. The deposition could be due to the levee and high sediment loads from San Antonio Creek. Approximately 2 feet of aggradation is expected in this reach over the next 50 years.

Another reach that is not stable is the reach between RM 3 and 5. This region is relatively confined and has historically undergone extensive channel incision. This reach is expected to continue to degrade in the future with up to 2 to 3 feet of erosion in this reach.

5.9.2. FUTURE WITHOUT-PROJECT CONDITIONS IN MATILIJA RESERVOIR

To estimate the future deposition in the reservoir and the deposition in the upstream delta, an equilibrium slope was estimated as one-half of the natural slope through the reservoir area. This estimate is based on criteria developed by the Los Angeles District of the U.S. Army Corps of Engineers and presented in the U.S. Army Engineering Manual, EM 1110-2-1601. In addition, Strand and Pemberton (1987) measured the slope of several deltas and found that the topset slope (the slope of the delta upstream of the reservoir) varied between 20% and 100% of the natural stream slope. The average was approximately 50%. Based on the current topography, 50% of the natural slope of the Matilija Creek canyon is approximately 1.1%.

For the purposes of this study, it was assumed that sediment deposits in an arcing pattern as it enters the reservoir. The arc has a radius of 1500 ft centered on the stream centerline. This is consistent with the present shape of the delta as shown in Figure 5.35. When a slope of 1.1% is projected upstream to the point where it terminates into grade using an arcing pattern, it intersects the current stream profile at a distance of 9,360 feet from the dam.

Three different estimates for the depositional rate were computed. A high estimate was selected by assuming the sediment continues to deposit at the same rate it has historically. The historical rate was developed using the data in Table 5.4 and is 72 ac-ft/yr. The low estimate is assuming the rate is 36 ac-ft/yr or half of the historical. One-half of the historical rates of deposition were chosen for the low estimate of reservoir deposition, because the trap efficiency will decrease as the reservoir fills. The third is the average of the high and low or 54 acre-ft/yr. This third estimate is considered the best estimate and should be used for planning purposes. Using the middle estimate for deposition, the best estimate of future conditions in the reservoir is given in Table 5.23. The reservoir delta is expected to reach an equilibrium condition by 2038 with a slope of 1.1% and a total of 9.3 million yd^3 of sediment stored behind the dam.

The future reservoir storage was also estimated using historical data and extrapolating the sedimentation rates and accounting for the reduction of the trap efficiency. The projected trap efficiency was computed from an exponential function below,

$$\textit{Trap Efficency},\% = 95\left(1-\exp\left(-0.029\sqrt{S}\right)\right) \qquad \text{Eq 5.9}$$

where S is the storage of Matilija Reservoir. The equation is an approximate fit to the Brune Curve for this reservoir. Using a current reservoir deposition rate of 72 ac-ft/yr, the predicted reservoir storage and trap efficiency of the reservoir is shown in Table 5.23. The reservoir is predicted to have less than 50 ac-ft of storage by 2020.

Table 5.23. Projected deposition with dam in place.

Year	**Dam Crest Elevation**	**Reservoir Storage (ac-ft)**	**Est. Trap Efficiency of Reservoir (%)**	**Est. Deposited Volume (yd^3)**
2000	1095	500	45	6,000,000
2010	1095	212	33	6,900,000
2020	1095	31	14	7,800,000
2030	1095	0	0	8,600,000
2040	1095	0	0	9,300,000
2050	1095	0	0	9,300,000
2060	1095	0	0	9,300,000

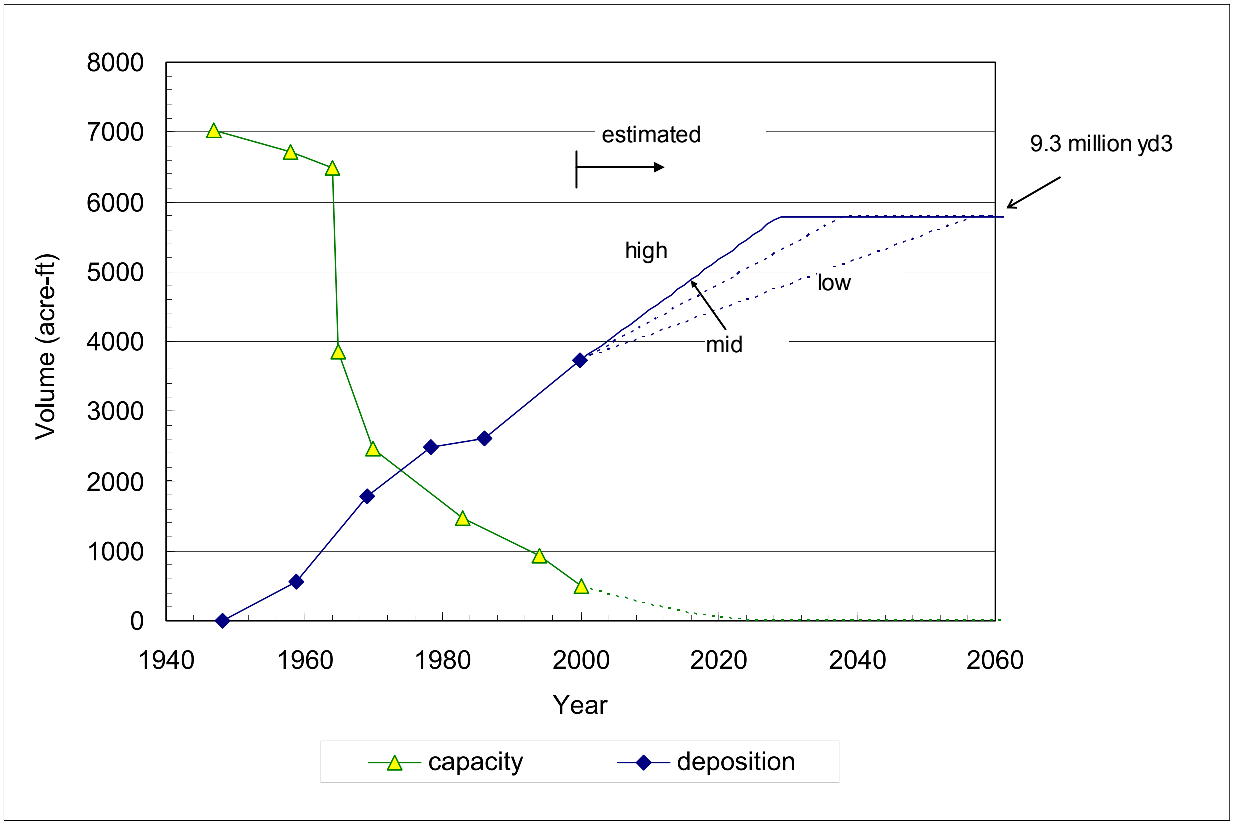


Figure 5.32. Historical and projected future deposition in Matilija reservoir.

The delta is continuing to progress into the reservoir and has become heavily vegetated as seen in Figure 5.35. The first photo was taken in 1973 and shows a non-vegetated delta approximately 2,500 feet upstream from the dam. The next photo, taken in 1985, shows vegetation on the delta. The delta has progressed approximately 500 feet closer to the dam. The last photo, taken in 2001, shows the delta in the present location. It is heavily vegetated and has encroached to within 1200 feet of the dam face. Analyzing the photos implies an average progression rate of 46 ft/yr. This would indicate the delta would reach the dam face in 25 years. This prediction correlates with the high estimate of the projected deposition in Matilija Reservoir as presented in Table 5.23. It is expected that the delta progress will slow and the delta will reach the dam face at the same time the equilibrium condition of the delta is obtained, in 2038.

Analyzing the historical photos indicates the channel can migrate after large floods within the lower end of the reservoir. As compared to the present morphology, in 1974, the main channel was on the opposite side of the delta as it enters the lower end of the reservoir. The exact depositional pattern of reservoir sediments is heavily influenced by the main channel location. Therefore, the channel location can change unpredictably from flood to flood, and it is difficult to predict depositional patterns with great accuracy.

Figure 5.33. 1973 Photograph of Matilija Delta. Note: The red circle is located at the same location in the following pictures of the Matilija Reservoir.

Figure 5.34. 1985 Photograph of Matilija Delta. Note: The red circle is located at the same location in each photo.

Figure 5.35. 2001 Photograph of Matilija Delta.

Note: The red circle is located at the same location in each photo. Arc with radius of approximately 1500 feet centered on the stream centerline is shown on last photo.

6. Description of Alternatives

Seven alternatives were analyzed:

Alternative	Description
No Action	No removal of dam or sediments
1	Full Dam Removal/Mechanical Sediment Transport: Dispose Fines, Sell Aggregate
2a	Full Dam Removal/Natural Sediment Transport, Slurry “Reservoir Area” Fines Offsite
2b	Full Dam Removal/Natural Sediment Transport, Natural Transport of “Reservoir Area” Fines
3a	Incremental Dam Removal/Natural Sediment Transport, Slurry “Reservoir Area” Fines Offsite
3b	Incremental Dam Removal/Natural Sediment Transport, Natural Transport of “Reservoir Area” Fines
4a	Full Dam Removal/Sediment Stabilization on Site, Permanent Stabilization
4b	Full Dam Removal/Sediment Stabilization on Site, Temporary Stabilization

[1] Throughout this document, the term “Reservoir Area” will be used to refer to the area normally covered by water due to Matilija Dam.

Analysis will be presented for all alternatives, but in some cases, the sediment impacts between alternatives are similar. For example, the sediment impacts of Alternative 1 (Mechanical Sediment Transport) are expected to be similar to those of Alternative 4a (Permanent Stabilization). In addition, the long-term impacts of Alternative 2, 3, and 4b will be similar. Therefore, in many cases the impacts from several alternatives are discussed simultaneously. The items relating to sediment management for each alternative are briefly described below.

<u>Alternative 1</u>

1. Removal of reservoir fines by hydraulic slurry line. There is approximately 2.1 million yd^3 of sediment in the reservoir area that is 30 % clay, 53 % silt, and 17 % sand that will be removed and deposited on the terraces in the downstream river valley.

2. Complete removal of dam in one stage.

3. Construction of temporary revetment to stabilize the remaining sediment.

4. Removal of remaining sediment over a period of time by truck.

<u>Alternative 2a</u>

1. Removal of reservoir fines by hydraulic slurry line as in Alternative 1.
2. Complete removal of dam in one stage.
3. Construction of pilot channel through sediments.
4. Natural erosion of remaining sediment.

<u>Alternative 2b</u>

1. Complete removal of dam in one stage. Some reservoir sediment will be removed from behind the dam to facilitate dam removal. This sediment will be placed on top of the delta sediment and will be allowed to erode along with the delta sediments.
2. Construction of pilot channel through sediments.
3. Natural erosion of all remaining sediment.

<u>Alternative 3a</u>

1. Removal of reservoir fines by hydraulic slurry line as in Alternative 1.
2. Removal of dam to an elevation of 1020 feet.
3. A waiting period until first flood passes through reservoir.
4. Removal of remaining dam.

<u>Alternative 3b</u>

1. Removal of dam to an elevation of 1020 feet. Some reservoir sediment will be removed from behind the dam to facilitate dam removal. This sediment will be placed on top of the delta sediment and will be allowed to erode along with the delta sediments.
2. A waiting period until first flood passes through reservoir.
3. Removal of remaining dam.

<u>Alternative 4a</u>

1. Removal of reservoir fines by hydraulic slurry line as in Alternative 1.
2. Complete removal of dam in one stage.
3. Construction of pilot channel through sediments and stabilization of all remaining sediments.

<u>Alternative 4b</u>

1. Removal of reservoir fines by hydraulic slurry line as in Alternative 1.

2. Complete removal of dam in one stage.

3. Construction of pilot channel through sediments and temporary stabilization of all remaining sediments.

4. Staged removal of temporary stabilization structures until all structures are removed.

5. One flood will be allowed to pass through the reservoir area before any revetment is removed.

6. There will be at least three stages of revetment removal with there being most likely four separate removals of revetment.

7. Analytical Modeling of the Deposition Downstream of Matilija Dam

This section describes an analytical model of the movement of the sediments behind Matilija Dam. The model is much simpler than the numerical model (GSTARS-1D) used in Section 8. It provides a check on the results of the more complicated numerical model and insight into the expected deposition impacts expected downstream.

The model is taken from Greimann et al. (2004), who extended the analytical description of aggradation of Soni et al. (1980) to describe downstream aggradation following dam removal. In Greimann et al. this analytical model is also verified using experimental data from St. Anthony Falls Laboratory in Minneapolis, MN. A schematic of idealized representation of the movement of a sediment accumulation is shown in figure 9.5. The sediment accumulation sits on top of the original bed material that is at a stable and uniform slope.

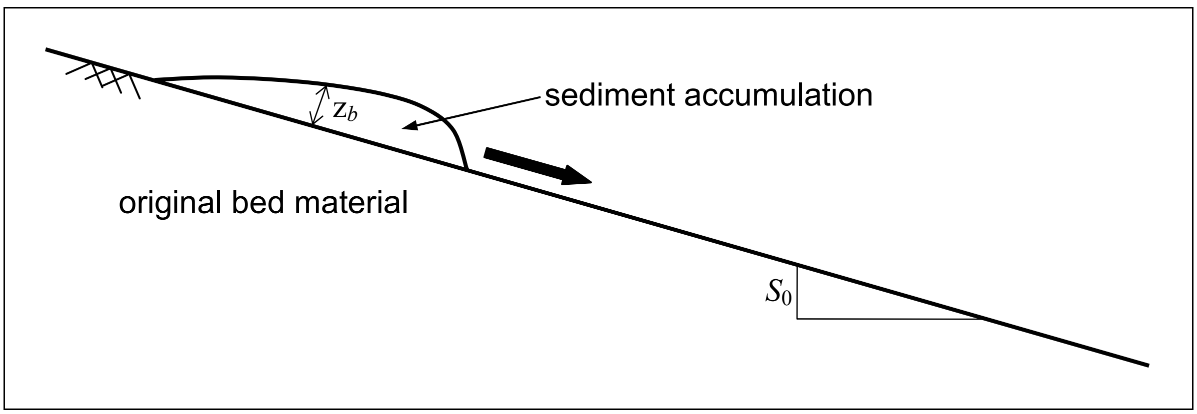


Figure 9.5. Schematic of idealized representation of the movement of a sediment accumulation, from Greimann et al. (2004).

The following equation was derived by Greimann et al. (2004),

$$\frac{\partial z_b}{\partial t} + u_d \frac{\partial z_b}{\partial x} = K_d \frac{\partial^2 z_b}{\partial x^2} \qquad \text{Eq 7.1}$$

where: z_b = depth of the sediment original trapped behind the dam,
u_d, = velocity of sediment wave translation.

The variable u_d is defined as,

$$u_d = \frac{\left(G_d^* - G_0^*\right)}{h_d(1-\lambda)} \qquad \text{Eq 7.2}$$

where: G_d^* = transport capacity in units of volume per unit width of the deposit material,
G_0^* = transport capacity of the original bed material,

h_d = maximum depth of the deposit,
λ = sediment porosity.

The parameter, K_d, is the aggradation dispersion coefficient and is similar to Soni et al. (1980),

$$K_d = \frac{\left(b_d G_d^* + b_0 G_0^*\right)}{6 S_0 (1-\lambda)} \qquad \text{Eq 7.3}$$

The transport rate of a particular sediment type is related to the flow velocity,

$$G_d^* = a_d U^{b_d}, \qquad G_0^* = a_0 U^{b_0} \qquad \text{Eq 7.4}$$

where: U = averaged flow velocity
a_d , b_d = constants used to calculate the transport capacity of the deposit material
a_0, b_0 = constants used to calculate the transport capacity of the original bed material

The parameter b is generally bounded between 4 and 6 (Chien and Wan, 1999). Equation (7.1) can be solved analytically, and can be applied to arbitrary initial deposits by dividing the stream into N segments,

$$z(x,t) = \sum_{i=1}^{N-1} \frac{\left(z_{1i} + z_{1i+1}\right)}{4} \begin{bmatrix} \operatorname{erf}\left(\dfrac{x - u_d t - x_i}{2\sqrt{K_d t}}\right) - \operatorname{erf}\left(\dfrac{x - u_d t - x_{i+1}}{2\sqrt{K_d t}}\right) - \\ \operatorname{erf}\left(\dfrac{x + u_d t + x_i}{2\sqrt{K_d t}}\right) + \operatorname{erf}\left(\dfrac{x + u_d t + x_{i+1}}{2\sqrt{K_d t}}\right) \end{bmatrix} \qquad \text{Eq 7.5}$$

where the function "erf" is the error function and z_1 is the initial bed elevation. There may have to be some trial and error in determining appropriate distances between stream segments. There should be enough segments so that the initial deposit and resulting bed profiles are adequately defined.

If the sediment accumulation is composed of a variety of sediment sizes, it may not be appropriate to model its movement using one size class. Most often, the sediment accumulation can be divided into three general size classes: wash load, sand, and gravel/cobbles. Wash load is assumed to pass through the river system without depositing; the remaining sediment in the accumulation is divided into sand and gravel/cobbles. The usual demarcation between sand and gravel is 2 mm. Equation (7.5) is then applied separately to the portion of the accumulation that is sand and the portion that is gravel/cobble. The deposition thicknesses resulting from each is then added together. It can be represented in equation form as,

$$z_m(x,t)=\sum_{i=1}^{N-1}\frac{(z_{1i}+z_{1i+1})}{4}\begin{bmatrix}\operatorname{erf}\left(\frac{x-u_d t-x_i}{2\sqrt{K_d t}}\right)-\operatorname{erf}\left(\frac{x-u_d t-x_{i+1}}{2\sqrt{K_d t}}\right)- \\ \operatorname{erf}\left(\frac{x-u_d t-x_i}{2\sqrt{K_d t}}\right)+\operatorname{erf}\left(\frac{x-u_d t-x_{i+1}}{2\sqrt{K_d t}}\right)\end{bmatrix} \qquad \text{Eq 7.6}$$

where $z_m(x,t)$ is the deposition thickness due to size class m at time t and distance from the dam of x. The total deposition thickness can then be determined by,

$$z(x,t)=\sum_{m=1}^{NF} z_m(x,t) \qquad \text{Eq 7.7}$$

where NF is the total number of size fractions used. In this section, NF is equal to 2.

The error of this method is potentially great because of the simplifications made. A partial list follows:

- Assumes a prismatic channel
- Does not account for changes in channel geometry with distance along the channel
- Does not considered longitudinal slope breaks due to channel controls
- Assumes a steady flow rate
- Does not account for changes in roughness
- Is not applicable upstream of the sediment accumulation
- Assumes deposit travels as bed load.
- Ignores sediment sizes in the sediment accumulation that will travel as pure suspended load.

Despite these shortcomings, this method holds promise as a simple assessment tool to determine impacts associated with aggradation. This method requires a minimal number of input parameters and can be completed in a fraction of the time required to complete a more complicated and time-consuming numerical model. The parameters that need to be estimated to use the model are listed in Table 7.1. All the parameters except for b_d are physical quantities that can be measured. The parameter b_d is the exponent in the sediment transport relation and based on results from several researches is generally bounded between 4 and 6 (Chien and Wan, 1999).

Table 7.1. Description of parameters necessary to use proposed model.

Parameter	Range of Values or Method of Obtaining Value
S_0	Average natural stream slope. Measured from topographic maps.
G_d^* (L^2/T)	Transport capacity of sediment accumulation in units of volume per unit width
G_0^* (L^2/T)	Transport capacity of bed material in units of volume per unit width
λ	Sediment porosity, usually between 0.3 to 0.5
b_d	Exponent in sediment transport relation, usually between 4 and 6
h_d (L)	Maximum depth of sediment accumulation. Estimated from field surveys

The analytical model was applied to the case of the Matilija Dam removal using the parameters in Table 7.2. The initial deposit depths were taken from the 2001 survey information and 1947 pre-dam information. The silt and clays fractions were ignored in this analysis because they will not deposit in the river channel. The remaining sediment was divided into sand and gravel. It was assumed that the transport rate for sand was 5 times greater than that of the gravel. This analysis was mainly concerned with the maximum depths possible and not the timing of the deposition. Therefore, only the ratio of the transport rates are important and not their magnitude. The deposition due to each fraction was calculated using the method described above.

Table 7.2. Parameters used in analytical model of Matilija Dam removal.

Parameter	**Value for Matilija Dam**
S_0	0.015
G_d^* - sand (ft^2/s)	0.2
G_d^* - gravel (ft^2/s)	0.04
G_0^* (ft^2/s)	0.02
λ	0.4
b_d	5
h_d – sand (ft)	14.8
h_d – gravel (ft)	15.7

The results show that the deposition decreases rapidly downstream of the dam. By RM 14 (just downstream of Robles Diversion), the deposition is less than 3 feet. At RM 9 (just downstream of Santa Ana Bridge), it is less than 2 feet.

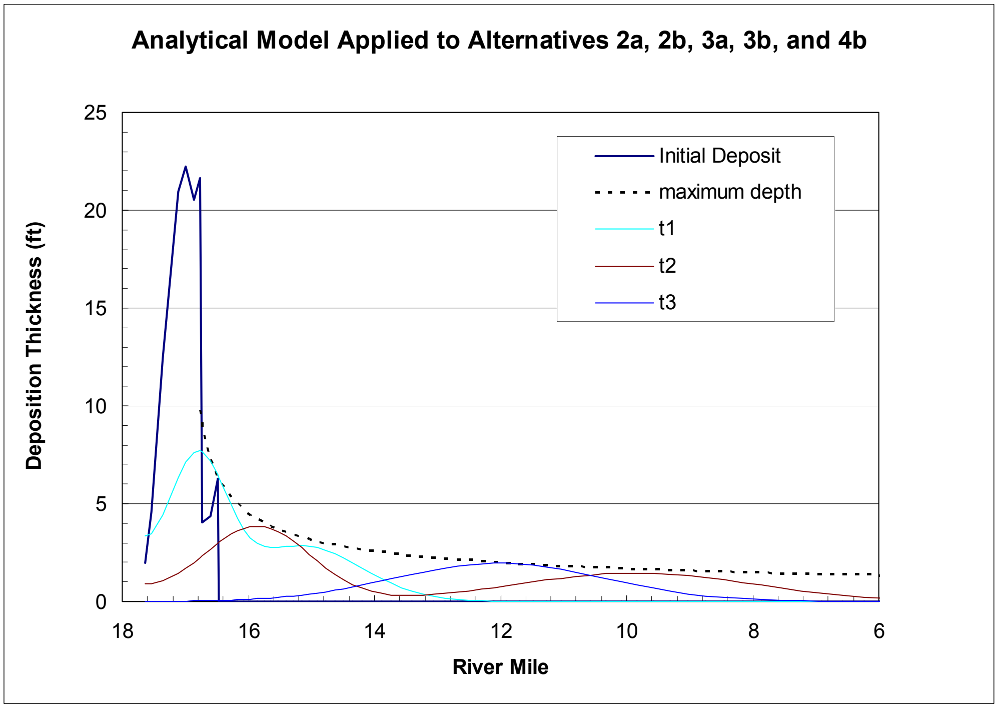


Figure 7.1. Analytical prediction of aggradation downstream of Matilija Dam for the alternatives that allow the coarse sediment to travel downstream (2a, 2b, 3a, 3b, and 4b). Each line on the graph represents a different time. The thick dotted line is the maximum aggradation expected at given locations throughout the river.

An upper limit of deposition was also calculated using the analytical model. The greatest amount of deposition downstream occurs when the sand and gravel transport rates are identical. In this case, at RM 14, the maximum deposition is approximately 5 feet and at RM 9, the deposition is approximately 3 feet.

Table 7.3. Parameters used in analytical model of Matilija Dam removal to calculate an upper limit estimate.

Parameter	Value for Matilija Dam
S_0	0.015
G_d^* - sand (ft^2/s)	0.04
G_d^* - gravel (ft^2/s)	0.04
G_0^* (ft^2/s)	0.02
λ	0.4
b_d	5
h_d – sand (ft)	14.8
h_d – gravel (ft)	15.7

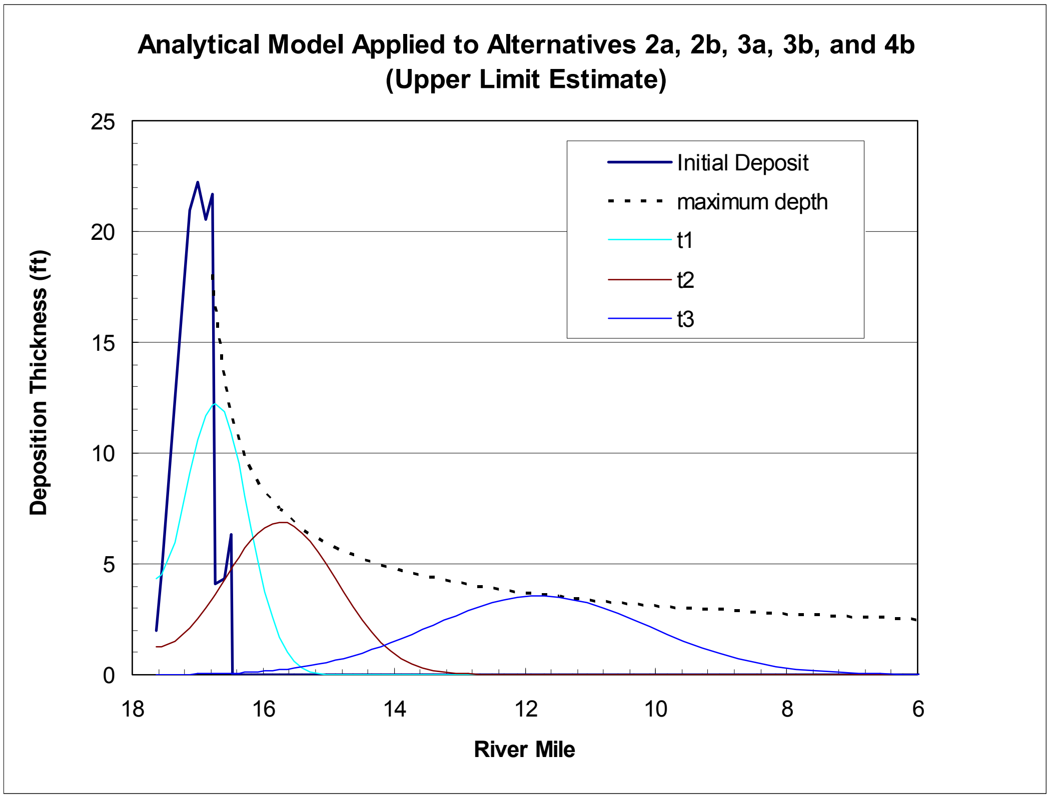


Figure 7.2. Upper Limit Estimate of Aggradation downstream of Matilija Dam for the alternatives that allow the coarse sediment to travel downstream (2a, 2b, 3a, 3b, and 4b). Each line on the graph represents a different time. The thick dotted line is the maximum aggradation expected at given locations throughout the river.

A comparison between the analytical and numerical model GSTARS-1D is shown in Figure 7.3. Alternative 2a was simulated using GSTARS-1D and the results are shown after a 50-yr period of simulation. In general, the aggradation predicted by the GSTARS-1D model is between the upper and best estimates of the aggradation predicted by the analytical model. The agreement between the two methods is a beginning in the verification process of each method.

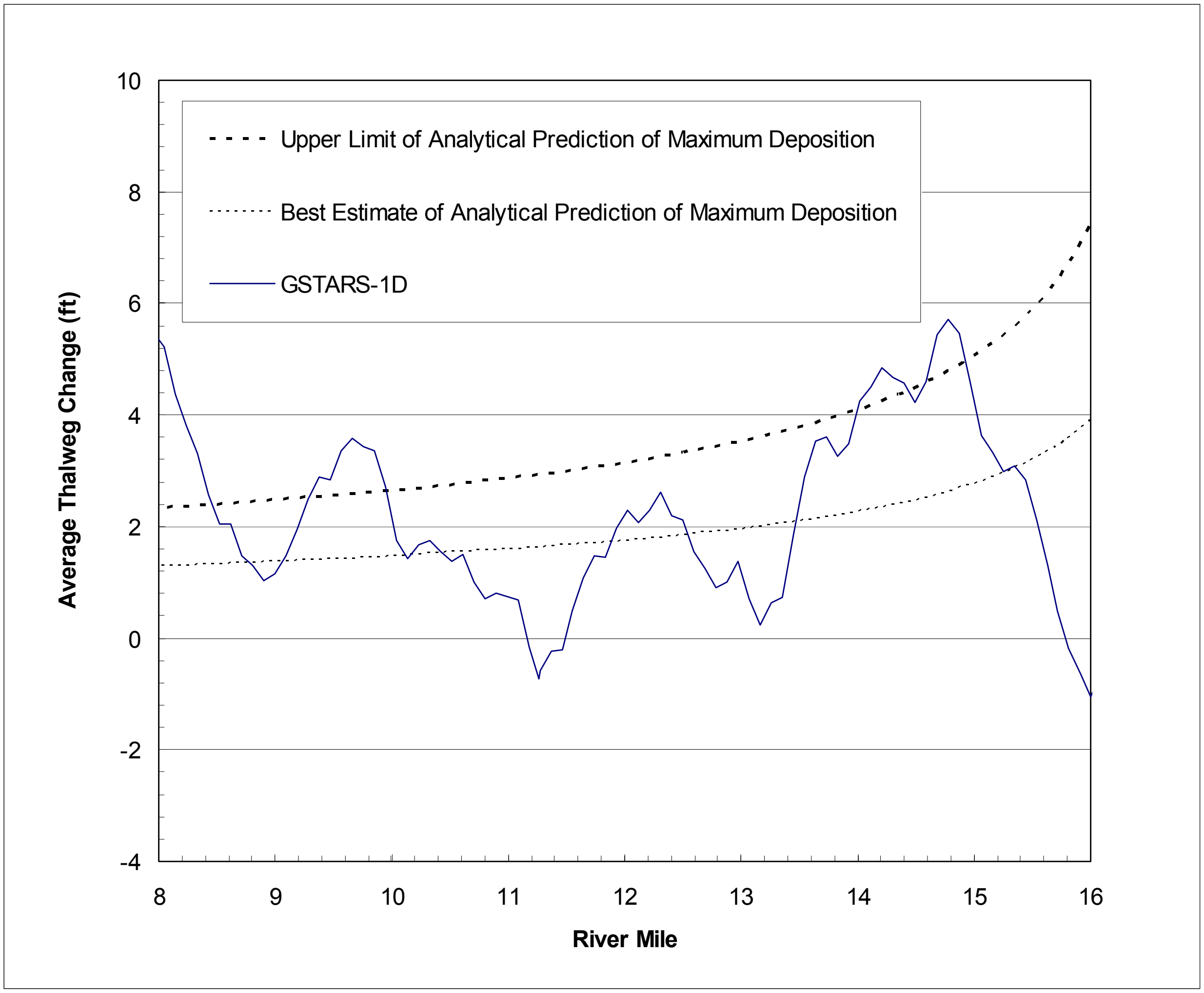


Figure 7.3. Comparison of 50-yr Simulation of Alternative 2a with Analytical Model.

8. Numerical Modeling of the Removal

The GSTARS-1D (Generalized Sediment Transport Model for Alluvial River Simulation – One Dimension) model was used to model the sediment transport resulting from the removal of Matilija Dam (Yang et al., 2003). It is a model that was developed by the Bureau of Reclamation with support from the USEPA. The model requires multiple inputs that can be divided into three main types: Hydrologic, Hydraulic, and Sediment input.

8.1. Hydrologic Input

Several different hydrological inputs were used in the evaluations of alternatives. The 1991 to 2001 hydrograph was simulated 5 times in succession to generate the 50-yr hydrograph. The period 1954 to 1960 was used as a representative dry hydrograph, and this was used to analyze the turbidity impacts associated with a drought period. Several single events were also simulated to assess the impacts associated with the first flood after dam removal. The single even

1991 to 2001 Hydrograph

The hydrological record from 1991 to 2001 was used in the 50-year period simulation (Figure 8.1). The period 1991 to 2001 was repeated five times to make a total simulation time of 50 years. The flow data from 1991 to 2001 has 15-minute flow data available and therefore it was possible to capture the rapidly varying flow conditions of the Ventura River. The inflows from North Fork Matilija Creek and San Antonio Creek were also simulated. Only the flows during the floods were modeled, therefore, low flows were ignored in the simulation. This is acceptable for sediment transport simulations because the only significant sediment transport takes place during flood events.

The period from 1991 to 2001 was a relatively wet period (see Figure 2.6 and Figure 2.7). The largest flows during this period occurred in 1992, 1995, and 1998 (Table 8.1). The largest flow rate at Matilija Dam was 14,000 cfs and occurred on 2/23/1998. If a relatively drier period were simulated, the resulting impacts would be similar but the time between floods would increase. For example, if the 1940s and 1950s were simulated, the deposition and concentration impacts would be similar to that simulated using the 1990s hydrology, however, it would take longer to generate the maximum deposition. In addition, the time between flood events would increase and therefore, it would take a greater number of years to approach equilibrium conditions for each alternative.

Table 8.1. Peak flows during period from 1991 to 2001 at USGS gage 11118500 in the Ventura River at Foster Park.

Date	Flow (cfs)	Return Period (yr)
3/19/1991	11300	5.1
2/12/1992	45800	19.5
1/18/1993	12500	5.5
2/20/1994	1820	<2
1/10/1995	43700	17.5
2/20/1996	3660	2
1/26/1997	4960	2.3
2/23/1998	38800	12.6
1/31/1999	106	<2
2/23/2000	3280	2
3/6/2001	19100	7.3

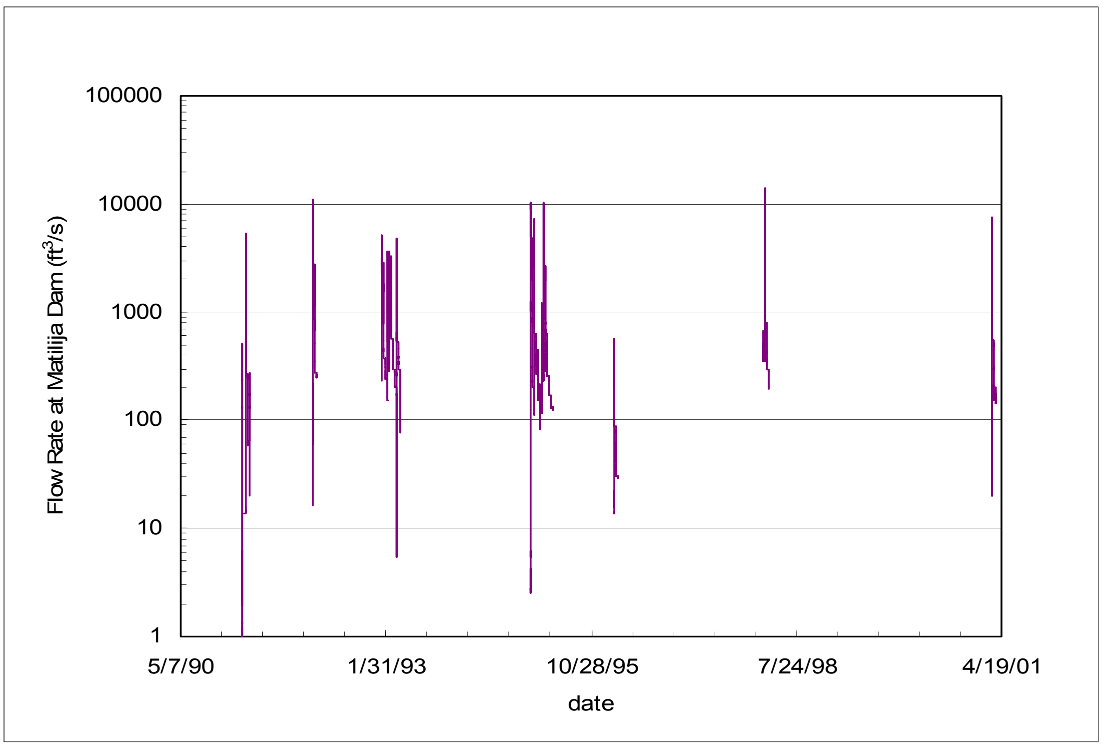


Figure 8.1. Flow at Matilija Dam for the period from 1991 until 2001, only floods are shown.

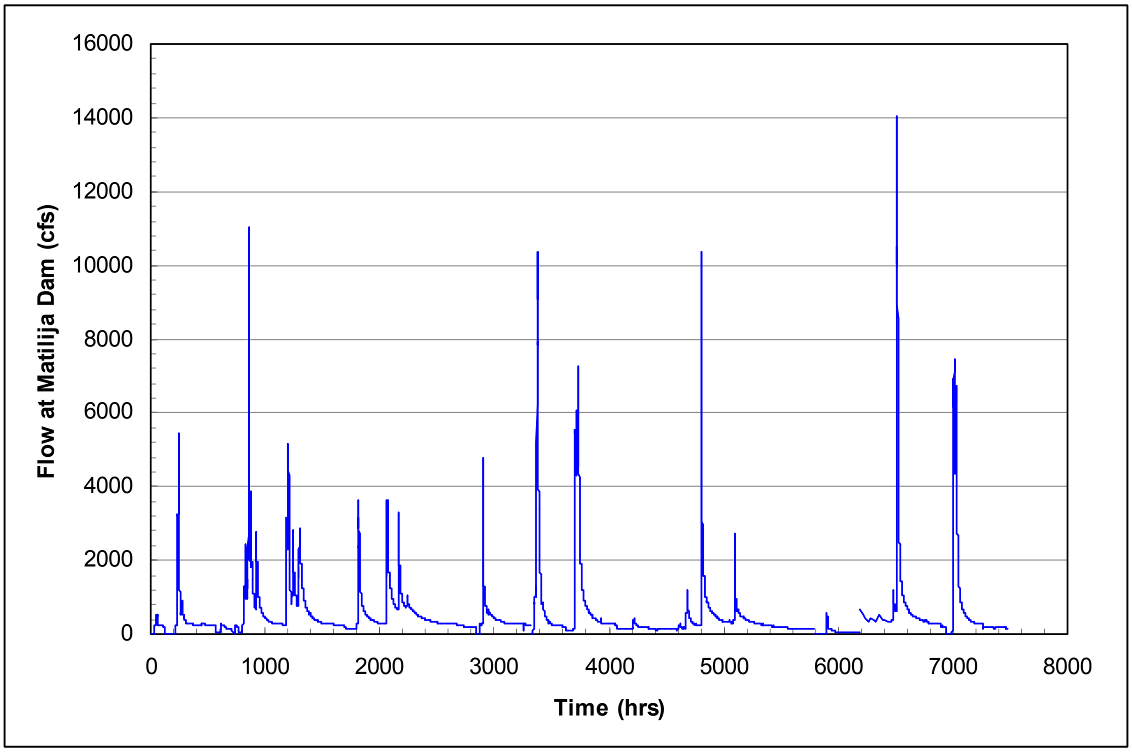


Figure 8.2. Flow at Matilija Dam for the floods between 1991 and 2001 with intervening time eliminated.

Dry Period Hydrograph

A low flow period was also used to simulate the effect that a dry period would have on the erosion of sediment from the reservoir and the resulting turbidity impacts downstream for the alternatives 2 and 3. The period from 1954 to 1960 was selected as a representative dry period.

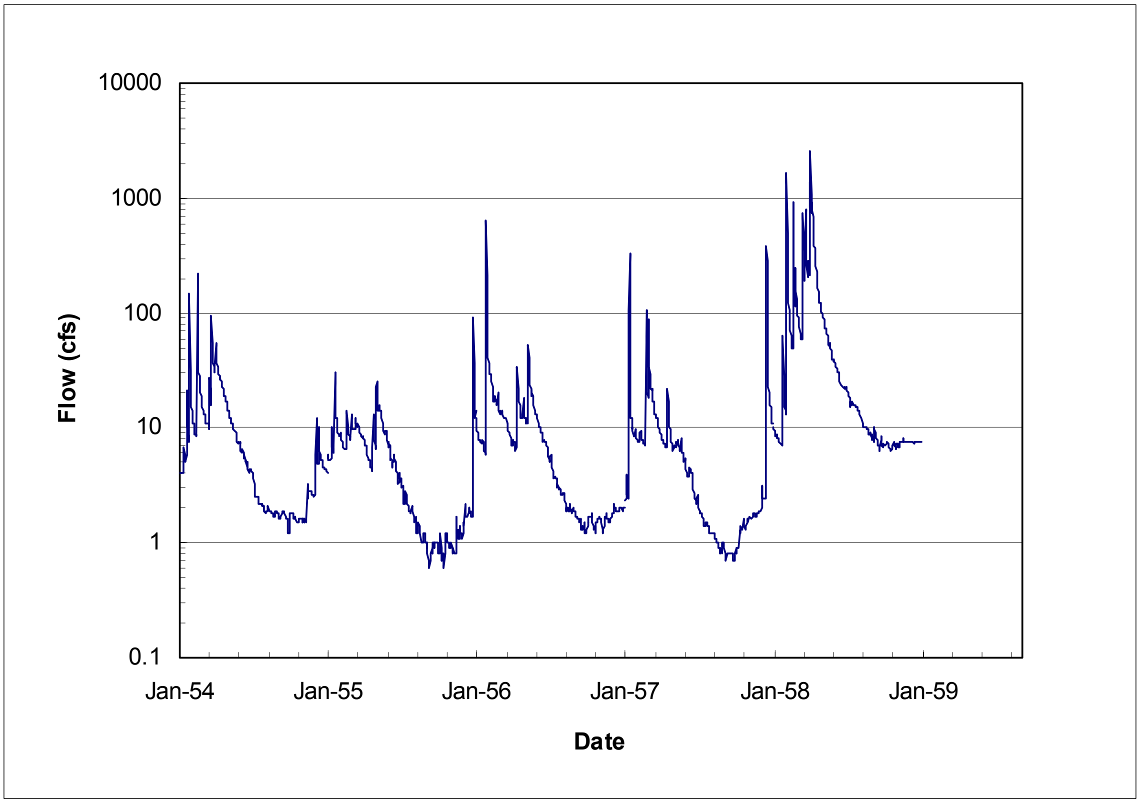


Figure 8.3. Daily average flows at USGS gage 11118500, near Foster Park on the Ventura River.

Single Event Hydrographs

In addition to the long-term simulations, several single floods were simulated. The floods of 1998 and 1991 were chosen as representative floods corresponding to the approximate 15 year and 3 to 4 year floods, respectively. The 100-yr flood was simulated by increasing the flows of the 1991 flood by the same factor so that a 100-yr peak was generated.

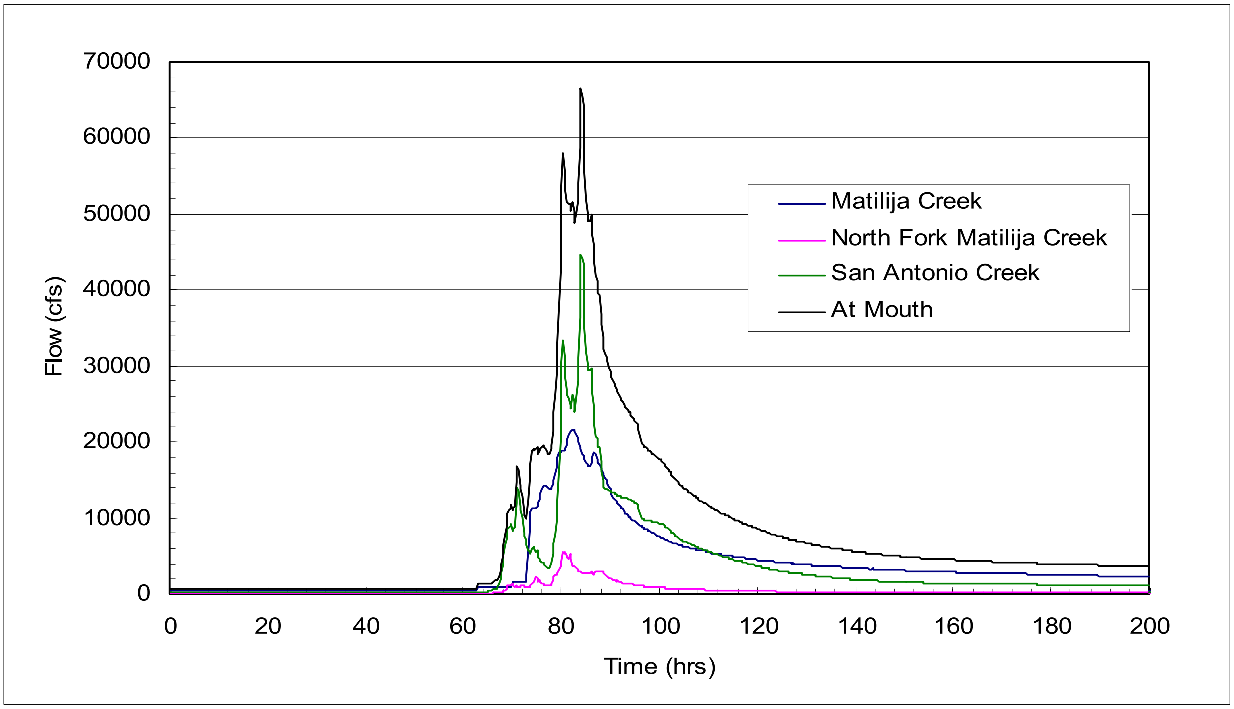


Figure 8.4. Hydrograph for synthetic 100-yr flood.

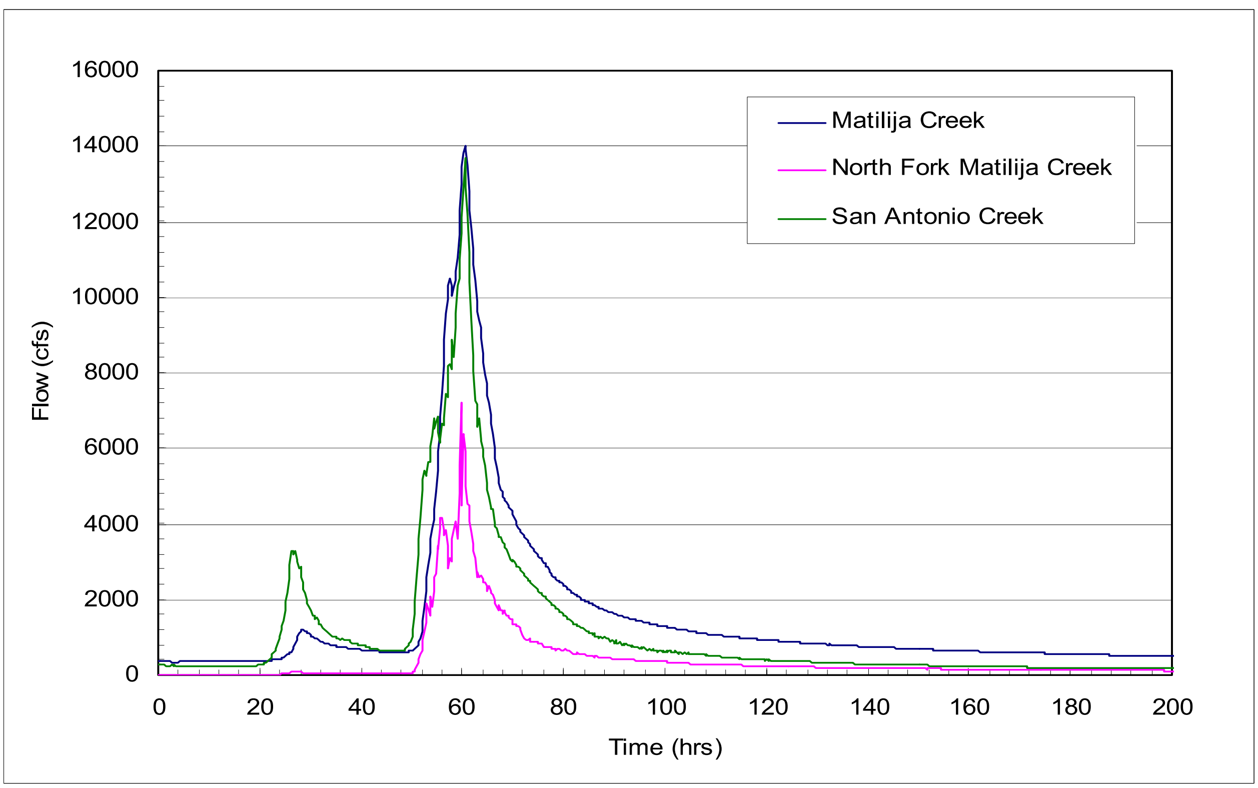


Figure 8.5. Hydrograph for 1998 flood.

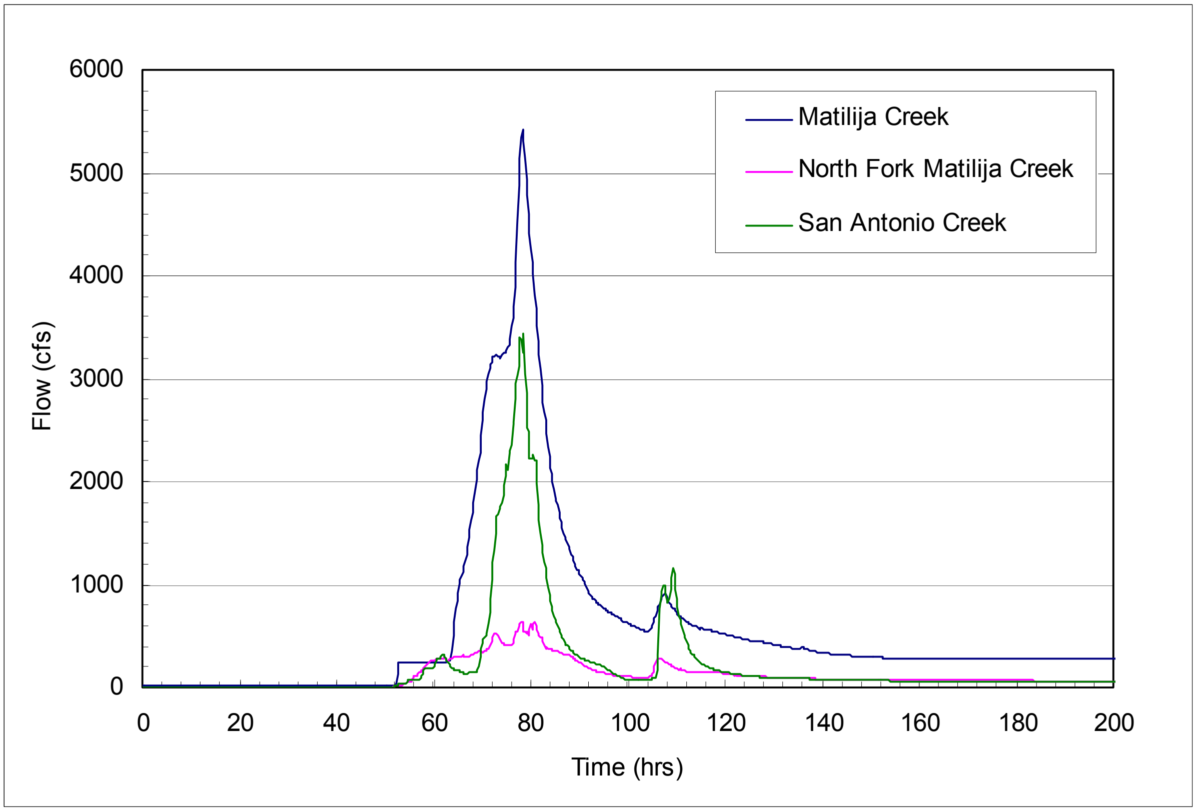


Figure 8.6. Hydrograph for 1991 flood.

8.2. Hydraulic Input

The channel geometry used in the sediment calculations was the same as used in the floodplain analysis reported in Section 4. Cross sections were usually spaced approximately 500 feet apart. The hydraulic roughness coefficients used in the model are listed in Table 8.2. The GSTARS-1D model does not allow the roughness to change with flow rate or water depth and therefore it is necessary to use a constant roughness. They roughness coefficients were increased slightly in the canyon immediately downstream of Matilija Dam because of the presence of large boulders in this area. The model sensitivity to roughness was analyzed in Exhibit L. Sensitivity of Alternative 2a Impacts to Changes in Numerical Model.

Bridges were not included in the sediment model. The bridges that could potentially affect the simulation are the Camino Cielo Bridge and Santa Ana Bridge. Camino Cielo is a low water crossing and has the potential affect of increasing the sediment deposition immediately upstream of this structure. The impact associated with Camino Cielo is only expected to be the area approximately 250 feet upstream of the structure. If analysis that is more detailed is required in this area, Camino Cielo Bridge can be included in future analysis. Concerning Santa Ana Bridge, this report current is suggesting that Santa Ana Bridge be redesigned to accommodate the increased sediment loads. The sediment modeling performed here assumes that Santa Ana Bridge is removed and replaced with a bridge that passes the 100-yr flood.

Table 8.2. Channel roughness used in the sediment model.

River Mile	Channel Roughness
>16.47	0.050
16.47 – 15	0.065
15 – 14.5	0.055
14.5 – 0	0.045

8.3. Sediment Transport Input

The information required for sediment transport calculations are the incoming sediment load, the sediment gradations in the bed and reservoir, transport relations for non-cohesive sediment, transport relations for cohesive sediment, and initial cohesive sediment bulk density.

8.3.1. INCOMING SEDIMENT LOAD

The sediment load that enters from the upstream end on Matilija Creek was calculated with the sediment transport function described in the section on non-cohesive transport relations (8.3.4). The bed material data was taken from the measurements reported in Section 5.3. The sediment loads were computed for each size fraction. Using Eq. 2 of section 5.2.2, a sediment rating curve was fit to the total load of the computed data and the coefficients were then calibrated to match the observed deposition in Matilija Reservoir (Table 8.4). The fraction of the sediment load was kept the same as was determined by the sediment transport function. The calibrated values of *a* and *b* were 0.045 and 1.83, respectively. It was assumed that 100% of the sediment sand sized and larger is trapped behind Matilija Dam. The amount of fraction of silt relative to the other size classes was assumed the same as that reported at Ventura Gage. The trap efficiency of the silt was adjusted based on the volume of deposition in Matilija Reservoir and using the Brune Curve (Table 8.3). The procedure to determine the silt trap efficiencies was iterative because the trap efficiency affects the reservoir storage, which is directly related to the trap efficiencies. Therefore, the trap efficiency was first assumed, and then the sedimentation rate was calculated. Based upon the sedimentation, the trap efficiency was updated. This procedure was continued until adequate convergence was obtained. There is a large amount of uncertainty in determining the trap efficiency of silt and clays in Matilija Reservoir. Therefore, the trap efficiencies reported in Table 8.3 are rough estimates. Because errors in the trap efficiencies will propagate into errors of computing the inflowing silt and clay load, there is potential that the inflowing silt and clay loads used in the sediment transport model are somewhat inaccurate. However, the silt and clays do not deposit in the river system and do not affect the deposition computed in the river. The largest impact of the silt and clays is in their contribution to turbidity. In terms of this alternative analysis, the relative increase or decrease in turbidity is what is most important and this is only minimal impacted by the errors in predicting the trap efficiency of Matilija Reservoir.

Table 8.3. Trap Efficiency of Silt and Clays in Matilija Reservoir

Year	Silt Trap Efficiency of Matilija Reservoir
1947	90
1965	80
1969	45
1978	35
1990	25
1991	20
1996	15

Table 8.4. Comparison between measured deposition and simulated using sediment rating curve for Matilija Creek.

	Total Deposition in Matilija Reservoir in 2000 (million yd^3)				
	Total	**Silt**	**Sand**	**Gravel**	**Cobble**
computed	5.9	2.78	2.18	0.91	0.050
measured	5.9	2.73	2.16	0.96	0.049

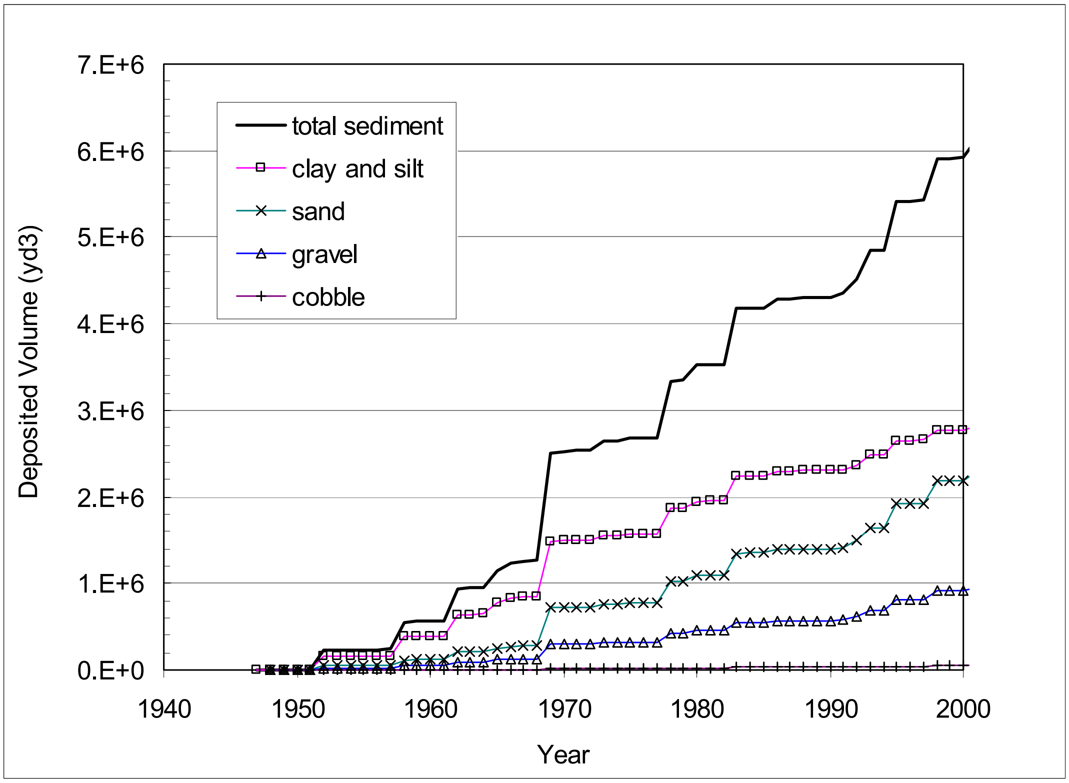


Figure 8.7. Simulated deposition in Matilija Reservoir using sediment rating curve for Matilija Creek.

8.3.2. TRIBUTARY INFLOW

It was also necessary to calculate the sediment load entering at North Fork Matilija Creek and at San Antonio Creek. Hill and McConaughy (1988) report some measurements of the sediment load at San Antonio Creek. However, they are not sufficient to develop sediment relationships for each size fraction present in the bed. The transport formula derived in the Non-cohesive Sediment Transport Section (6.3.3) is used to develop the flow versus sediment load information for each size fraction. A comparison of the values calculated for San Antonio Creek by this relationship and the data measured by Hill and McConaughy is shown in (Figure 8.8).

The computed sediment loads for North Fork Matilija Creek are shown in (Figure 8.9). Internal documents within Reclamation were found that estimated the bed-load from North Fork Matilija Creek. The studies were done during the design of Robles Diversion and subsequent analysis of this diversion. The computed sediment bed-load is primarily comprised of coarse sand and gravel.

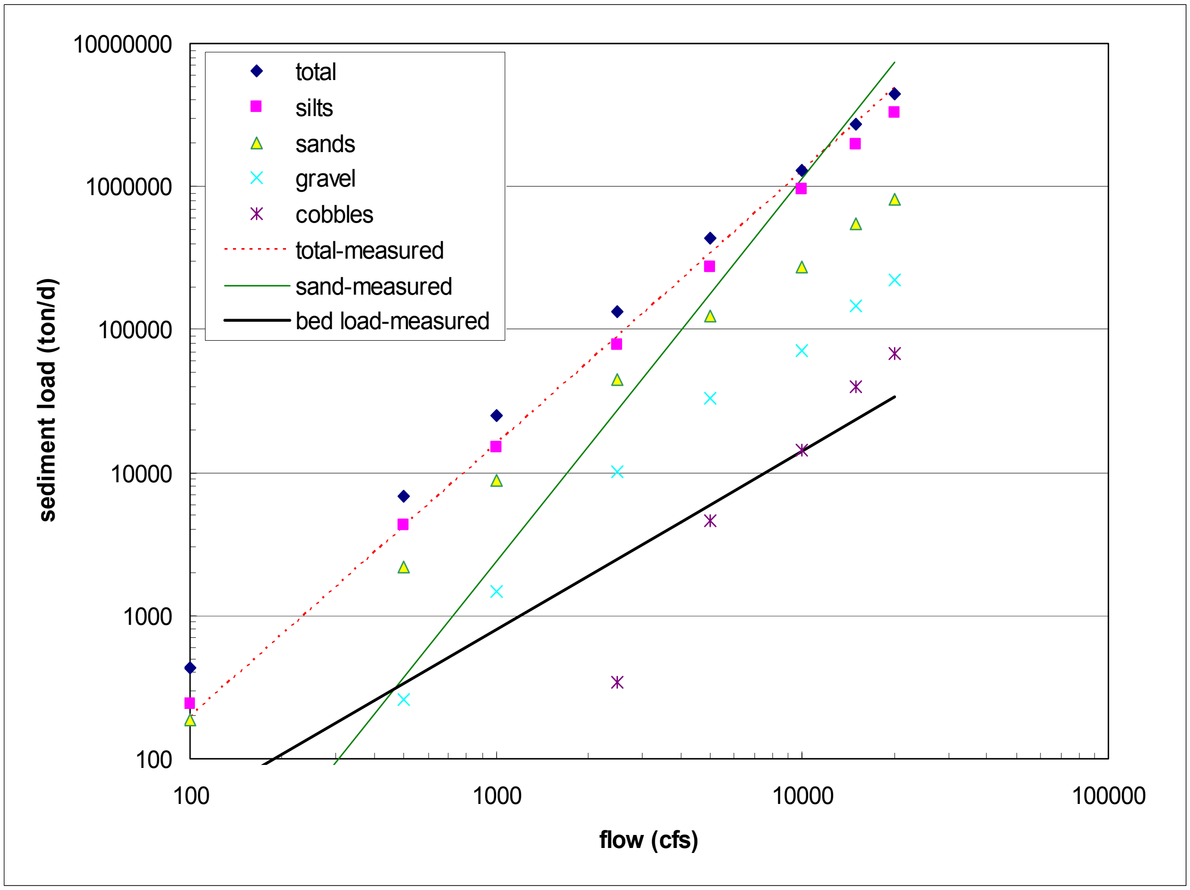


Figure 8.8. Comparison between the sediment load as reported by Hill and McConaughy and that computed from the transport relationship for San Antonio Creek.

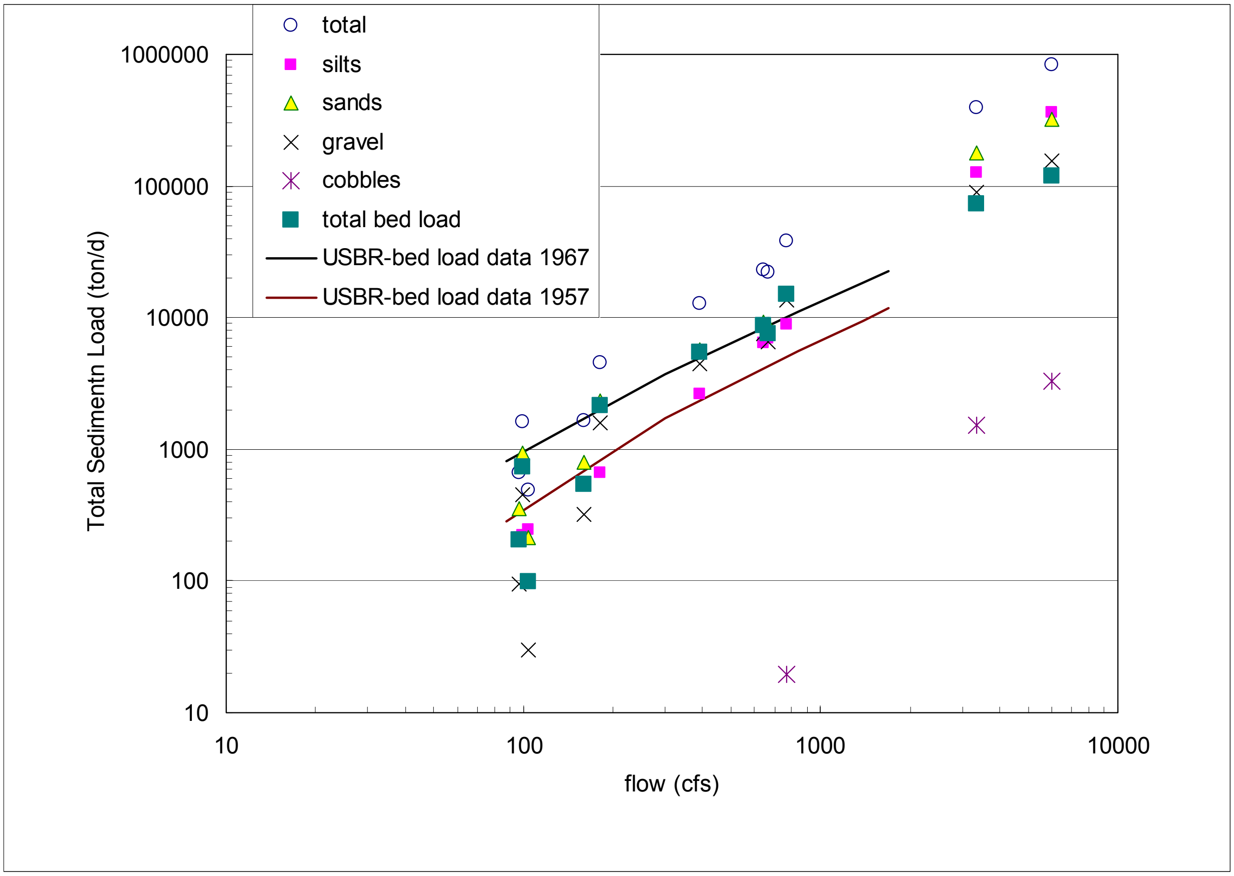


Figure 8.9. Computed sediment load versus flow relationship for North Fork Matilija Creek. Solid lines are data from previous Reclamation reports dated in 1967 and 1957. Points are computed from the transport formula used in this study (Eq. 8.2).

8.3.3. Sediment Gradation in Bed and Reservoir

The bed material gradations have been documented in Section 5.3 and the sediment characteristics of the sediment trapped behind Matilija Reservoir were documented in Section 5.4.2. The measured and computed bulk densities of the reservoir sediment are relatively close and a value of 73 lb/ft^3 is used for the bulk density of the reservoir sediment as a whole. Because the model requires the bulk density of just the cohesive sediment portion, the bulk density for the cohesive sediments was set to 68 lb/ft^3 because there is 17 % sand in the reservoir. Sand has an assumed bulk density of 99 lb/ft^3.

8.3.4. Non-Cohesive Sediment Transport Parameters

A new method to compute total bed material load was used in this study. The Ventura River contains a large range of sediment sizes, from fine sand to large boulders. Currently, no standard method exists for the computation of sediment loads in such rivers. The formula given by Parker (1990) and used by others (e.g. Andrews, 2000) is commonly accepted for the bed load and the formula of Englund-Hansen is commonly used to compute total load of sand transport. Wilcock and Crowe (2003) modified the work of Parker and others and specifically addressed the hiding function in sand-gravel mixtures. In the Ventura River, the sediment sizes range from cobbles and boulders that will travel as bed load to sands that most often travel as suspended load. Therefore, a combination of Wilcock and Crowe's model and Englund-Hansen is used to compute the transport of sediment sizes ranging from sands to cobbles. The advantage of using a single formula is that a smooth transition between bed-load and suspended load is assured. In addition, the hiding function of Parker (1990) is used to account for hiding of sand material within cobble and gravel beds.

The Englund-Hansen formula is:

$$q_s = 0.05V^2\left(\frac{d_{50}}{g(s-1)}\right)^{0.5}\left(\frac{\tau_b}{\gamma(s-1)d_{50}}\right)^{1.5} \qquad \text{Eq 8.1}$$

where q_s is the volumetric sediment transport rate per unit width, V is the cross section average velocity, τ_b is the total bed shear stress, d_{50} is the median diameter, g is the acceleration of gravity, γ is the specific weight of water, and s is the relative specific density of sediment (ρ_s/ρ). To account for mixtures and to make it applicable to sediment transport conditions near incipient motion, the sediment transport formula is rewritten for a given size class i as:

$$\frac{q_{si}g(s-1)}{(\tau_b/\rho)^{1.5}} = f_i \max\left[14, \frac{0.05V^2}{g(s-1)d_i}\right]G(\phi_i) \qquad \text{Eq 8.2}$$

where f_i is the fraction of sediment size class i in the bed, ρ is the density of water. The function G is taken from Wilcock and Crowe (2002) and is computed as:

$$G(\phi_i) = \begin{cases} \left(1 - 0.894/\phi_i^{0.5}\right)^{4.5} & \phi_i \geq 1.35 \\ 0.000143\phi_i^{7.5} & \phi_i < 1.35 \end{cases} \qquad \text{Eq 8.3}$$

The function has the behavior that as ϕ_i becomes large, $G(\phi_i)$ approaches 1. The parameter ϕ_i is computed as:

$$\phi_i = \theta_i/(\theta_c\xi_i) \qquad \text{Eq 8.4}$$

where θ_c is the critical Shield's parameter, θ_i is the Shield's parameter of the sediment size class i computed as:

$$\theta_i = \tau_b/(\gamma(s-1)d_i) \qquad \text{Eq 8.5}$$

The parameter ξ_i is the exposure factor, which accounts for the reduction in the critical shear stress for relatively large particles and the increase in the critical shear stress for relatively small particles:

$$\xi_i = (d_i/d_{50})^{-\alpha} \qquad \text{Eq 8.6}$$

where α is computed as in Wilcock and Crowe (2003):

$$\alpha = 0.67\left[1 + \exp\left(1.5 - d_i / d_m\right)\right]^{-1} \qquad \text{Eq 8.7}$$

where d_m is the mean particle diameter in the bed. The above equation has the behavior of approaching 0.67 for large d_i/d_m and approaching 0.11 for small d_i/d_m.

One parameter, θ_c, was calibrated using the available data and the value obtained was 0.04. This is near the recommended value of 0.036 given by Wilcock and Crowe (2003). The comparison between the computed and the measured values is shown in Figure 8.11 and Figure 8.12. Equation 8.2 has the behavior that it reduces to Wilcock and Crowe's bed load equation for pure bed load and reduces to Englund-Hansen's formula for pure suspended load.

As a comparison with the current method, the Corps of Engineer's program SAM (1996) was used to compute sediment loads using the same hydraulic and bed material information. The most appropriate formula in SAM was a combination of Toffaletti (1968) and Meyer-Peter and Müller (1948) formulas. The combination of Toffaletti and Meyer-Peter and Müller (Toff/MPM in the figure) formulas tends to predict more transport than the current method for material greater than 4 mm for all flow rates. Both the current method and the Toff/MPM methods predict more transport of material greater than 4 mm than the measured transport rates. The discrepancy is probably due the difficultly in measuring bed load. The large bed material is difficult to capture and the high flow velocities make sampling difficult, if not dangerous. It is assumed that the measured transport rates are lower than the actual transport rates in the river. It is difficult to determine which transport formula is best without accurate measured data and therefore different transport formulas will be used to predict the downstream impacts.

The Toff/MPM combination gives surprisingly similar results to the new method for the material finer than 4 mm. Both the new method and Toff/MPM give much high transport rates then the measured values for flows below 10,000 cfs. The reason for the discrepancy is most likely the availability of sand is much less at low flows than it is at high flows. As stated in Section 5.3 (titled "Bed Material"), the sampling of the bed was performed on point bars that are only accessed at relatively larger flows. The wet portion of the main channel may contain less sand at low flows than high flows.

It is useful to compare the equations used in the Wilcock and Crowe methodology to that of Meyer-Peter and Müller's. In revised form, the Meyer-Peter and Müller's bedload formula (1948) is:

$$\frac{q_{si} g(s-1)}{\left(\tau_b / \rho\right)^{1.5}} = 8\left[\left(\frac{K_s}{K_r}\right)^{1.5} - \frac{\theta_c}{\theta_i}\right]^{1.5} \qquad \text{Eq 8.8}$$

where,

$$\theta_c = 0.047 \qquad \text{Eq 8.9}$$

The ratio K_s/K_r is a correction to the applied shear stress so that only the grain shear stress is used to compute sediment transport rate. The values of K_s and K_r can be computed from:

$$K_s = \frac{V}{C_m R^{2/3} S^{1/2}} \qquad \text{Eq 8.10}$$

and

$$K_r = \frac{26}{d_{90}^{1/6}} \qquad \text{Eq 8.11}$$

where d_{90} = the size of sediment for which 90 percent of the material is finer than and is in meters. One can see that the Meyer-Peter and Müller formula is of similar form to the Wilcock-Crowe formulation, providing that θ_c is similar in both equations. For our case, the ratio of K_s/K_r was found to vary between 0.7 and 0.9. A comparison between the Meyer-Peter and Müller formula and Wilcock-Crowe is given in Figure 8.10. One can see that the formulations are similar, but not equivalent. The Meyer-Peter and Müller formulation generally gives a higher transport for low values of θ and less transport for high values of θ. Because there is no hiding function in Meyer-Peter and Müller, the difference between Wilcock-Crowe and Meyer-Peter and Müller depends upon the particular size class and particle distribution being modeled. However, the Wilcock and Crowe formulation has the ability to model the interactions of the grain sizes with more detail. Therefore, the formulation of Wilcock and Crowe will be used for most of the analyses presented in this document.

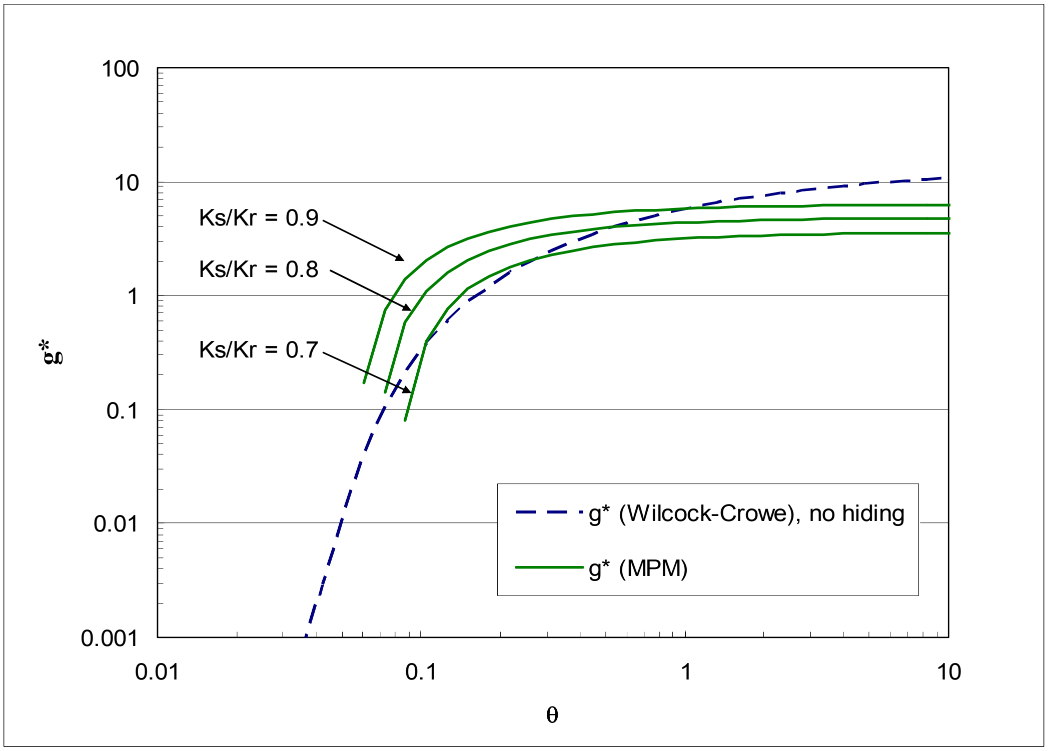


Figure 8.10. Comparison between the Wilcock and Crowe method and the Meyer-Peter-Muller Method (MPM).

The hydraulic properties used to compute the transport capacity is given in Table 8.5. The bed material is given in Section 17 Exhibit E. Ventura River Bed Material.

Table 8.5. Hydraulic properties used to compute sediment transport capacity at Foster Park.

Q ft^3/s	**A** ft^2	**T** ft	V_{mean} ft/s	**Depth** ft	**R** ft	S_f ft/ft
100	47	82	2.63	0.61	0.57	0.01199
500	114	105	4.37	1.34	1.09	0.01069
1000	215	147	5.12	1.66	1.46	0.01014
4130	640	290	6.79	2.43	2.21	0.00911
9820	1266	431	7.88	3.11	2.94	0.00839
35200	3101	625	11.28	5.28	4.96	0.00811
44400	3580	637	12.24	5.96	5.62	0.00794
56600	4304	682	12.93	6.58	6.31	0.00782

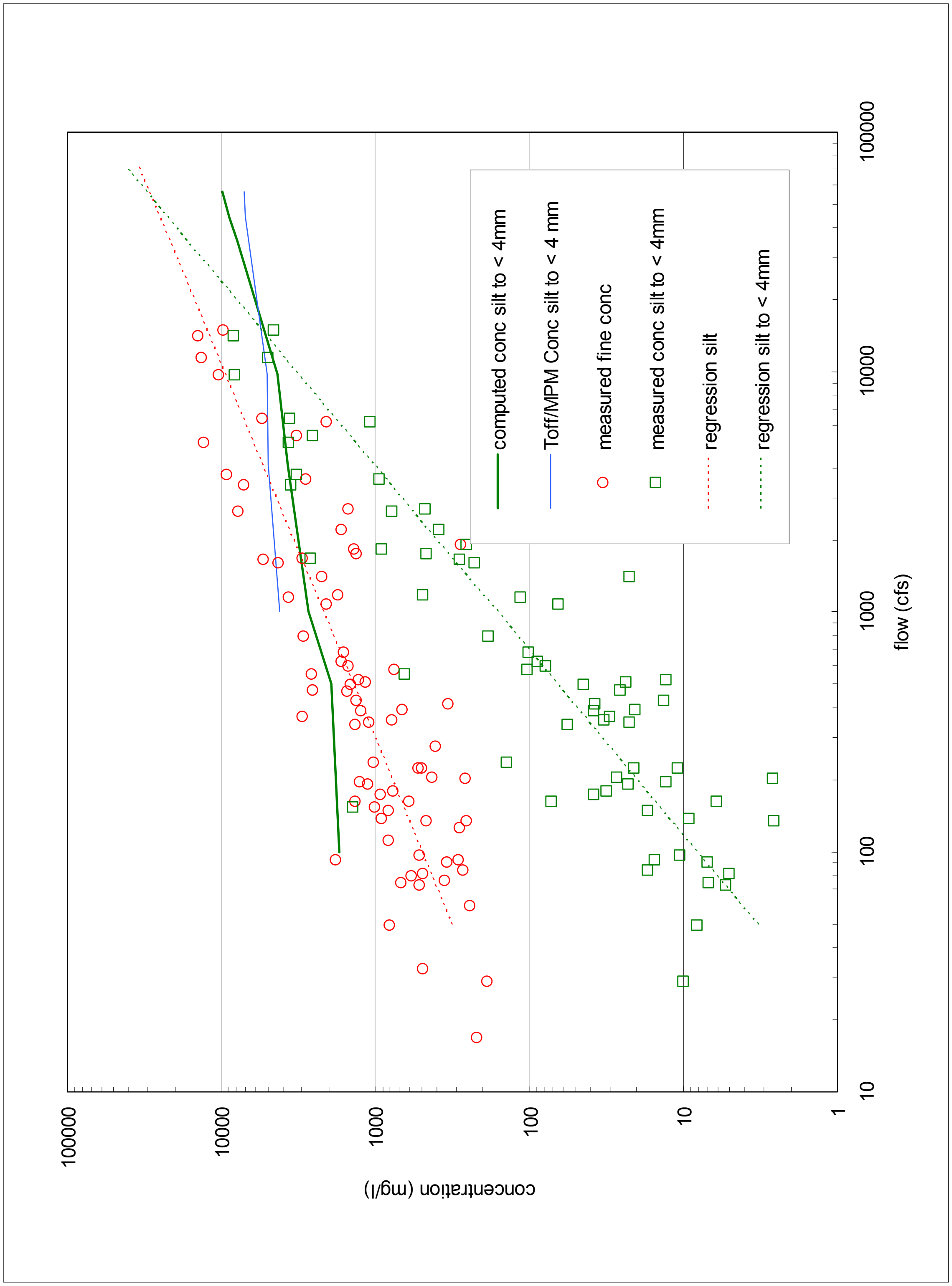


Figure 8.11. Computed and measured suspended sediment concentrations at Foster Park on the Ventura River.

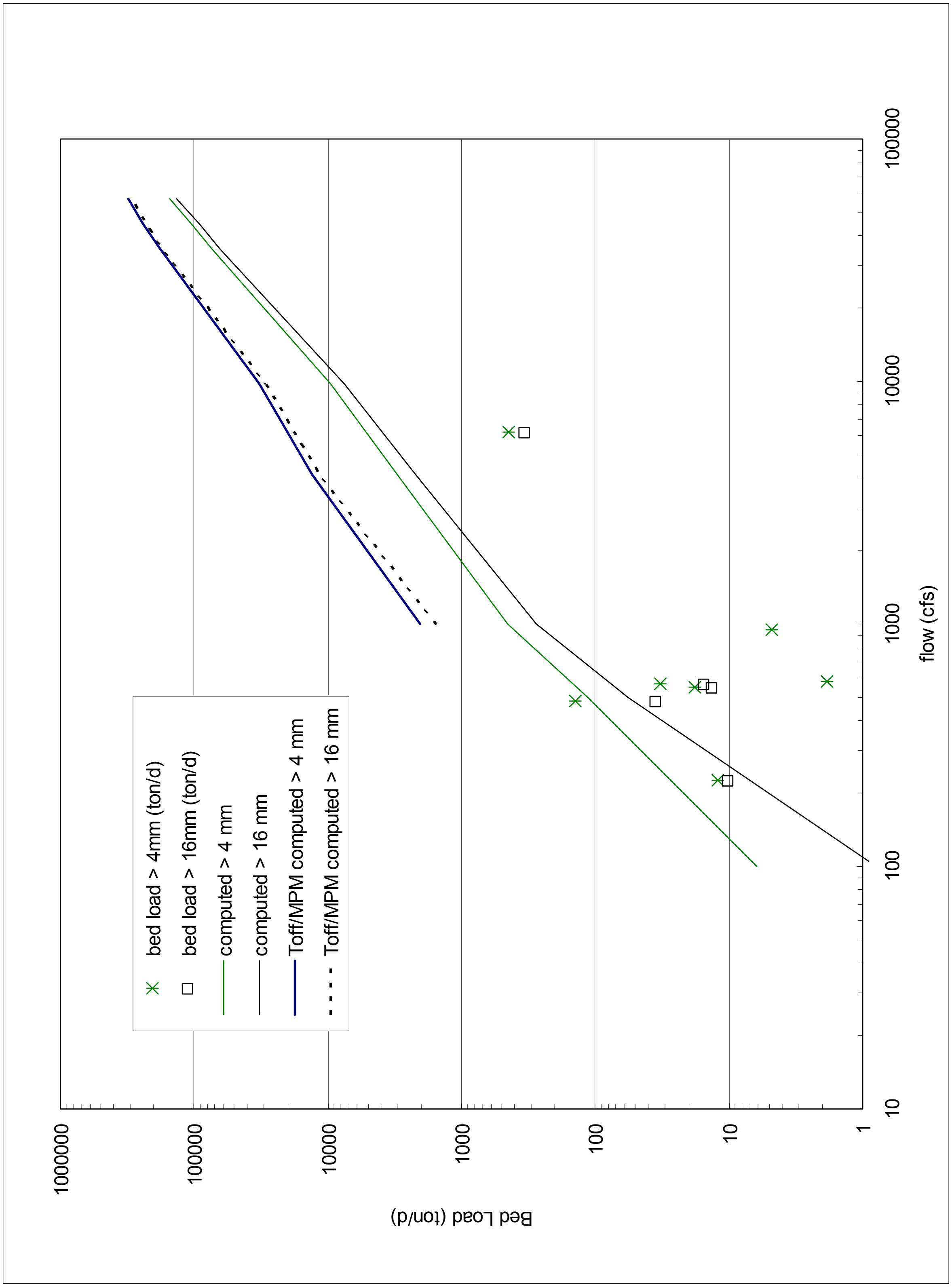


Figure 8.12. Computed and measured bed load in Ventura River at Foster Park.

8.3.5. COHESIVE SEDIMENT TRANSPORT PARAMETERS

Two main types of cohesive sediment erosion modes are found in natural systems: surface and mass erosions. Surface erosion occurs when the bed shear stress is just above critical value and is gradual, particle-by-particle erosion. At higher levels of stress, the bed shear stress exceeds the bulk shear strength of a layer of bed and that layer of bed is susceptible to mass erosion.

The GSTARS-1D model uses the following expression for the removal of cohesive material under surface erosion:

$$Q_{se} = \begin{cases} P_{se}\left(\dfrac{\tau - \tau_{se}^{c}}{\tau_{me}^{c} - \tau_{se}^{c}}\right) & \tau \geq \tau_{se}^{c} \\ 0 & \tau < \tau_{se}^{c} \end{cases} \qquad \text{Eq 8.12}$$

where Q_{se} = surface erosion rate; τ and τ_{se}^{c} = bed shear stress and critical surface erosion shear stress, respectively; P_{se} = surface erosion constant; and τ_{me}^{c} = critical mass erosion shear stress. The parameters τ_{se}^{c} and P_{se} are site-specific and need to be determined experimentally or by comparison with field data.

There is more uncertainty in estimating, P_{se}, the rate of surface erosion. Van Rijn (1990) states that published values vary by a factor of 50. A value of 0.67 lb/hr/ft^2 is used for the silts and clays in the Matilija reservoir deposits, which is among the highest reported by van Rijn (1990). The high side of the range is used because one would expect the fine material in the Matilija Reservoir to be highly erodible. The sediment in Matilija Reservoir is approximately 50% silt with 30 % clay. The silt has little cohesiveness. Assuming a high value will also give a conservative estimate for the downstream impacts in term of magnitude of sediment loads. However, it is not be conservative in terms of the duration of those impacts.

GSTARS-1D uses a similar equation for mass erosion as for surface erosion:

$$Q_{me} = P_{me}\left(\frac{\tau - \tau_{me}^{c}}{\tau_{me}^{c}}\right) + P_{se} \qquad \tau \geq \tau_{me}^{c} \qquad \text{Eq 8.13}$$

where Q_{me} = mass erosion rate; τ and τ_{me}^{c} = bed shear stress and critical mass erosion shear stress, respectively; P_{me} = mass erosion constant. Mass erosion rate usually depends on the model setup and its used time scale. The mass erosion constant, P_{me} was set to a value of 3 times the surface erosion constant.

For every simulation, the shear stresses were most often in the mass erosion range. The maximum suspended sediment concentration that was allowed was 10% by volume or 260,000 mg/l.

Deposition is also simulated for the fine sediment. The rate of deposition is computed as:

$$Q_d = P_d \omega c \qquad \text{Eq 8.14}$$

where Q_d = deposition rate and P_d = the deposition probability. The variable P_d is also the probability of particles sticking to the bed and not being re-entrained by the flow. A fraction of sediments settling to the near bed region cannot withstand the high shear stresses at the sediment-water interface and are broken up and re-suspended. The probability of deposition is given by,

$$P_d = \begin{cases} 1 - \tau / \tau_d & \tau \le \tau_d \\ 0 & \tau > \tau_d \end{cases} \qquad \text{Eq 8.15}$$

where τ = bottom shear stress; and τ_d = critical shear stress for deposition. The critical shear stress for deposition was estimated to be 0.005 lb/ft^2, which is on the higher side of the values reported by van Rijn for lake and river sediments. The shear stresses in the Ventura River are usually higher than 0.005 lb/ft^2 and therefore deposition of silt and clay does not occur.

Table 8.6. Summary of cohesive parameters used in simulations, assuming a dry bulk density of 68 lb/ft^3 (1.17g/cm3).

Parameter	Value
Bulk Density of Fines, ρ_b	68 lb/ft^3
Critical Shear Stress for deposition, τ_d	0.005 lb/ft^2
Critical Shear Stress for Surface Erosion, τ_{se}^c	0.01 lb/ft^2
P_{se}	0.67 lb/ft^2/hr
Critical Shear Stress for Mass Erosion, τ_{me}^c	0.03 lb/ft^2
P_{me}	2.0 lb/ft^2/hr
Maximum Concentration	10% (260,000 mg/l)

8.3.6. Width Adjustment in Reservoir

An important process in the erosion of reservoir sediment is the widening of the channel through the reservoir sediments. A general description of the sedimentation processes following dam removal is given by Doyle et al. (2003), which is a modification of the geomorphic head cut model of Shumm (1984). The various stages are shown in Figure 8.13 and a summary of the model of Doyle et al. follows:

Stage A. This stage is the initial conditions before dam removal. Sediment has built up behind the dam.

Stage B. The dam is removed or the reservoir is drawn down.

Stage C. This stage is characterized by rapid, primarily vertical erosion proceeding from dam to upstream. Large amount of sediments are released at this stage and the downstream concentrations will of highest of any stage. Depending upon the grain sizes present in the reservoir and the magnitude of the initial drawdown, this erosion may proceed as a head cut, or may be primarily fluvial. The erosion is not expected to cut below the original bed elevation. The initial width of the channel formed by this erosion will be governed by the stability of the material in the reservoir.

Stage D. If the incision of Stage C produces banks that are too high or too steep to be stable, channel widening will occur by means of mass wasting of banks.

Stage E. Sediment from the upstream reach starts to be supplied to the previously inundated reach. Some of this sediment is deposited in the reach as the degradation and widening processes have reduced the energy slope within the reach. Some additional widening may occur during this stage, but at a reduced rate as compared to Stage D.

Stage F. This is the final stage and is the stage of dynamic equilibrium in which net sediment deposition and erosion in the reach is near zero.

Several unique characteristics of erosion in reservoir deposits are not well represented with either one-dimensional or two-dimensional models. Some of the processes or features that are generally not well represented in sediment transport models are listed below:

- head cut migration through cohesive material
- bank erosion
- large width changes
- stratified bed sediment

Some more recently developed models have some ability to model these situations. Langendoen (2000) developed the CONCEPTS model to consider bank erosion by incorporating the fundamental physical processes responsible for bank retreat: fluvial erosion or entrainment of bank material particles by the flow, and mass bank failure, for example due to channel incision. It has not been applied to the case of dam removal, but has been applied to several rivers (Langendoen et al., 2000; Langendoen and Simon, 2000; Langendoen et al., 2001; Langendoen et al., 2002). CONCEPTS also accounts for stratified bed sediment.

MBH Software (2001) has made recent developments to the HEC-6T code to make it applicable to dam removal. In this model, the erosion width is determined by an empirical relationship between flow rate and channel width. Bank stability is modeled using a user input critical bank stability angle. If the bank becomes steeper than the input angle, the bank fails to that angle.

Stillwater Sciences has developed DREAM (Dam Removal Express Assessment Models), a model that is applicable to dam removal (Stillwater Sciences, 2002). The geometry in the reservoir is modeling assuming a simplified trapezoid shape. The user inputs the initial width and the model calculates the evolution of this channel based on transport capacity.

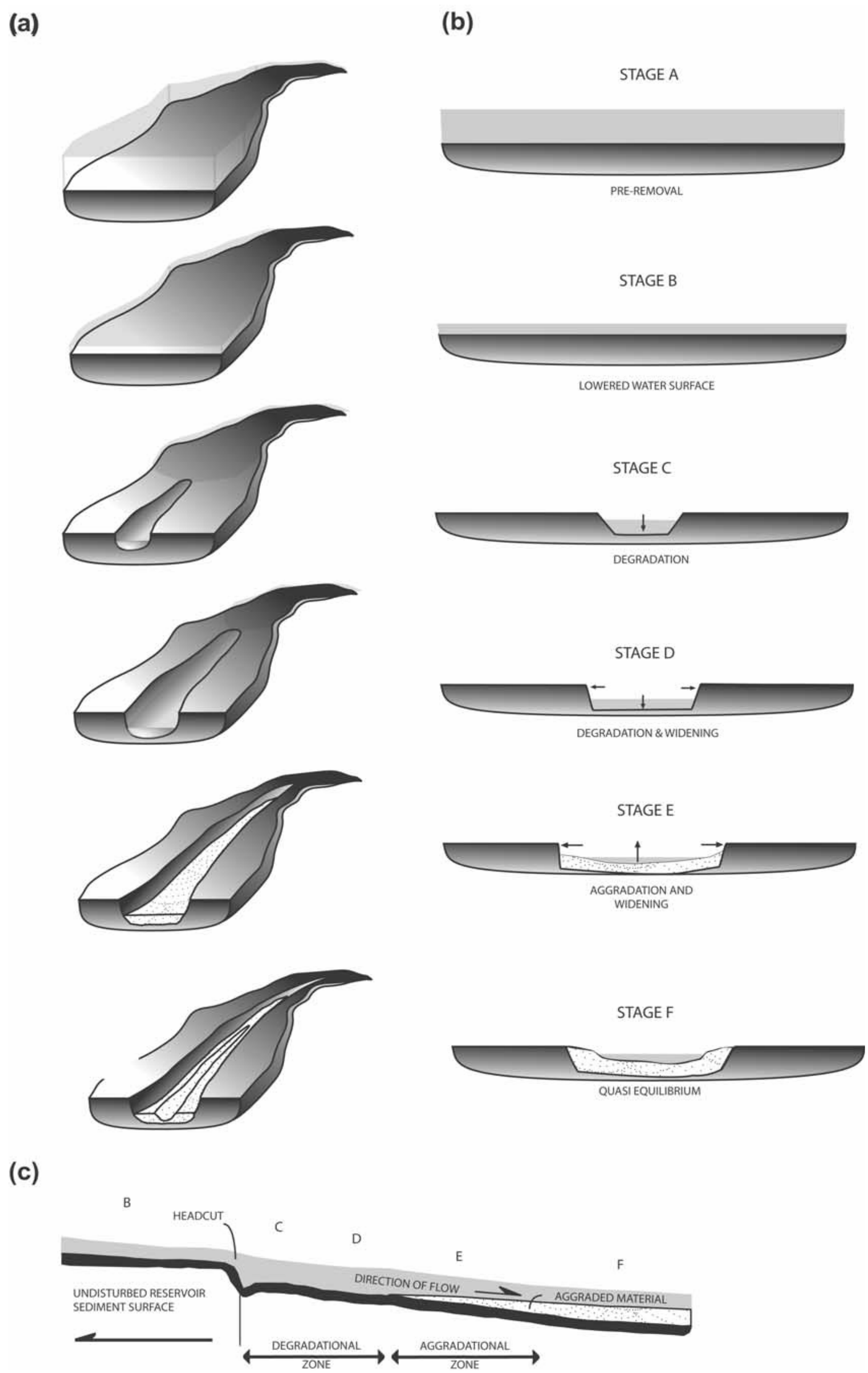


Figure 8.13. Schematic description of reservoir erosion process through delta deposits, from Doyle et al. (2003). (a) oblique view, (b) cross section view, (c) profile view.

The widening method used in the sediment transport model has important implications in the reservoir region where the channel will form through the sediments. If no such channel widening method was used the model would under-predict the amount of material removed from the reservoir. The following method is used to compute erosion in the sediment behind Matilija Dam. The cross sections in the reservoir are in fact treated in same way as the other cross sections in the downstream channel. Non-equilibrium sediment transport is also calculated in the same way as the other cross sections. The non-equilibrium sediment transport method modifies the sediment transport capacity using the following equation:

$$\frac{dQC}{dx} = \alpha B w_f \left(C^* - C\right)$$

where Q is the flow rate, C is the sediment concentration, x is the stream wise distance, α is a constant, B is the channel width, w_f is the sediment fall velocity, and C^* is the computed sediment transport capacity. The effect of using a non-equilibrium transport method is that there is a distance required to reach the transport capacity.

The only difference between the erosion predicted in the reservoir region and that predicted elsewhere is that erosional limits are placed at the pre-dam elevations. In addition, a smaller angle of repose is used to predict the bank failure. The angle of repose below water in the reservoir is set to 15 degrees and 25 degrees above water. Downstream in the river channel, the angle of repose is set to 25 degrees below water and 90 degrees above water.

To further explain the procedure for predicting reservoir erosion, at each cross section within the reservoir the following procedure is followed:

1. For each cross section subject to erosion, horizontal and vertical limits are placed on the erosion based on the pre-dam geometry and the valley walls.

2. The transport capacity of the flow is computed using the current geometry and results from the hydraulics computations. The transport capacity is adjusted based upon the non-equilibrium sediment transport method as normally implemented in GSTARS-1D. The non-equilibrium sediment transport method is derived from the method of Han (1980).

3. The transport capacity is compared against the incoming sediment load from the next cross section upstream and the difference between the two is assumed the erosion for that section.

4. The erosion volume is taken from the section. Only points that are below water or that are adjacent to points below water are allowed to erode.

5. Once the vertical limits are reached, material is taken from the points nearest the main channel to satisfy the transport capacity.

6. The bank erosion rate is limited by the relative length of the cross section that is wet and above the vertical limit.

As an example of the widening process, the 100-yr flood is modeled two times in succession and the results are shown in Figure 8.14 to Figure 8.16. The model predicts the first flood erodes to the pre-dam thalweg elevations through most of the delta region (Figure 8.14). It takes two 100-yr floods to reach the pre-dam bed elevations in the remaining upstream portions of sediment deposit. Examples of the changes to the cross section shape are shown in Figure 8.15 and Figure 8.16. First, the erosion occurs vertically until the pre-dam elevations are reach, then the cross section widens slowly. The width of the initial incision is expected to be approximately 200 to 400 feet, and this is the range of the widths predicted by GSTARS-1D.

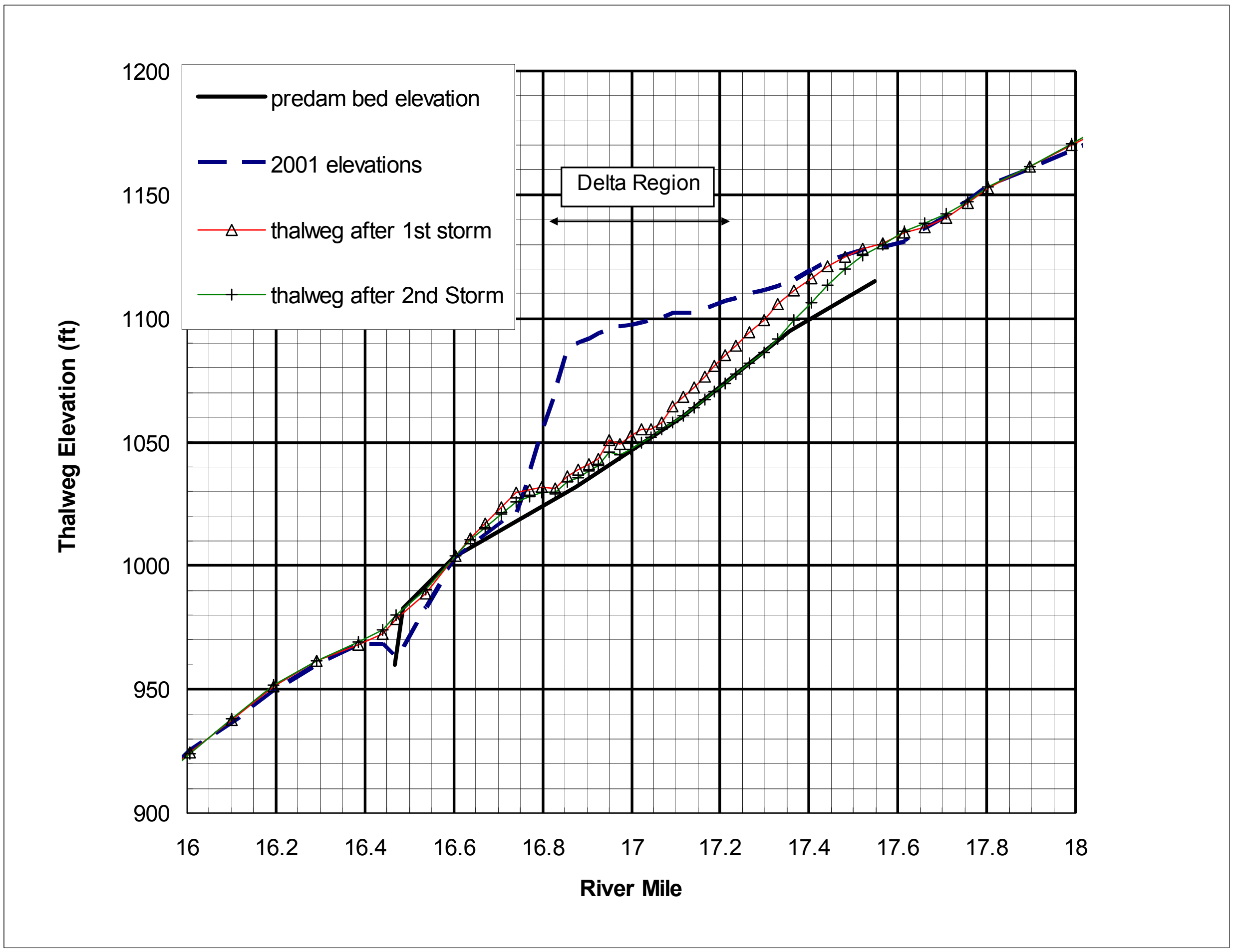


Figure 8.14. Thalweg elevations through reservoir region for Alternative 2a after the simulation of the 1998 flood twice in succession.

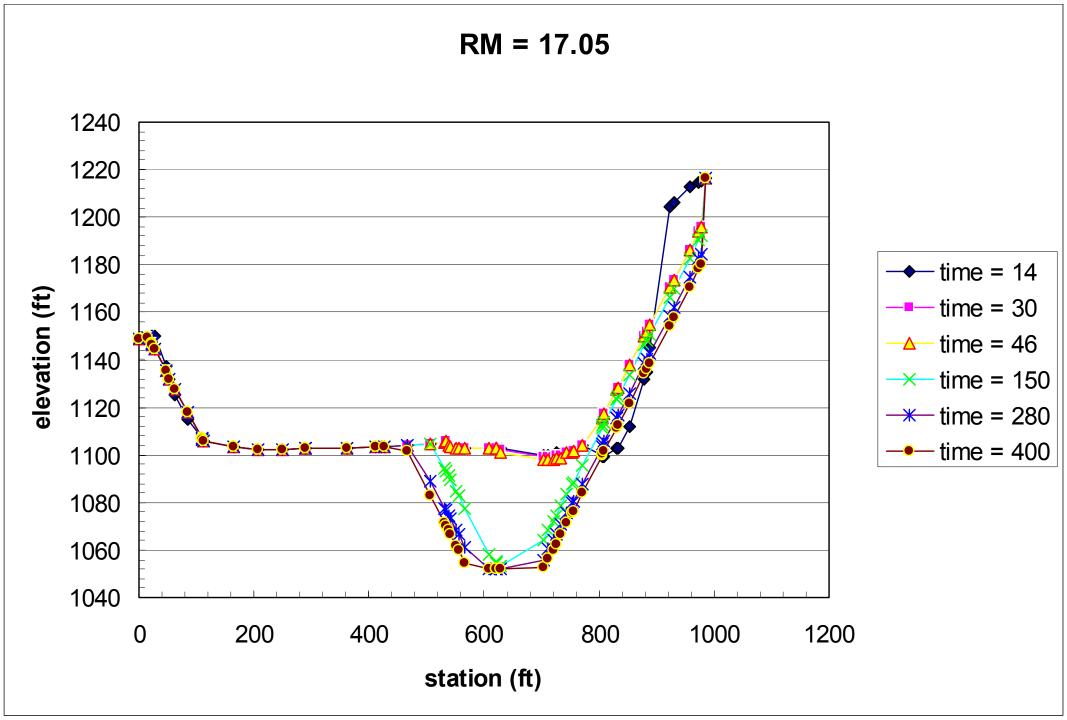


Figure 8.15. Example of Cross Section at Reservoir Delta for Alternative 2a for Two 100-yr Floods in Succession.

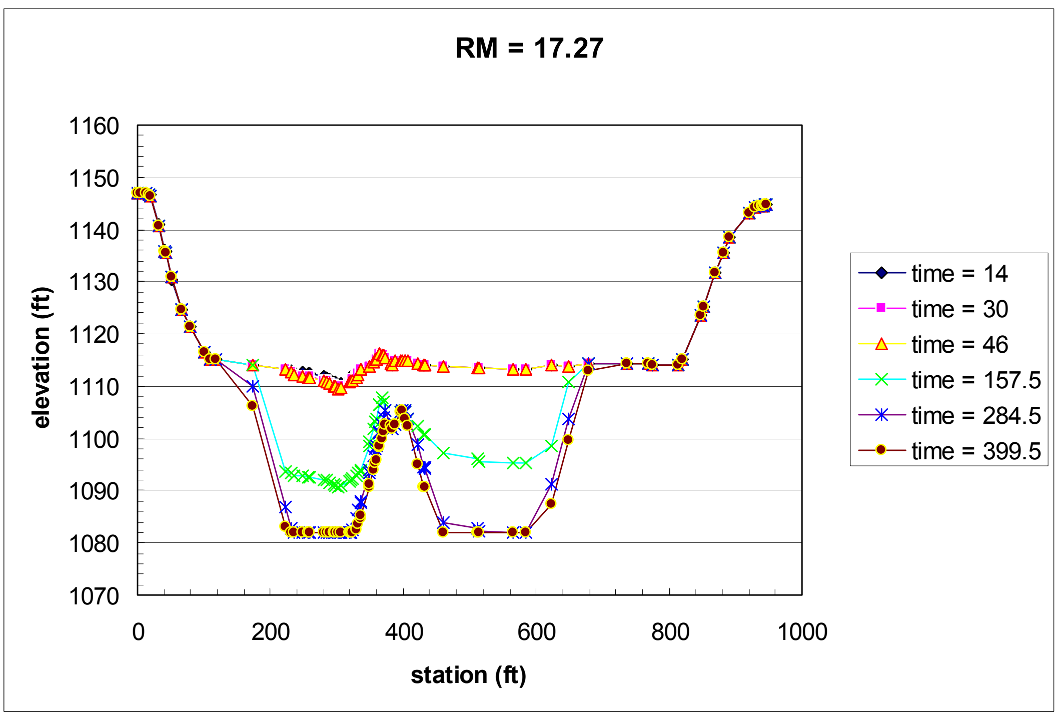


Figure 8.16. Example of Cross Section in Upstream Delta for Alternative 2a for Two 100-yr Floods in Succession.

8.4. Changes to GSTARS-1D

Throughout the course of the study, several changes were necessary to improve the model results. Initial results of the model were updated and this report contains only the most recent results.

1. Calibration of sediment transport formula for bed load: Initially, the transport function was calibrated to measured bed-load samples of Hill and McConaughy (1980). However, it was later realized that the bed-load samples were most likely not representative of true bed-load movement in the Ventura River. As a result, the critical shear stress decreased from 0.06 to 0.04. This had a large effect on results. The largest effect was to increase the deposition in the upper reaches and to decrease the deposition in the lower reaches.

2. Calibration of sediment transport formula for suspended load: The original sediment transport function had an effective sediment-hiding coefficient of 0.77 for the sand sized material. The hiding function was matched to existing data from the Ventura River. However, it was later realized that the reason for the marked decrease in sand concentration at lower flows was that the availability of sand is much lower at the low flows than the high flows. As the flow rate increases, the water inundates higher portions of the cross section where the sand sized sediment is a larger percentage of the bed material. To improve the prediction of the sand transport capacity the Wilcock and Crowe (2003) hiding function was used instead of the Parker (1980) hiding function. The Wilcock and Crowe (2003) was an effective hiding coefficient of 0.11 for sand sized material. The effect of lowering the hiding coefficient was to increase the transport capacity of sand in the reaches downstream of Matilija Dam. This caused less deposition in these reaches and carried more sediment downstream to the lower reaches, including the estuary.

3. Changes to the upstream sediment load in Matilija Creek. The original model used the sediment transport formula to predict the amount of sediment entering Matilija Creek upstream of the reservoir. The incoming sediment load was revised so that the correct volume of sediment deposited behind Matilija Reservoir. The revised sediment loads were significantly higher than the original.

4. Debugging of the reservoir erosion scheme: The specific numerical methods used to predict the erosion of sediment from the reservoir were developed for this project and therefore some debugging of the scheme was necessary. The initial model did not conserve mass when the banks eroded laterally. The result of this is that less sediment was introduced into the downstream river channel than was actually eroded behind Matilija Dam. The result was that less sediment was carried to the downstream reaches. The present model is now mass conserving.

5. Inclusion of Levees in the Hawthorne Acres Reach: The original model did not have levees in the Hawthorne Acres Reach (the reach immediately downstream of Robles Diversion Dam). After including the effect of levees in this reach, the deposition was limited to a smaller width. This had the effect of increasing some of the depths of deposition.

8.5. Testing of GSTARS-1D using Historical Data

The Corps of Engineers completed a survey of the entire Ventura River stream channel in 1970 (USCOE, 1971). To improve the accuracy of GSTARS-1D, the period from 1971 to 2001 was simulated and compared against the measured cross section data from 2001. There was little flow during the period from 1970 to 1971 so the cross section data was essentially unchanged during this period.

The hydrology was taken from the daily average stream gage records from 1971 to 2001. The hydrograph was modified to account for the peak flows that are much larger than the daily average flows. A triangular hydrograph was used where the volume of flow for that day is conserved and the measured peak flow is the apex of the triangle. An example of the constructed hydrograph for one flood is shown in Figure 8.17. The duration of the constructed hydrograph is given by the following equation,

$$\Delta t = \frac{2Q_2 - (Q_1 + Q_3)}{Q_p - \frac{1}{2}(Q_1 + Q_3)}$$

The North Fork Matilija Creek and San Antonio Creek were simulated in a similar manner.

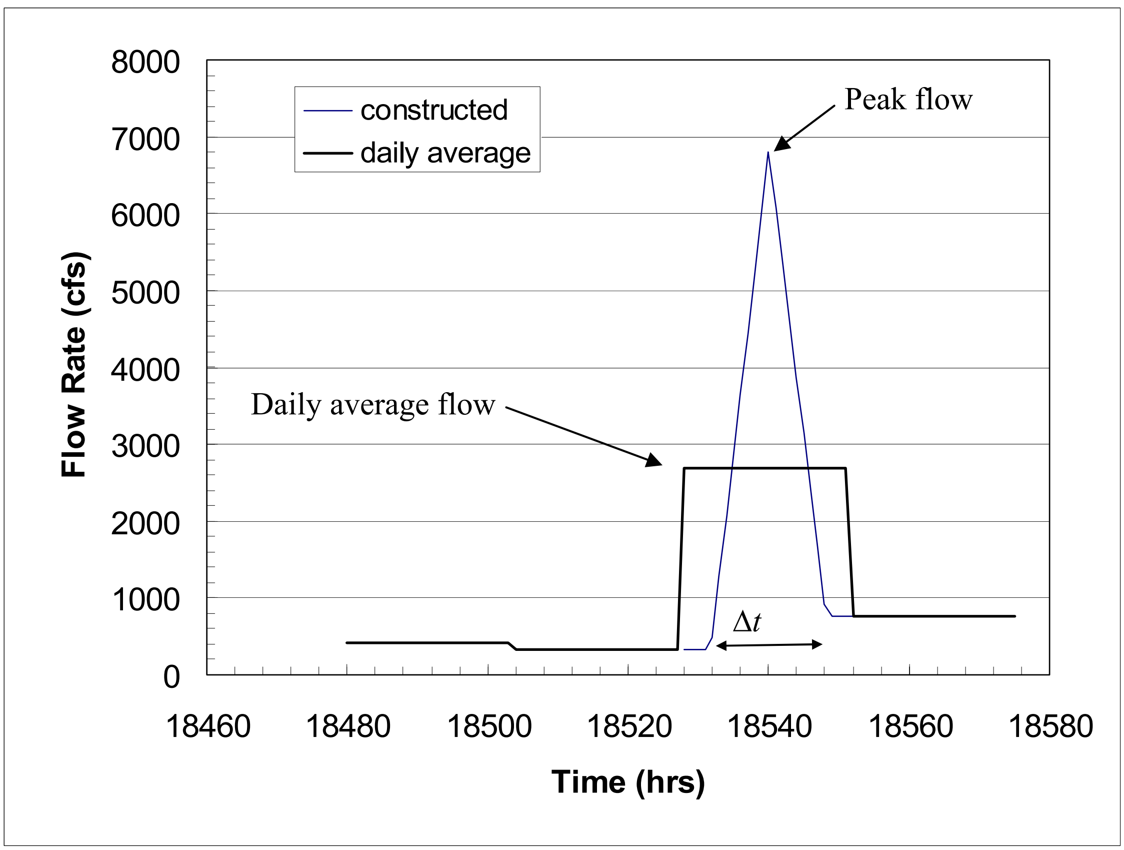


Figure 8.17. Example of Constructed Hydrograph.

The sediment rating curves for the incoming and tributary sediment loads were assumed the same as was used for the alternative predictions. The same transport formula was used and same non-cohesive sediment transport parameters were used as well. The hydraulic roughness is the same used in the alternative analysis as well. No bed material data is available from 1970 and therefore it is assumed that the bed material is similar what was sampled in 2002. However, because some reaches have degraded, these reaches may have had finer sediment in 1970 then than they do today. Another difficulty in the comparison is that the 2001 survey was done at a much higher resolution. The 1970 cross sections were obtained from a 2-foot contour interval topomap with a stated accuracy of +/- 2.5 feet. Therefore, changes of less than 2.5 feet between 1970 and 2001 should be considered not significant.

The operations at Robles Diversion were not accounted for in the model. CMWD presently removes sediment from behind Robles Diversion (Table 1.4) and this would affect the sediment transport immediately downstream and upstream of the diversion. The model does represent the filling of material behind Robles Diversion, but it allows the sediment to start going over the top when the basin fills. Therefore, it may overestimate the amount of sediment travel downstream over long simulations. Further work could be done to model the removal of sediment at this location, but at this stage in the analysis, this was not done. It is recommended that the removal of sediment behind Robles be accounted for in future work. However, a current mitigation measure for each alternative is to install a sediment bypass structure and therefore the amount of relative amount of sediment being removed at Robles will decrease.

A comparison between the calculated and measured changes in the thalweg elevations is shown in Figure 8.18 and a comparison between the calculated and measured thalweg profile is shown in Figure 8.19. A comparison between cross section data is given in Figure 8.20. In general, there were two reaches with significant erosion during the period from 1971 to 2001. One reach was from just below Robles Diversion Dam (RM 14) to approximately RM 13. The other reach that degraded extended from Foster Park to the Ventura Levee (RM 6 to RM 2). The model predicts that both of these reaches will degrade. However, there are some discrepancies. From RM 14 to RM 13.5 there is less erosion predicted than actually occurred. There could be several reasons for the discrepancy. One is that the model does not account for the removal of sediment at Robles by CMWD. The diversion is accounted for in the model, but sediment is allowed to go over the top of the diversion after the diversion fills with sediment. Another reason for the discrepancy could be that the sediment was finer in 1970 in this reach. From RM 13.5 to RM 13, erosion is generally well predicted based on the profile and cross section comparison.

The erosion in the lower reach begins at approximately RM 5.5; however, the model predicts that the erosion begins at approximately RM 5. From RM 5 to RM 4, the erosion is well predicted, but the model does not predict erosion continuing beyond RM 4. Changes in bed material between 1970 and 2002 could account for this discrepancy. Additional simulations and geomorphic analysis is being conducted to investigate the cause of the erosion downstream of RM 5.

A comparison of erosion volumes is not possible because the exact location of the 1970 cross sections is not known.

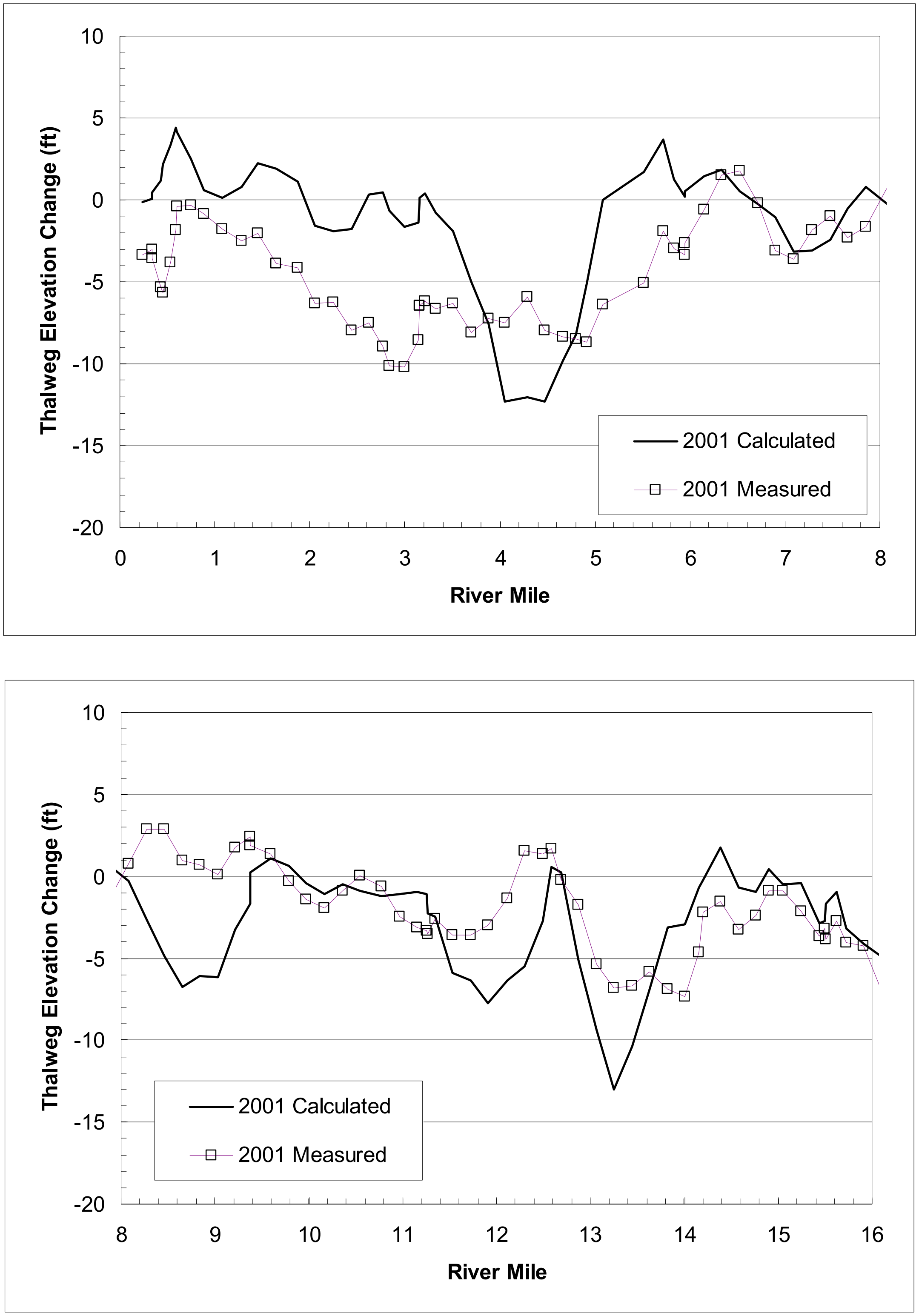
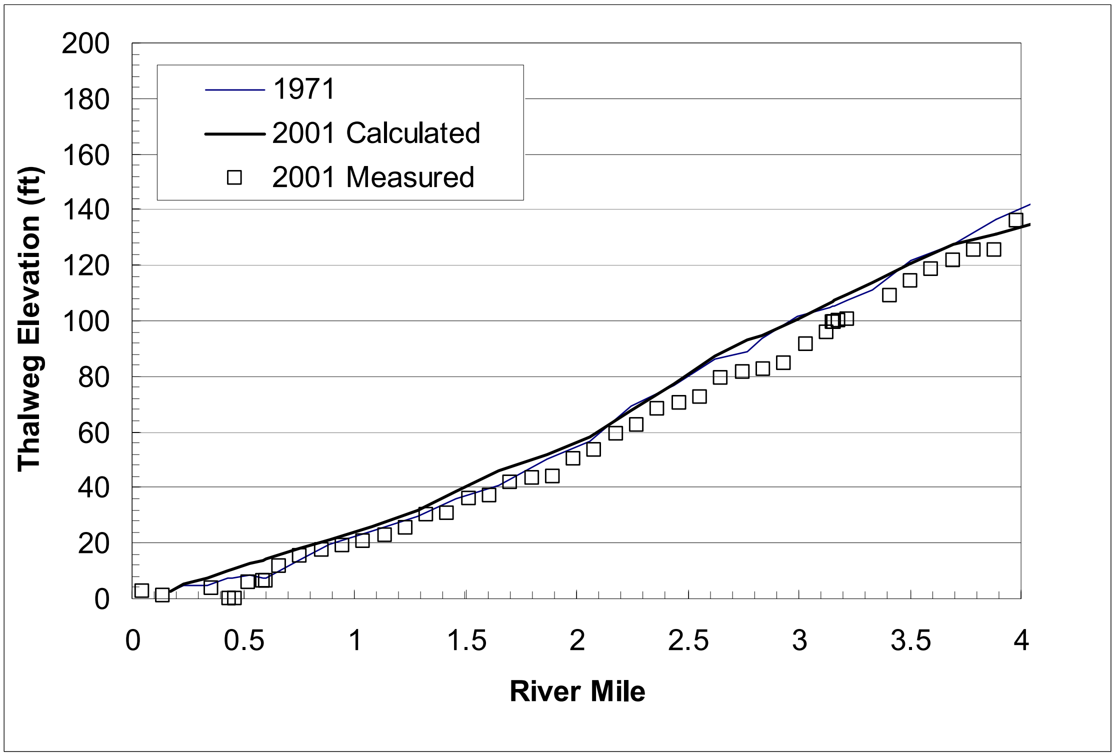
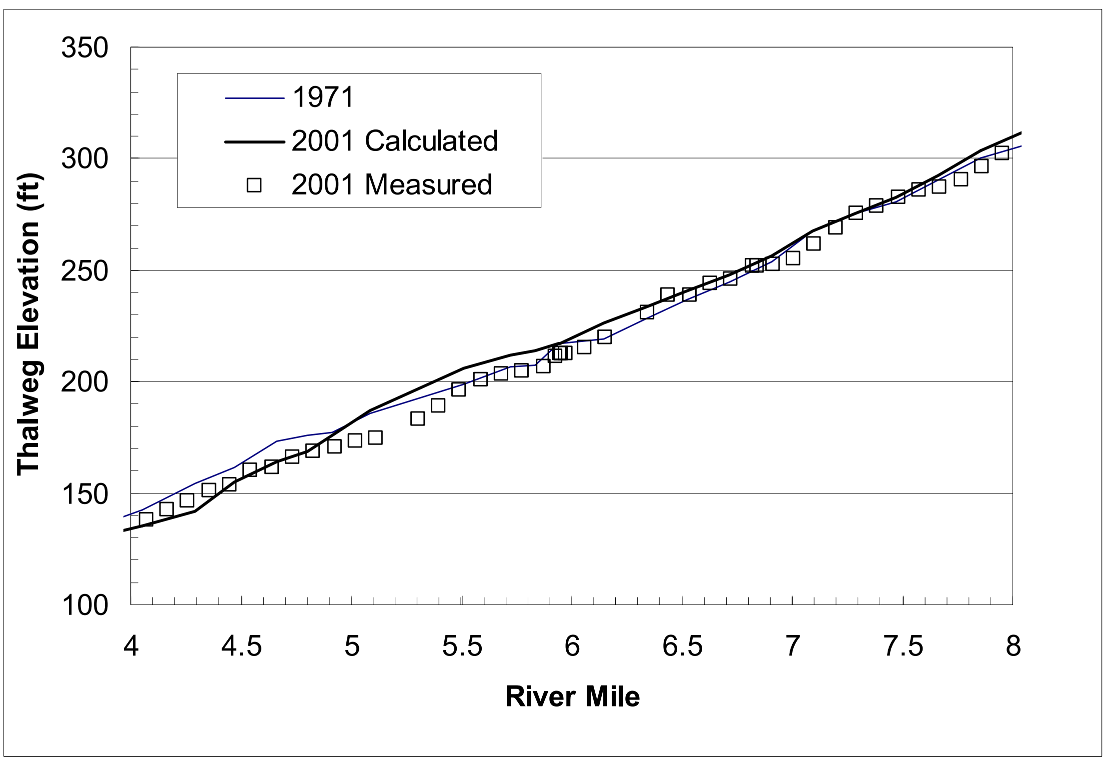


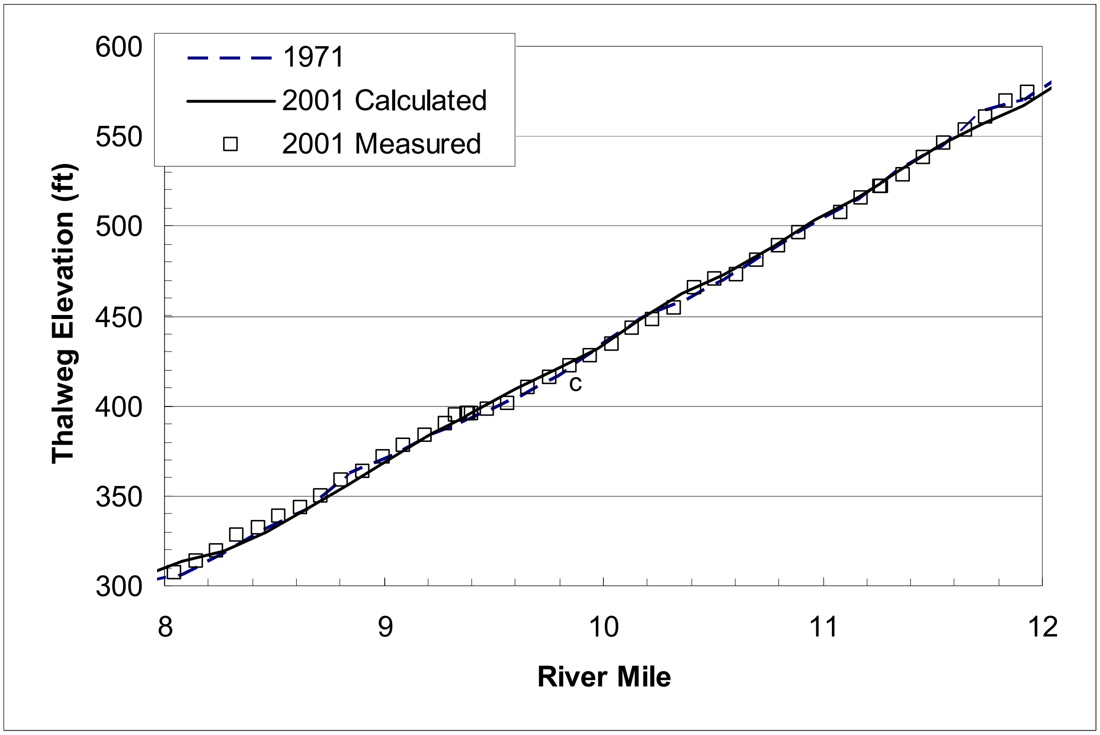
Figure 8.18. Comparison of measured and calculated changes in thalweg elevation for the simulated period from 1971 to 2001.



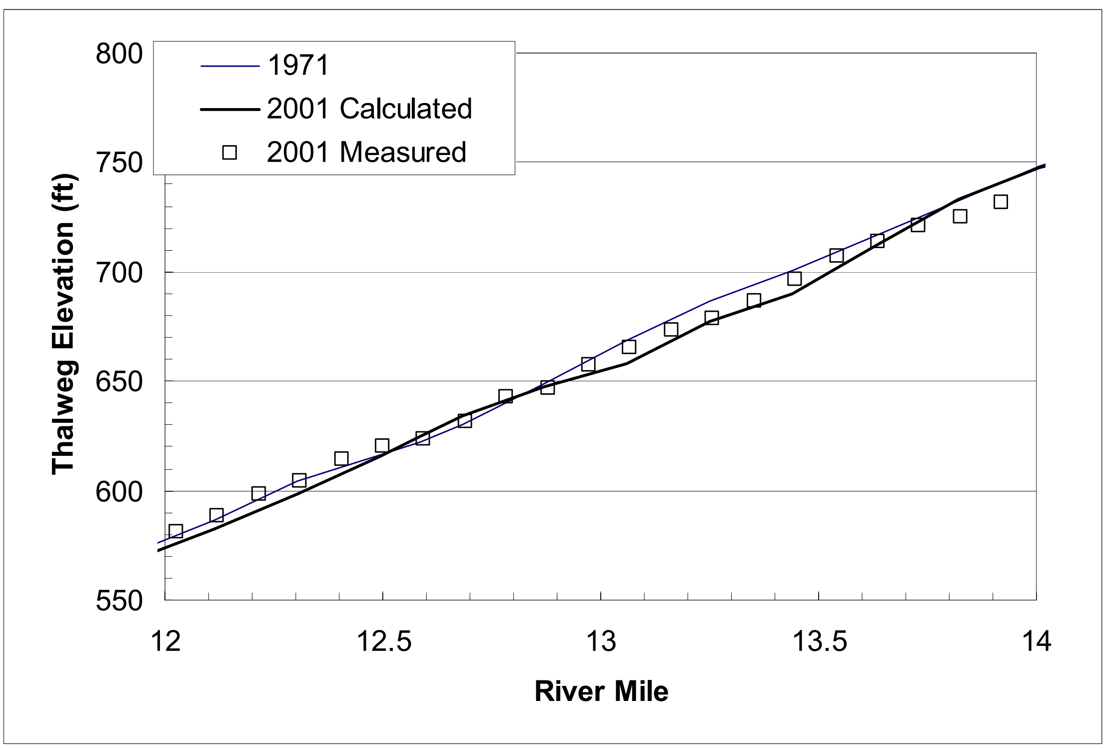
1971
2001 Calculated
2001 Measured
Thalweg Elevation (ft)
River Mile
0
20
40
60
80
100
120
140
160
180
200
0
0.5
1
1.5
2
2.5
3
3.5
4



1971
2001 Calculated
2001 Measured
Thalweg Elevation (ft)
River Mile
100
150
200
250
300
350
4
4.5
5
5.5
6
6.5
7
7.5
8



1971
2001 Calculated
2001 Measured
Thalweg Elevation (ft)
600
550
500
450
400
350
300
8
9
10
11
12
River Mile
c



1971
2001 Calculated
2001 Measured
Thalweg Elevation (ft)
800
750
700
650
600
550
12
12.5
13
13.5
14
River Mile

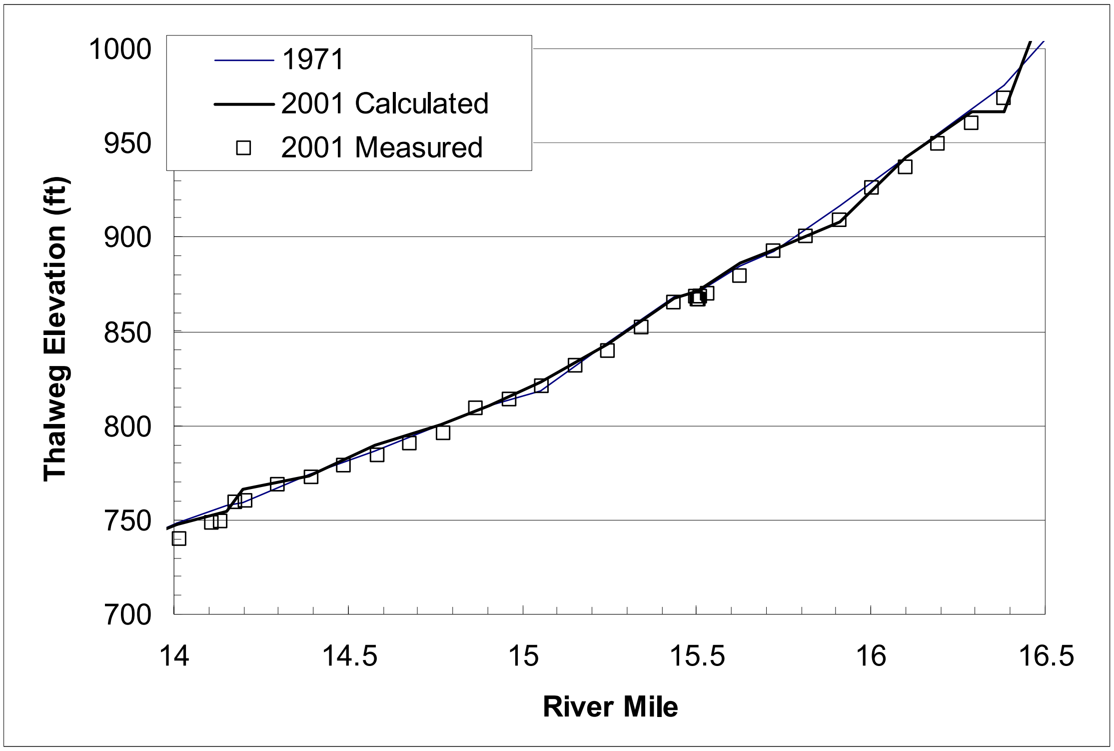
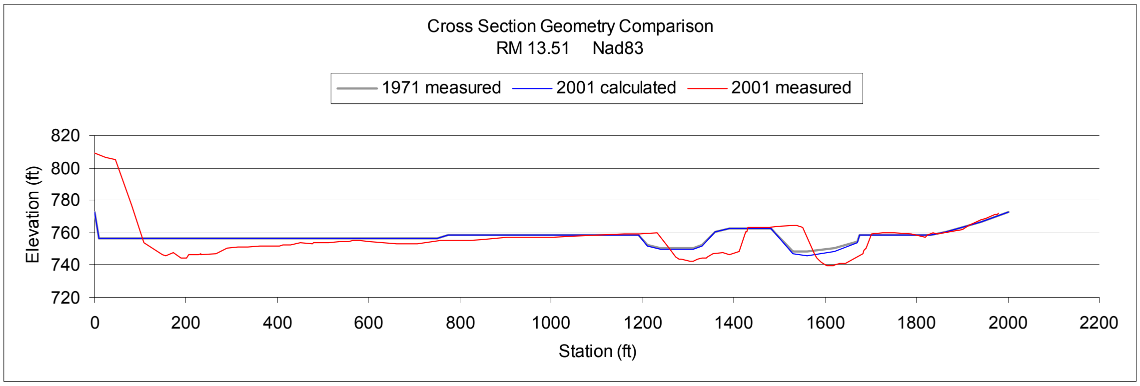
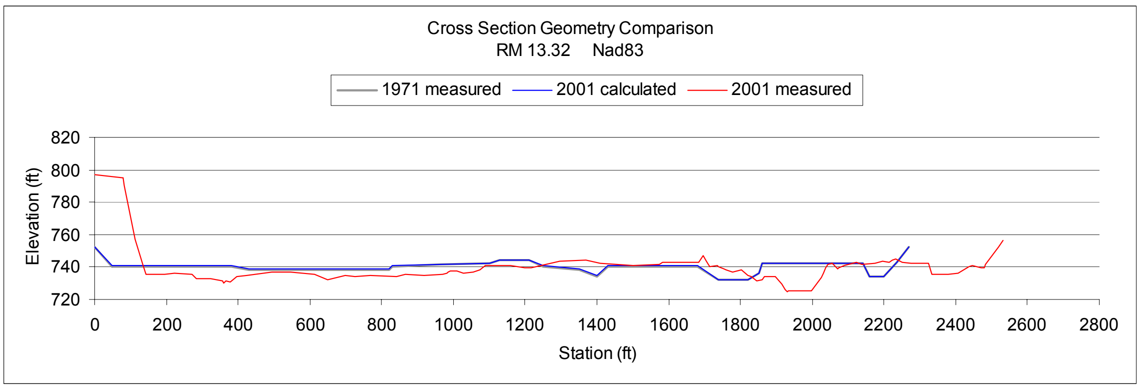
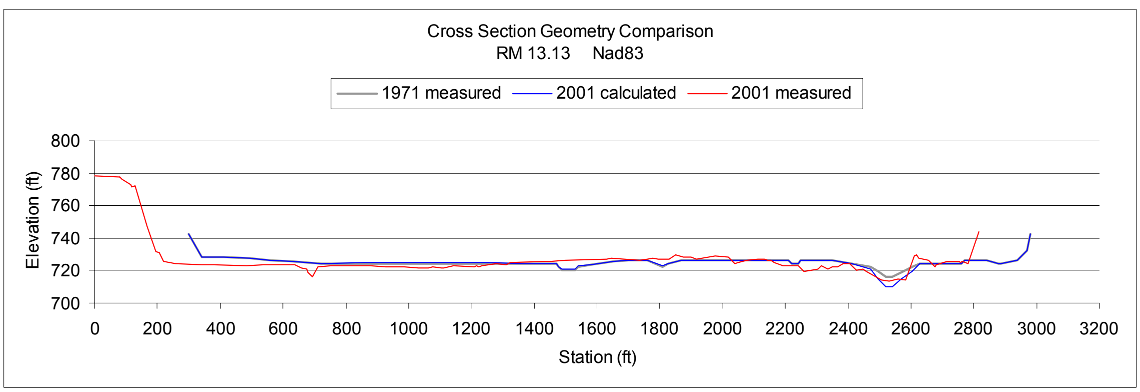
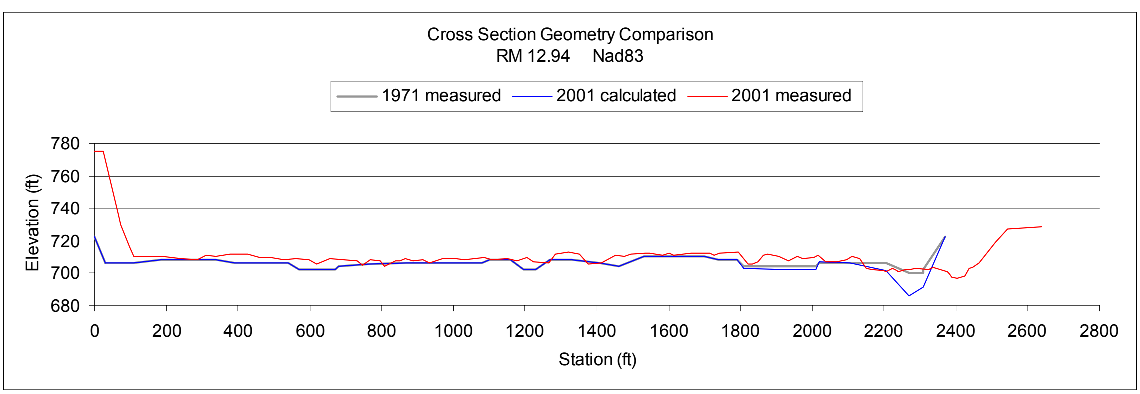


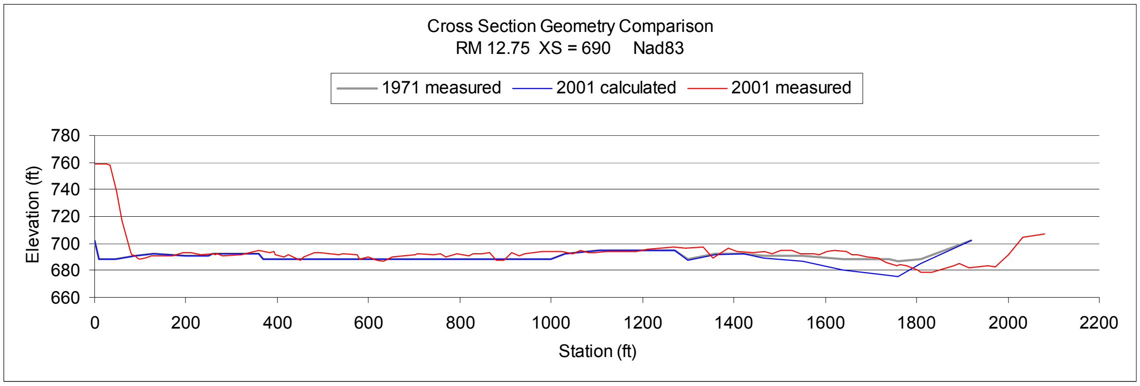
Figure 8.19. Comparison of measured and calculated thalweg profiles for the simulated period from 1971 to 2001.

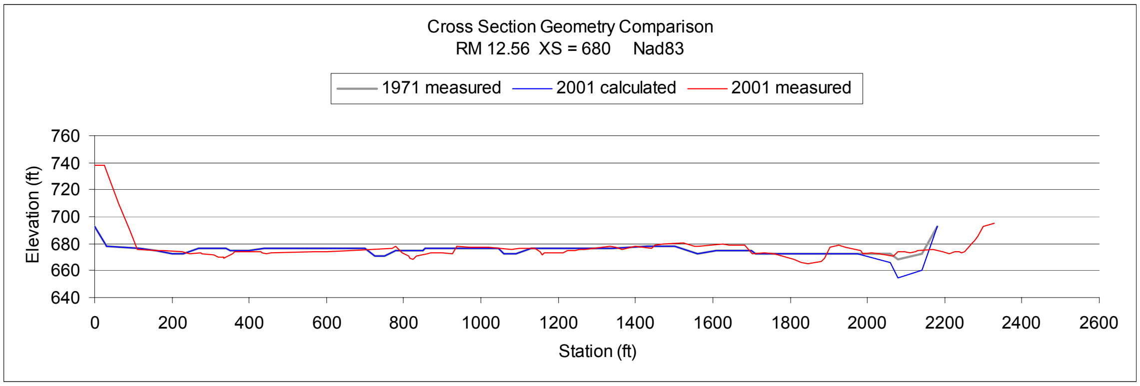

Cross Section Geometry Comparison
RM 13.51 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
820
800
780
760
740
720
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
Station (ft)

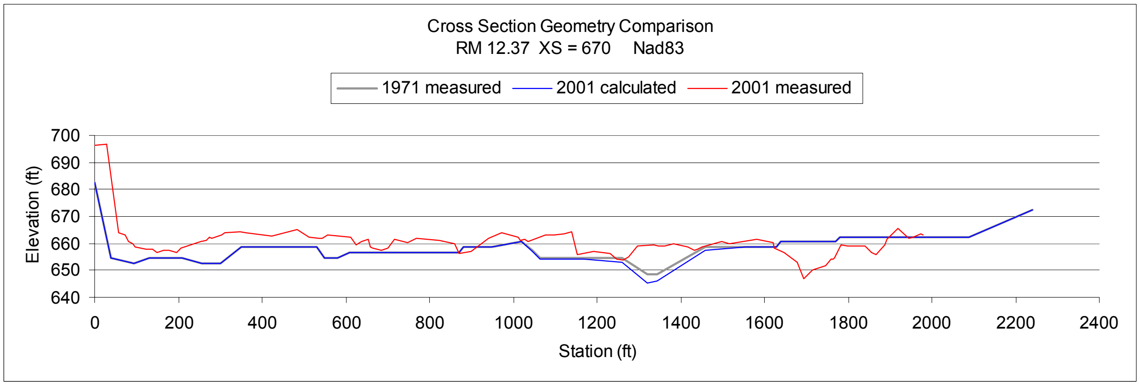

Cross Section Geometry Comparison
RM 13.32 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
820
800
780
760
740
720
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
Station (ft)

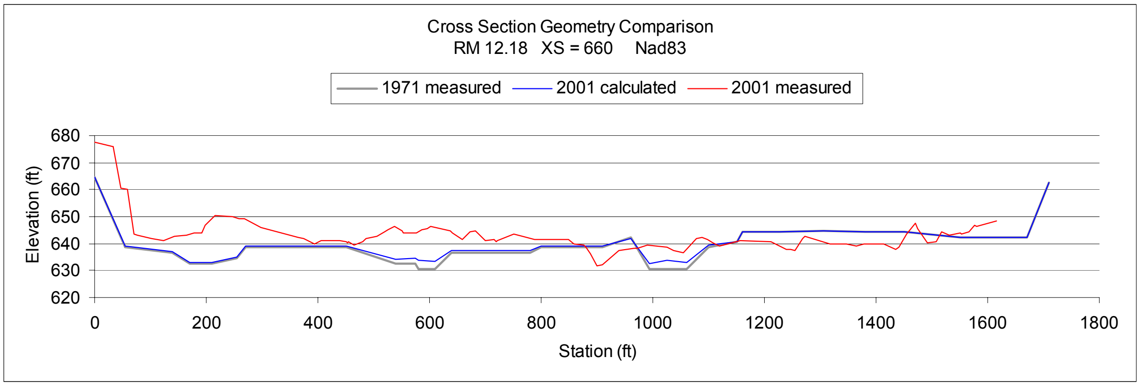

Cross Section Geometry Comparison
RM 13.13 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
800
780
760
740
720
700
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200
Station (ft)

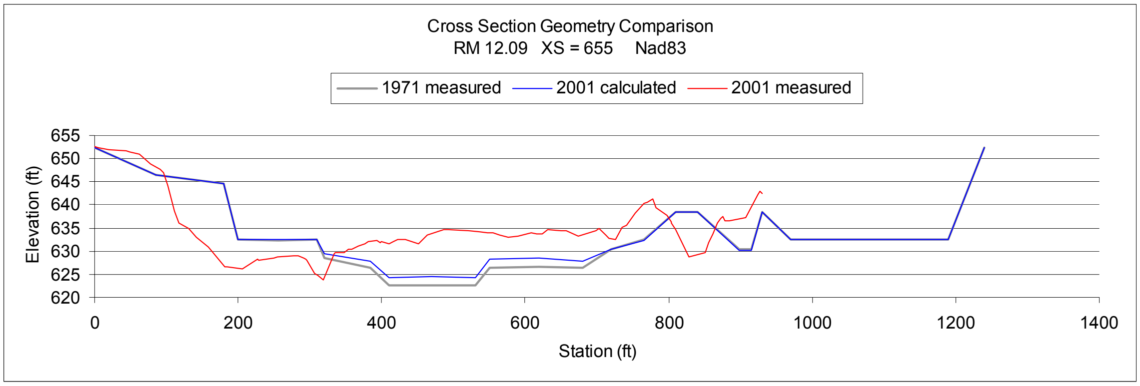

Cross Section Geometry Comparison
RM 12.94 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
780
760
740
720
700
680
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
Station (ft)

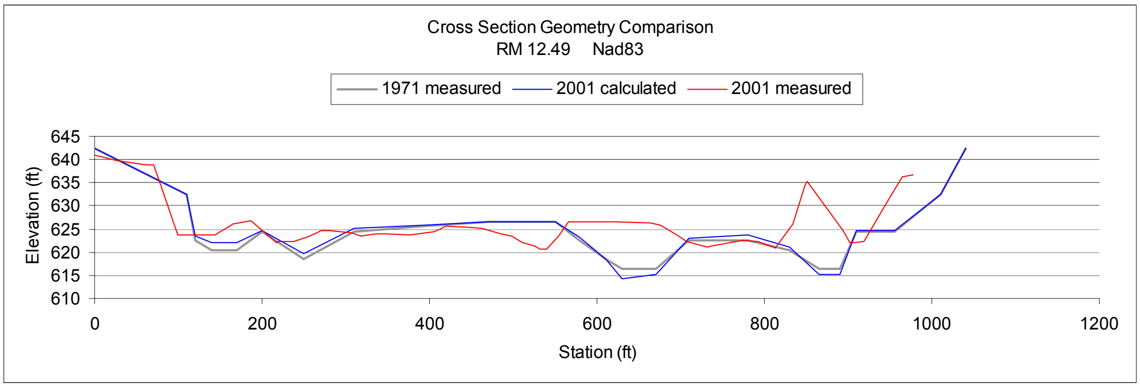

Cross Section Geometry Comparison
RM 12.75 XS = 690 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
Station (ft)

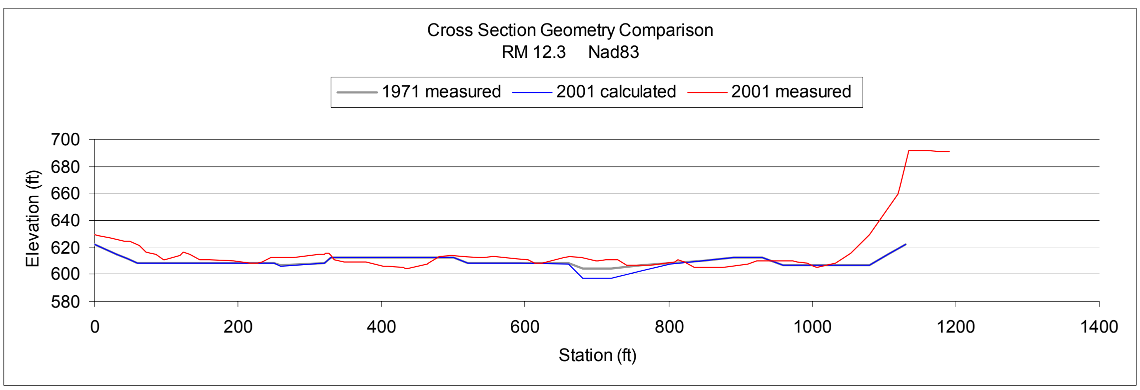

Cross Section Geometry Comparison
RM 12.56 XS = 680 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
Station (ft)

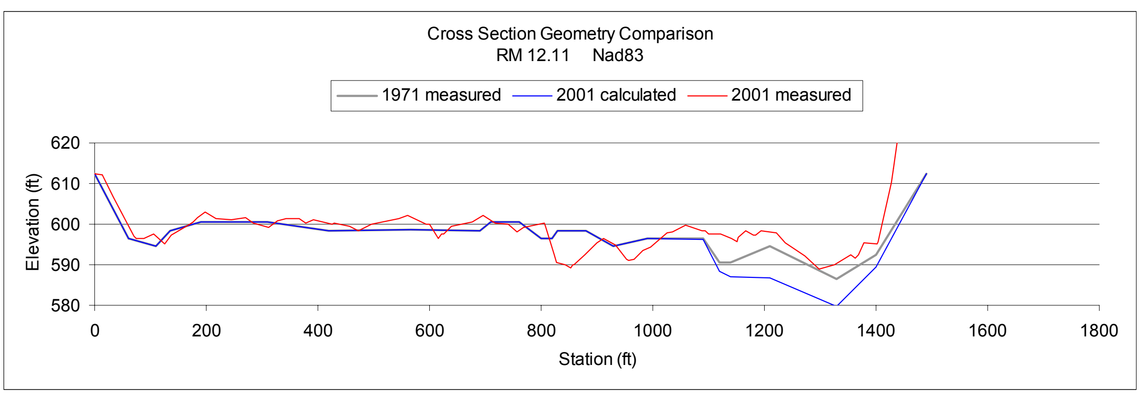

Cross Section Geometry Comparison
RM 12.37 XS = 670 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
Station (ft)

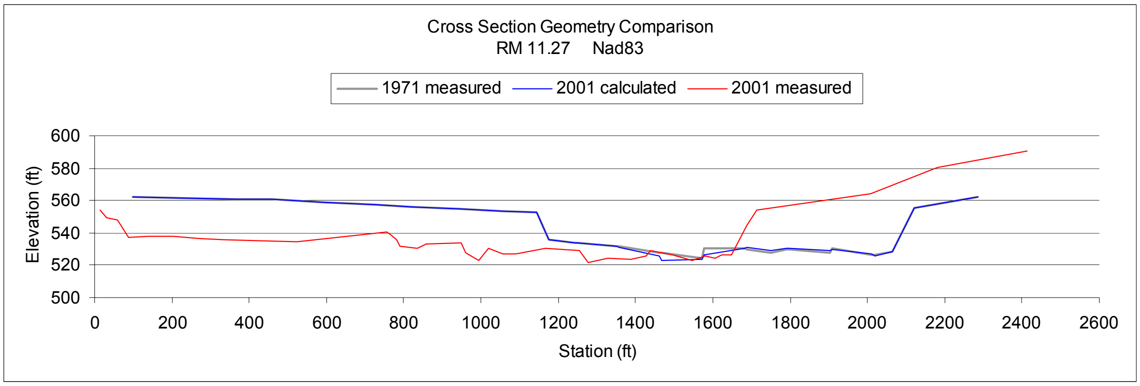

Cross Section Geometry Comparison
RM 12.18 XS = 660 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
Station (ft)

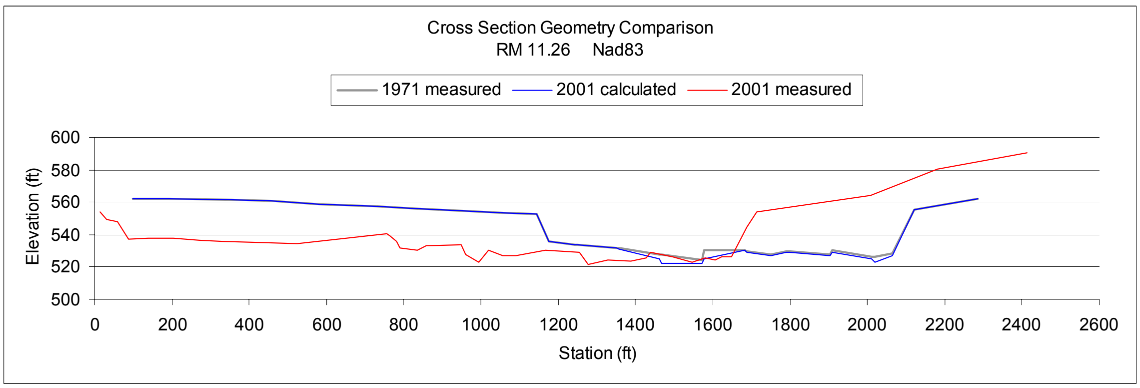

Cross Section Geometry Comparison
RM 12.09 XS = 655 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
655
650
645
640
635
630
625
620
0
200
400
600
800
1000
1200
1400
Station (ft)

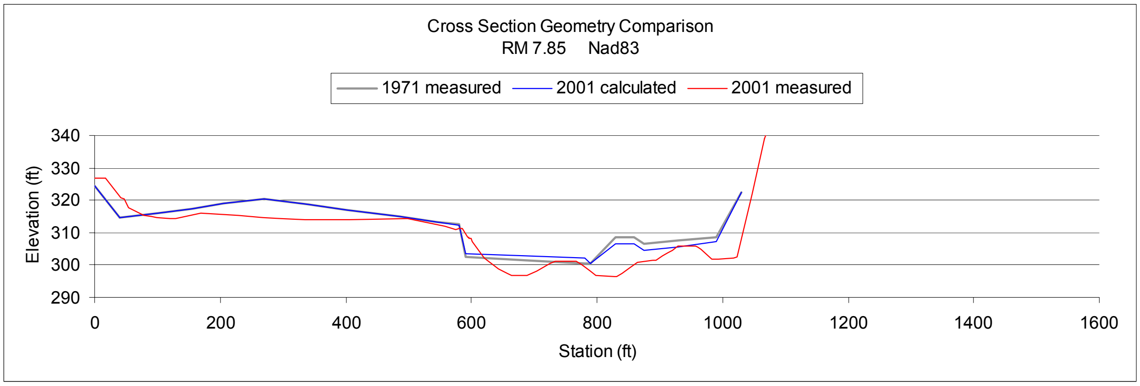

Cross Section Geometry Comparison
RM 12.49 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
645
640
635
630
625
620
615
610
0
200
400
600
800
1000
1200
Station (ft)

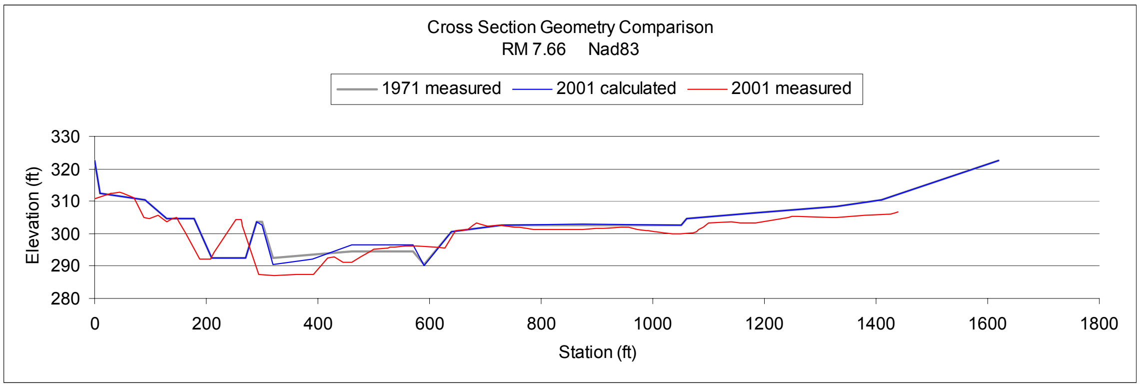

Cross Section Geometry Comparison
RM 12.3 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
700
680
660
640
620
600
580
0
200
400
600
800
1000
1200
1400
Station (ft)

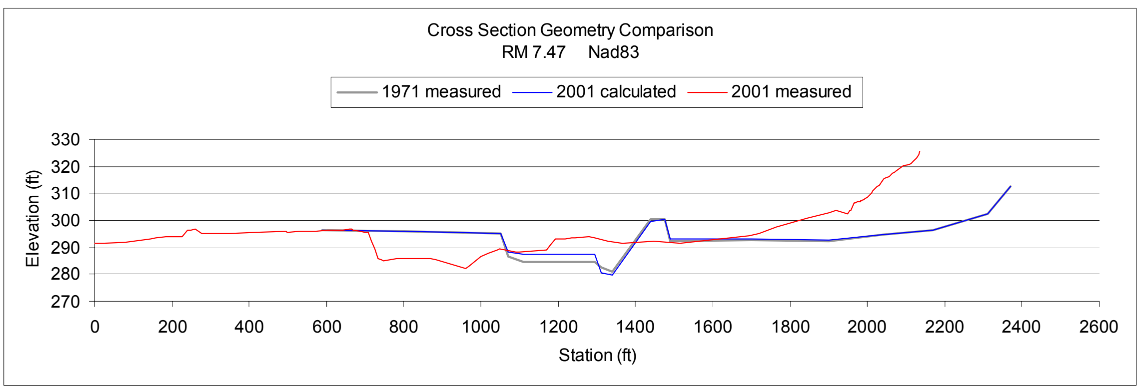

Cross Section Geometry Comparison
RM 12.11 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
620
610
600
590
580
0
200
400
600
800
1000
1200
1400
1600
1800
Station (ft)

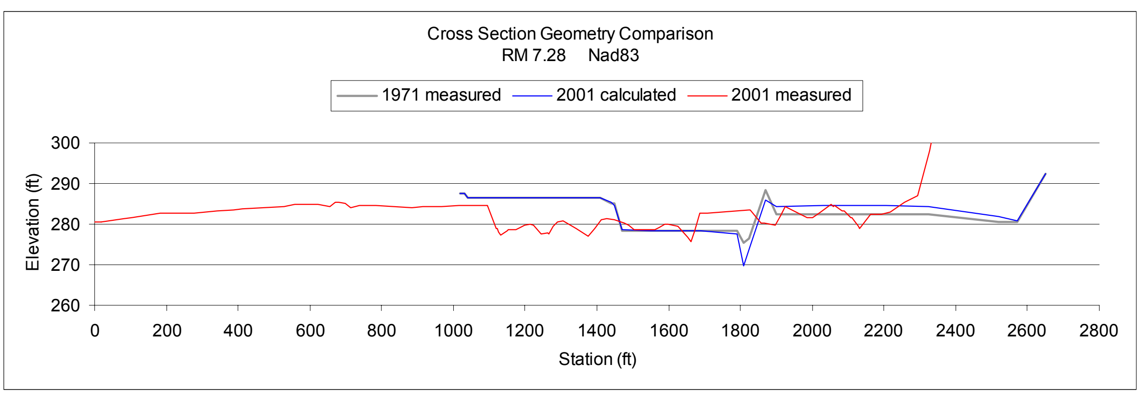

Cross Section Geometry Comparison
RM 11.27 Nad83
1971 measured
2001 calculated
2001 measured
600
580
560
540
520
500
Elevation (ft)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
Station (ft)


Cross Section Geometry Comparison
RM 11.26 Nad83
1971 measured
2001 calculated
2001 measured
600
580
560
540
520
500
Elevation (ft)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
Station (ft)


Cross Section Geometry Comparison
RM 7.85 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
340
330
320
310
300
290
0
200
400
600
800
1000
1200
1400
1600
Station (ft)


Cross Section Geometry Comparison
RM 7.66 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
330
320
310
300
290
280
0
200
400
600
800
1000
1200
1400
1600
1800
Station (ft)


Cross Section Geometry Comparison
RM 7.47 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
330
320
310
300
290
280
270
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
Station (ft)


Cross Section Geometry Comparison
RM 7.28 Nad83
1971 measured
2001 calculated
2001 measured
Elevation (ft)
300
290
280
270
260
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
Station (ft)

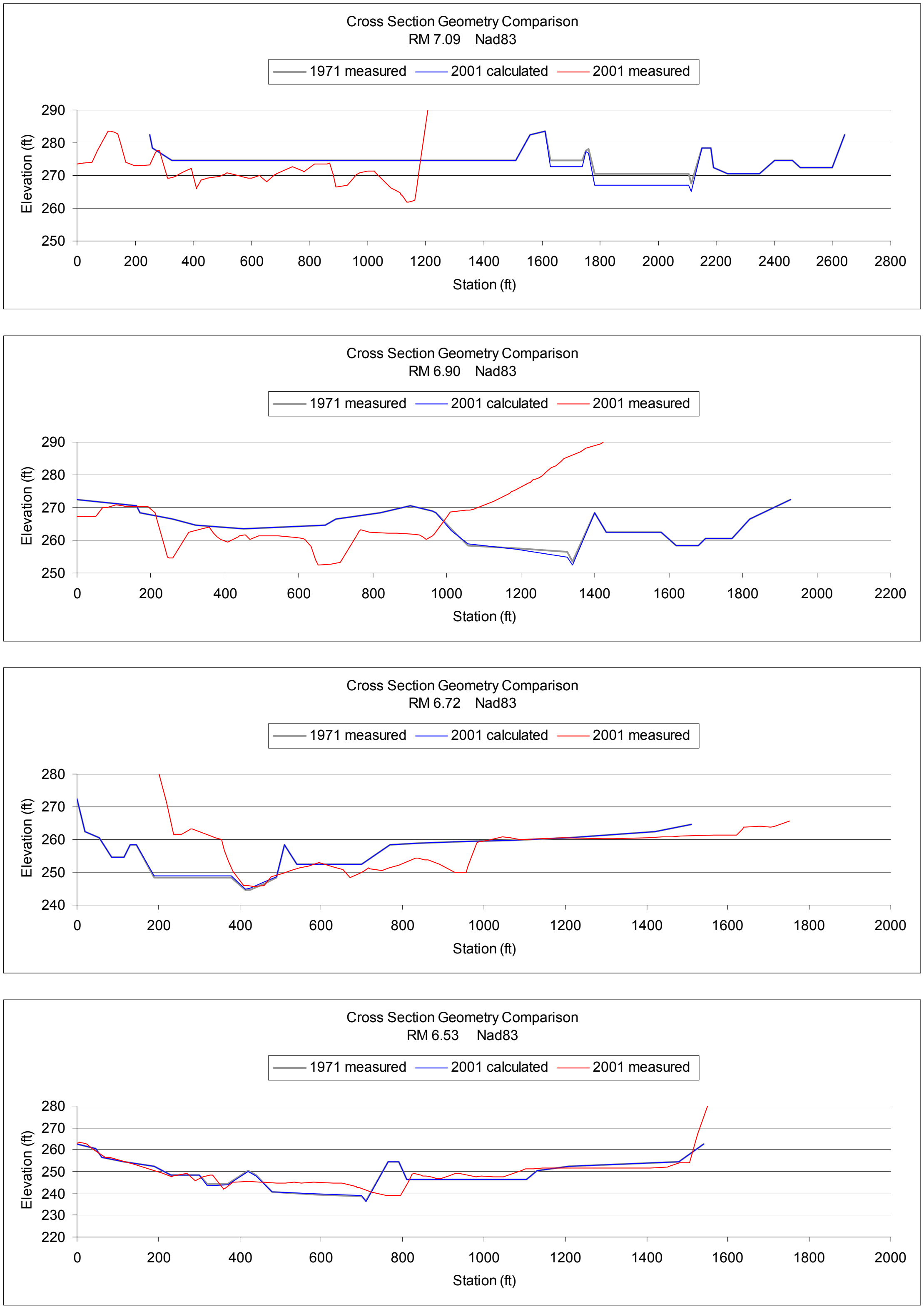


Figure 8.20. Comparison of measured and calculated cross section plots for the simulated period from 1971 to 2001.

Additional simulations were performed to test the affect of changing important parameters in the model. The original parameter set is referred to as the 'base' model and then changes to the base model were made. Manning's roughness coefficient, the critical shear stress, and the active layer thickness were changed. A comparison between the base model and the various runs is given in Figure 8.21, Figure 8.22, and Figure 8.23.

In the reach immediately below Robles Diversion, increasing Manning's *n* had the effect of causing more erosion, while decreasing Manning's n had the opposite effect (Figure 8.21). However, the effect was relatively small in both cases. In the reach immediately downstream of Foster Park, increasing Manning's *n* caused slightly more deposition, but from RM 5 to 3 increasing Manning's *n* cause more erosion. Decreasing Manning's *n* caused slightly less erosion in general, but the effect was small.

The non-dimensional critical shear stress was also varied (Figure 8.22). The non-dimensional critical shear stress is used by the transport formula to determine when particles are in motion and it affects the transport rates. Increasing the non-dimensional critical shear stress from 0.04 to 0.045 had little overall effect on the erosion and deposition. Decreasing the non-dimensional critical shear stress caused more erosion in the reach downstream of Robles Diversion.

As a further test of the model, the active layer thickness was increased and decreased (Figure 8.23). The active layer thickness control how much mixing is allowed in the riverbed. A larger the active layer thickness requires a longer time to armor. Increasing the active layer thickness in general allowed more erosion to occur in places where erosion was predicted and more deposition to occur where deposition was predicted. Decreasing the active layer thickness had the opposite effect.

Table 8.7. Description of Historical Simulations using GSTARS-1D.

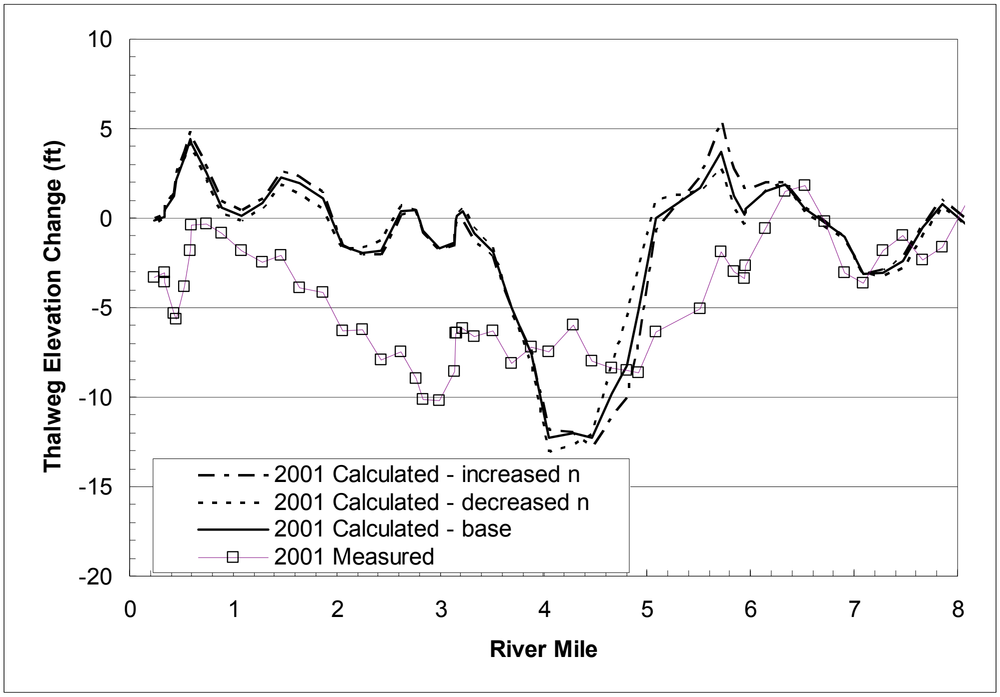
Run	**description**	**Value**
1	Base (critical shear, Manning's n, Active Layer)	0.04, 0.045, 28 cm
2	Increased critical shear	0.045
3	Decreased critical shear	0.035
4	Increased Manning's n	0.055
5	Decreased Manning's n	0.035
6	Increased Active Layer	57 cm
7	Decreased Active Layer	14 cm

Summary of Historical Comparison

The GSTARS-1D model reproduced the general trends of the Ventura River from 1971 until 2001. The erosion predicted by the model, however, was in general more localized than that observed in the river. The measured erosion occurred over longer reaches than predicted by the model. A similar behavior is expected when the model is applied to the prediction of alternatives. While the general behavior of the river will be captured by the model, the model will predict deposition that is more localized than will actually occur. For example, the model may predict that deposition is severe over a channel distance of a few thousand feet while in reality the deposition may be spread out over a longer distance.

The model is somewhat sensitive to the roughness coefficient, the critical shear stress, and the active layer thickness. However, the differences are generally small and do not qualitatively change the model predictions. The base parameter set is considered sufficient for alternative analysis. The choice of the optimal set of parameters is difficult because the 1970 data set is not complete. Most importantly, bed material samples are not available from 1970 and the exact locations of the 1970 cross sections are not known.

8.5.1. AFFECT OF MANNING'S N



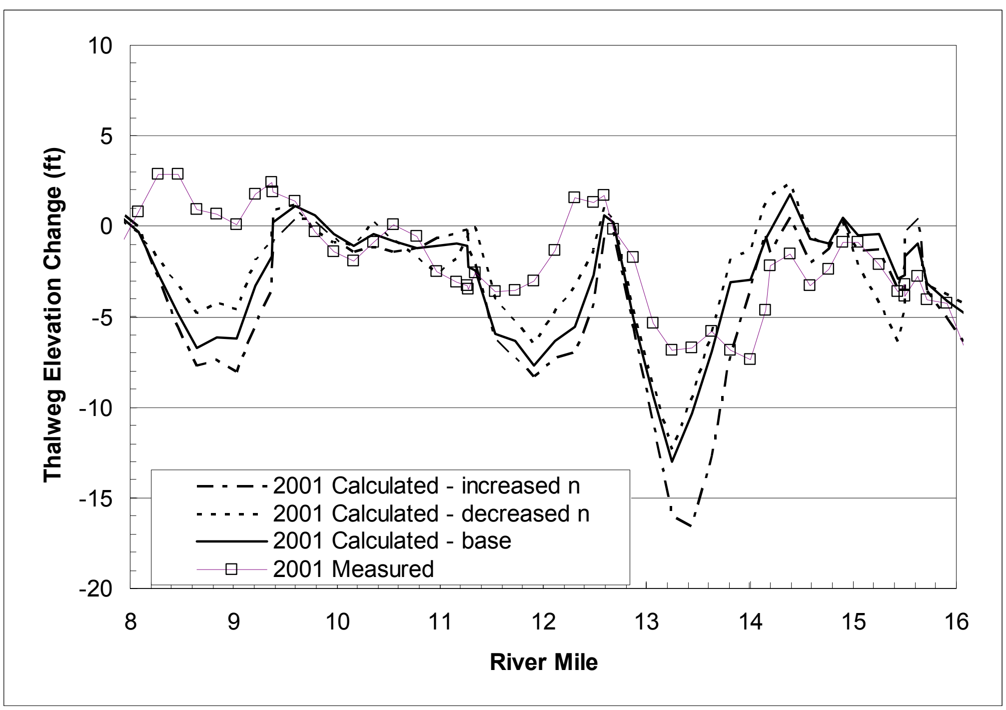
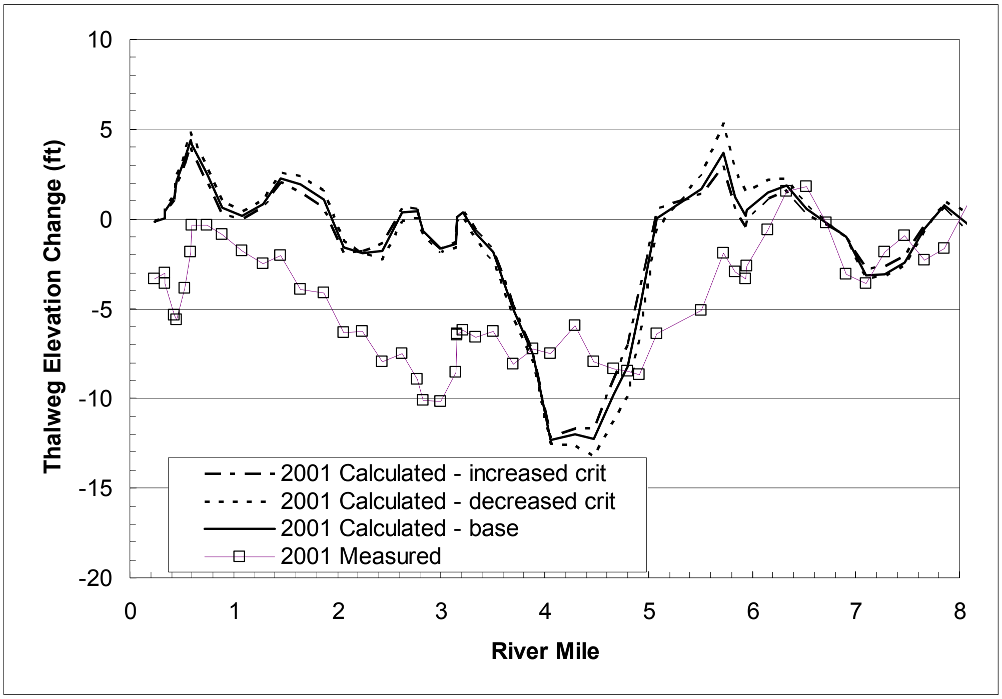


Figure 8.21. Comparison of Measured and Calculated Cross Section Plots for the Simulated Period from 1971 to 2001. Increased Manning's *n* is 0.055; Reduced Manning's *n* is 0.035; Base Manning's n is 0.045.

8.5.2. AFFECT OF CRITICAL SHEAR STRESS



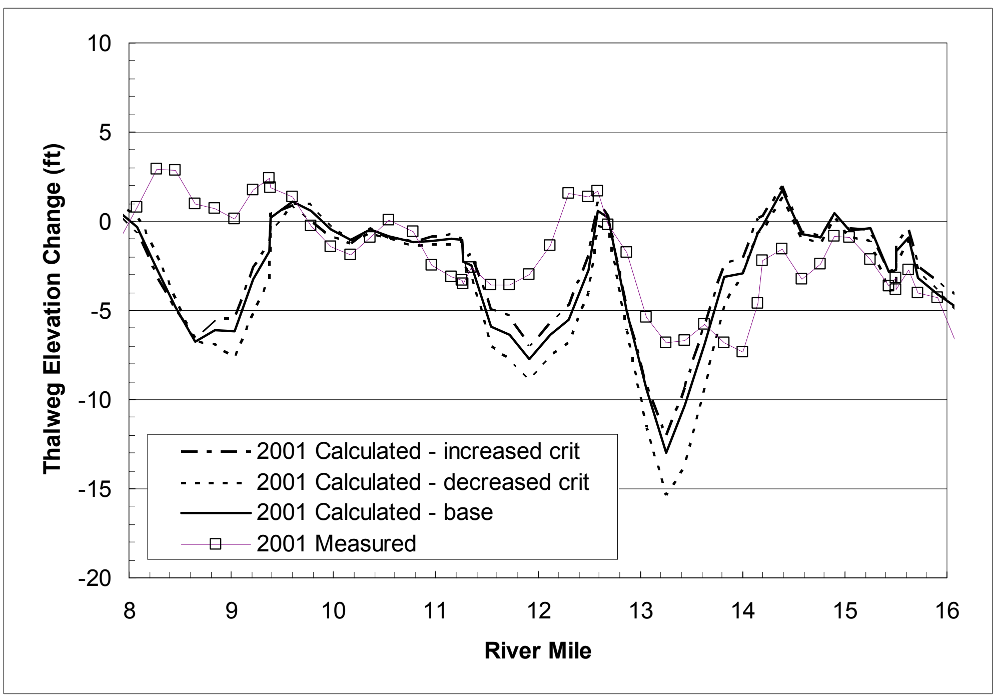
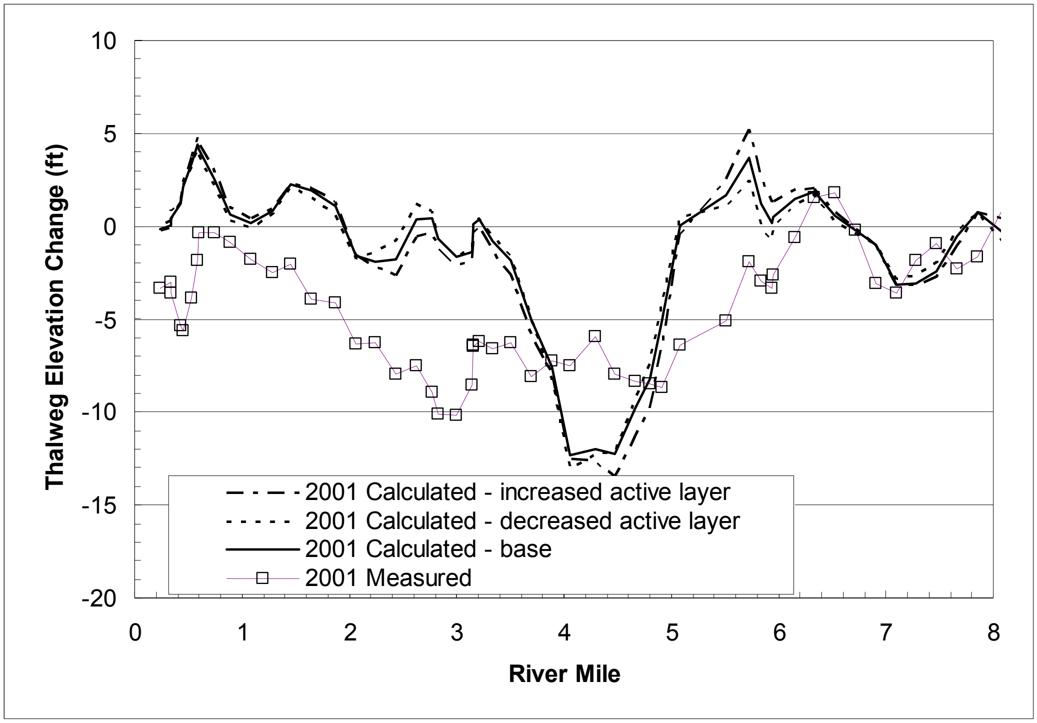


Figure 8.22. Comparison of Measured and Calculated Cross Section Plots for the Simulated Period from 1971 to 2001. Increased Non-Dimensional Critical Shear Stress is 0.045; Reduced Non-Dimensional Critical Shear Stress is 0.035; Base Non-Dimensional Critical Shear Stress is 0.04.

8.5.3. AFFECT OF ACTIVE LAYER THICKNESS



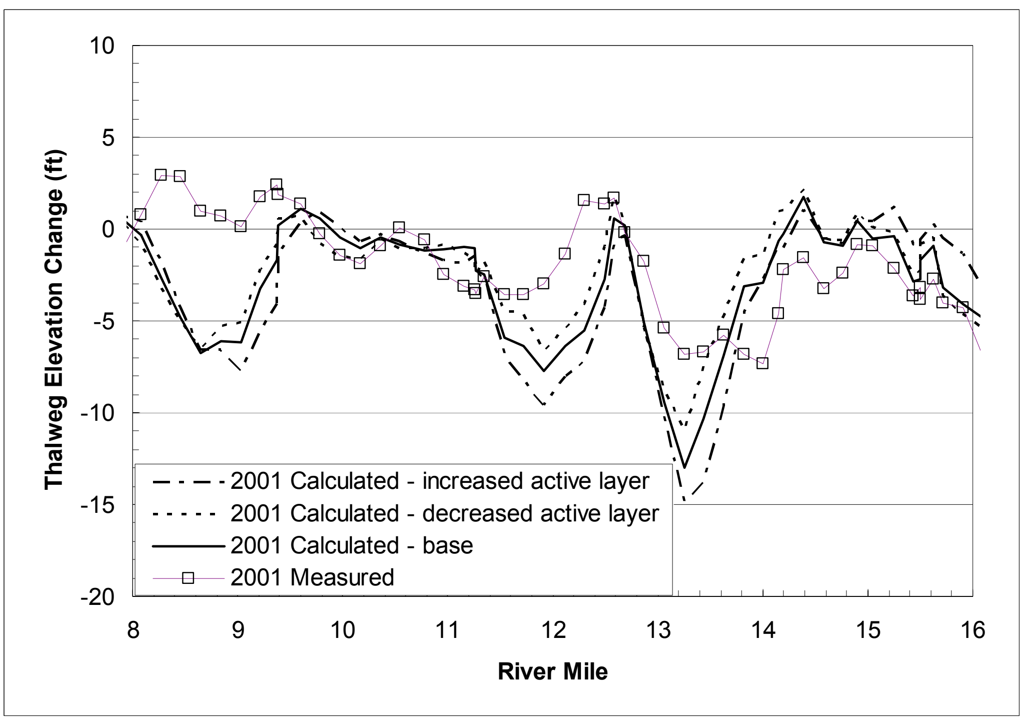


Figure 8.23. Comparison of Measured and Calculated Cross Section Plots for the Simulated Period from 1971 to 2001. Increased Active Layer Thickness is 57 cm; Reduced Active Layer Thickness is 14 cm; Base Active Layer Thickness is 28 cm.

9. Impact Descriptions

These sections describe the impacts of each alternative. The impacts are divided into: 1. Average Deposition Impacts by Reach, 2. Flooding Impacts, 3. Sediment Concentrations, 4. Robles Diversion Impacts, 5. Foster Park Diversion Impacts, 6. Impacts to Groundwater Use, 7. Delivery of Sediment to Ocean, 8. Impacts to Stream Hydraulics. Section 20 titled "Exhibit G. Model Results for All Simulations" contains graphs of many of the results of the simulations. Section 24 titled "Exhibit L. Sensitivity of Alternative 2a Impacts to Changes in Numerical Model" contains an analysis of the sensitivity of the model results to changes in certain model parameters.

9.1. Average Erosion/Deposition Impacts by Reach

The construction of Matilija Dam in 1947 has played a role in the erosion that has occurred in the upper reaches of the Ventura River. However, there are other important factors as well. The most important is perhaps the effect of the 1969 flood. This flood drastically changed the width and channel geometry of the Ventura River. Since that flood, the river has been readjusting to the smaller flows and has become significantly narrower since 1970; however, the present river width is remarkably similar to the river width in 1947 (see Figure 5.24, p. 154).

Another important factor is structures affecting the local conditions along the river. Robles Diversion traps coarse sediment and some of the coarse sediment is then removed from the river system by mechanical means. Therefore, there is a sediment deficit immediately downstream of Robles Diversion that causes erosion of the streambed. Based on the comparison of cross section data between 1970 and 2000, it is likely that sediment trapping and excavation at Robles Diversion has contributed to erosion of the streambed that has occurred from RM 14 to RM 13. Downstream, from RM 13 to RM 5, the riverbed elevation has remained relatively stable since 1970.

Another structure that affects local deposition and erosion is Santa Ana Blvd Bridge and the Live Oaks Levee. Both the bridge and the levee severely constrict the Ventura River during high flows. The constriction reduces the sediment carrying capacity of the river and causes deposition upstream of the bridge. The County currently has to excavate sediment from under the bridge to maintain flow capacity there.

From RM 5 to RM 2, there has been a significant amount of erosion taking place, with over 10 feet of bed elevation change in many locations. The Ventura River along this reach is actively incising and creating a narrow, deep channel that contains even the 100-yr flood. The erosion is at the same location as northern flank of the Ventura Avenue Anticline, which is currently rising. There are bedrock controls upstream of RM 5 that prevent erosion from progressing further upstream. These bedrock controls are evident at Foster Park Bridge. It is suspected that the erosion is due to a combination of factors. The sediment supply from the upstream reaches has been decreased due to the construction of debris basins in San Antonio Creek watershed, the construction of Casitas Dam, and the construction of Matilija Dam. In addition, the 1990's were a relatively wet period that could have caused erosion in this reach. The Ventura River system is generally a capacity-limited system, which means that there is ample sediment supply and the transport of sediment is limited by the amount of flow in the channel. If more flow is present, then overall more sediment is transported in the river and degradation can result if the supply is not also increased.

After removal of the dam, the sediment supply would increase. The increase in supply would affect the upper reaches more than the lower ones. In fact, below the Santa Ana Blvd Bridge, the deposition caused by the removal of the dam would be minimal. The deposition in the upper reaches would continue until the sediment supply has come to equilibrium with the transport capacity. The equilibrium condition would be approximately that of the pre-dam condition. Each alternative would eventually reach an equilibrium condition. Even if the dam is not removed (the No Action Alternative), sediment would eventually start to spill over the top of the dam and re-supply sediment to the downstream reach.

The deposition resulting from dam removal would change the channel plan form characteristics, channel geometry, and riverbed material. However, the changes would not necessarily be large or a cause of concern. In most cases the changes to the river is incremental and not a wholesale change in the river's characteristics. In the following sections, the deposition impacts in each reach would be described for each alternative. The impact descriptions are based upon the results from GSTARS-1D using the 1991 to 2001 hydrology repeated five times. The depths of deposition reported are changes in thalweg elevations. In most cases, the depths of deposition are the average change in the thalweg elevation over a reach or at least several cross sections. The results presented in this section describe the best estimate of the sediment impacts and are most appropriate in the analysis of environmental impacts. They do not take the uncertainty of the model predictions into account. A more detailed uncertainty analysis was performed to design the levees for the preferred alternative (see Section 10.2 titled "Revised Flood Protection for the Preferred Alternative (Alternative 4b)").

General Description of Impacts for Entire River for each Alternative

<u>No Action</u>: Significant coarse sediment would start to pass over the dam in approximately 40 years. Up until that time, erosion would continue to take place downstream of the dam in the area immediately below Robles Diversion and in the reach between RM 5 and RM 2. The erosion in the reach immediately below Robles Diversion would gradually reverse to a trend of deposition once the coarse sediment starts to pass over the top of the dam. The time required to reach equilibrium conditions in the upper reaches would be several decades after the coarse sediment starts to pass over the dam, meaning that equilibrium would not occur for at least 50 years and it may be closer to 100 years. The erosion in the reach between RM 5 and 2 would be expected to continue in the next 50 years, but at a slower rate.

<u>Mechanical Removal/Permanent Stabilization (1, 4a)</u>: The stabilization and mechanical removal alternatives would immediately re-supply sediment to the downstream reach. The reach below Robles Diversion would be expected to start to aggrade shortly after the removal of Matilija Dam. The return to equilibrium conditions would take place gradually. The complete return to equilibrium conditions may take approximately as long as it took to degrade the river, or 50 years. The changes would occur more rapidly in the first 10 years and then gradually slow down.

<u>Natural Transport with Removal of Reservoir Fines (2a)</u>: The Natural Transport alternatives would cause an initial oversupply of sediment that would quickly return the upper reaches of the

Ventura River to pre-dam elevations. This process would be expected to occur within the first 10 years. The channel may actually aggrade above pre-dam elevations at select locations if sediment is supplied to the reach too quickly. The possibility of this excessive aggradation in some reaches would require that levees be constructed relatively high. Erosion may continue to occur in the reach from RM 5 to RM 2, but the additional sediment supply would slow the erosion.

<u>Natural Transport with No Removal of Reservoir Fines (2b)</u>: The reservoir fines would not be expected to remain in the river system for long periods. These fine sediments would be easily mobilized and would quickly wash through the river system if there is water to carry them. Therefore, the effects of deposition for this alternative would be similar to those of Alternative 2a.

<u>Natural Transport with Removal of Reservoir Fines and Staged Removal (3a)</u>: The aggradation for Alternative 3a would be expected to be slightly less than for Alternative 2a because the sediments would be released over a longer period and therefore the oversupply of sediment for a given flood would be less.

<u>Natural Transport with No Removal of Reservoir Fines and Staged Removal (3b)</u>: The aggradation for Alternative 3b would be expected to be slightly less than for Alternative 2b because the sediments would be released over a longer period and therefore the oversupply of sediment for a given flood would be less. The reservoir fines would not be expected to remain in the river system for long periods. These fine sediments would be easily mobilized and would quickly wash through the river system if there is water to carry them. Because there would be two notchings, there would be two times were large concentrations of fine sediment would be present in the rivers.

<u>Temporary Stabilization with Removal of Reservoir Fines (4b)</u>: In the Temporary Stabilization Alternative, a 100-foot wide channel would be formed through the reservoir sediments and the banks of the channel would be temporarily stabilized with 3 to 7 feet of revetment. The impacts for the Temporary Stabilization Alternative would be similar to the impacts associated with 2a. However, because there would be stabilization of the toe of the bank in the reservoir region, the initial oversupply of sediment would be expected to be less severe. In addition, the removal of the temporary structures could be staged so that the sediment would be removed gradually. Once the temporary structures would be removed, there would be an additional sediment supply. This sediment would be easily eroded from the riverbanks. During a flood, the peak flows would erode sediment from the banks and make it available for transport.

Because Alternative 4b is choosen as the preferred alternative, additional investigation was done to determine the types of sediment that would deposit in the bed of the Ventura River. If the river bed were to change its composition, spawning might be affected. Plots of the d_{16}, d_{50} and d_{50} are found in Section 19.4.5, Figure 19.149, Figure 19.150, and Figure 19.151, respsectively.. The d_{16} is the diameter of which 16% of the sediment in the bed is finer than. The release of sediment from behind the dam does cause the bed to become slightly finer, but the bed still remains coarse. In addition, the bed would eventually return to very near current conditions. The d_{16} would be greater than 6 mm for all times after dam removal in all reaches upstream of RM 2. In most reaches the d_{16} would be above 10 mm for all times above RM 2. The d_{35} would be above

35 mm for all reaches above RM 2 for all times after dam removal. The d_{50} remains above 60 mm for all reaches above RM 2 for all times after dam removal.

Upstream of Reservoir Area (Reach 8)

This reach is upstream of any influence of the project and therefore no significant differences between the current condition and the alternatives would be expected in this reach.

Reservoir and Delta Area (reach 7a and 7b)

No Action: 3,000,000 yd^3 would be deposited in the reservoir in the next 50 years and most of the current active storage of Matilija Reservoir would be lost within 10 years. The delta would be expected to reach the dam face in approximately 30 to 40 years.

Mechanical Removal/Permanent Stabilization (1, 4a): The Permanent Stabilization Alternative (4a) removes the dam and necessarily stabilizes all the sediment trapped behind Matilija Dam. Therefore, no significant changes in the reservoir region from what would be constructed would be expected. Under the Mechanical Removal Alternative (1), the sediment trapped behind Matilija Dam would be removed and/or stabilized. After a majority of the sediment would be removed by truck, the stream channel would develop back to naturally functioning stream and would be near pre-dam conditions.

Natural Transport – Full Dam Removal (2a, 2b): In Alternatives 2a and 2b, Stage C (see Section 8.3.6 for description of reservoir erosion stages) would take place during the first flood and a large amount of sediment would be eroded from the reservoir. Assuming that the first flood is of average size, approximately 1,000,000 yd^3 would be moved in the first flood for Alternative 2a and approximately 2,000,000 yd^3 would be moved in the first flood for alternative 2b. The pre-dam thalweg within the reservoir would be obtained within the first year. Stage C would be expected to be complete after one flood that is as large as the average annual flood. The channel width after Stage C would be very narrow at the base. Stage D would take place during the subsequent 2 to 3 floods. During Stage D, it is expected that the channel would widen to a width approximately equal to the width of the channel upstream, which is approximately 200 feet. The current widths of the sediment deposited behind Matilija would be between 350 feet near the dam face to 1100 feet at the widest part of the reservoir and delta. The average width is 700 to 800 feet. Using an average with of 750 feet gives a river channel to reservoir width of approximately 0.27. At the end of Stage D it is expected that 2,200,000 yd^3 would have been eroded from the reservoir for Alternative 2a, and 2,800,000 yd^3 for Alternative 2b. It is possible that the estimate of 2,800,000 for Alternative 2b is too small because the channel through the reservoir region could be much wider than 200 feet and this would cause more fines to be eroded from the reservoir. However, this additional material would not affect the flooding risk. In addition, the uncertainty in the amount of fine sediment eroded from the reservoir is taken into account in the design of the downstream mitigation measures (e.g. levees).

A photo of the portion of Matilija Creek upstream of Matilija Dam is shown in Figure 9.1. An estimate of the equilibrium active channel is shown in red, which was taken from the 1947 stream centerline. The reservoir is much wider than the active channel at several locations and a significant amount of sediment may remain after Stage D in these areas.

Stage E would take place more slowly, probably on the order of decades. Larger and larger floods would be required to erode the remaining sediment. When a large flood occurs, it may cause the channel to migrate laterally and erode more sediment from the reservoir. The amount of material eroded during Stage F is uncertain because it would be entirely dependent upon the particular hydrology. If a very large flood occurs, such as the 1969 flood, it could potentially mobilize much of the remaining sediment. During the 1969 floods, over 6,000,000 tons of sediment passed Foster Park (approximately 5,000,000 yd^3). The approximately 1,600,000 yd^3 of sediment remaining after Stage D under alternative 2a represent 27 % of this amount. Therefore, the oversupply of sediment is much less than during Stages C and D.

The transition between Stage E and Stage F may not be well defined. The river would only be in equilibrium for the largest flood that had occurred since dam removal. For example, suppose that three floods occur in a period of 10 years, the largest being a 10-yr flood. The channel would then be relatively stable for flows less than or equal to a 10-yr flood, but flows larger than that would cause additional widening of the channel. Suppose again that a 100-yr flood occurs soon after dam removal. Such a flood would perhaps remove more than suggested by the numbers found in Table 9.4 and Table 9.5. Table 9.4 and Table 9.5 are the best estimate of erosion and deposition in the reservoir. They are not the estimates of the extreme events, be it extreme floods or extreme drought. After this 100-yr flood, the channel would be near stable for flows smaller than the 100-yr flood.

The largest downstream impacts occur during Stage C, with the relative impacts decreasing each subsequent stage. This is because Stage C creates the largest *oversupply* of sediment. Because all flows carry sediment, it is not the supply of sediment that creates adverse impacts, but it is the *oversupply* of sediment. The channel formation and down cutting that occurs during Stage C are processes that create very high sediment concentrations. The processes that occur during Stages D and later do not create sediment concentrations that are as high and therefore do not create the same oversupply of sediment that occurred during Stage C.

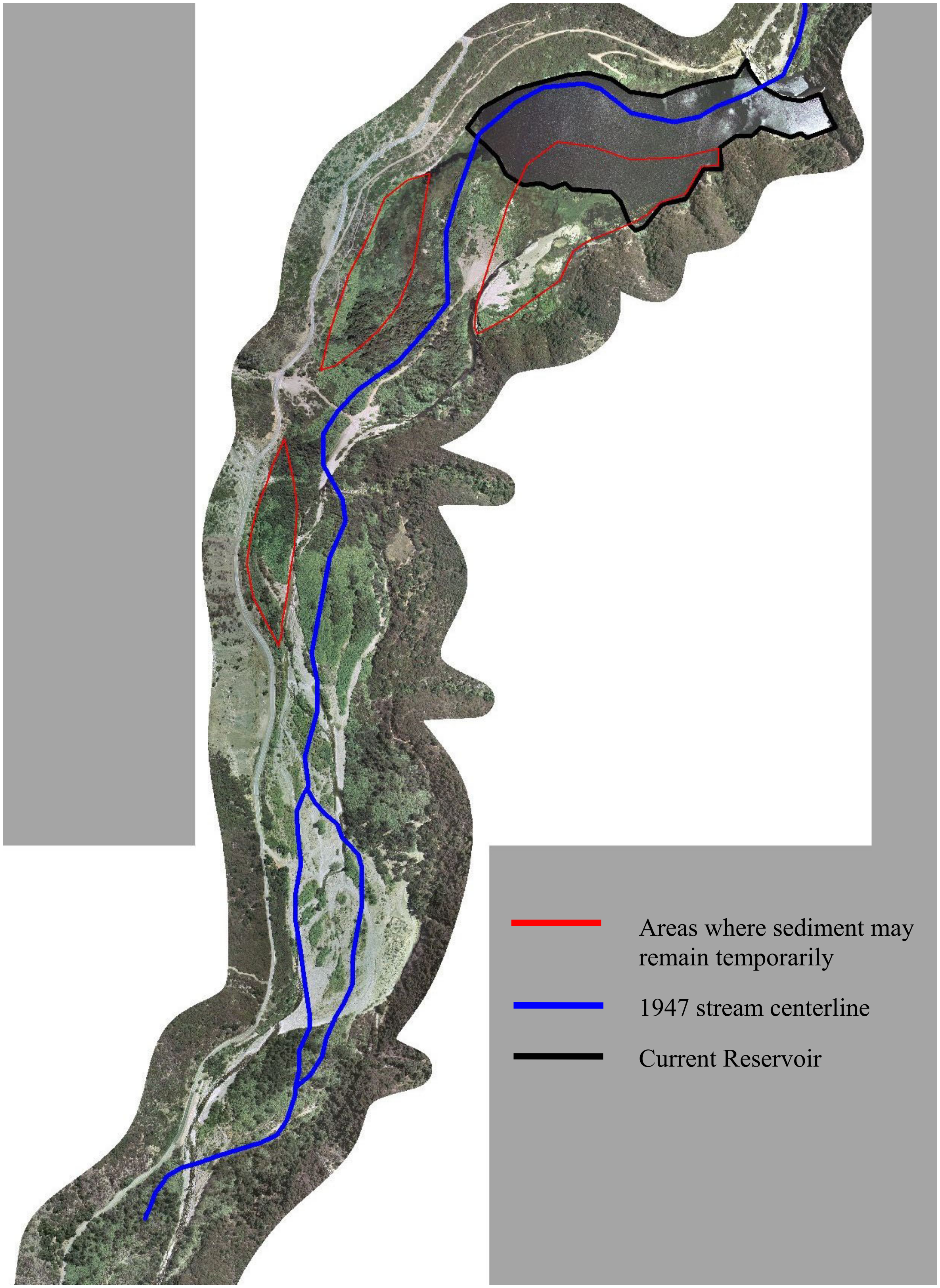


Figure 9.1. 2001 Photo of area upstream of dam.

Natural Transport – Incremental Dam Removal (3a, 3b): The processes would be much the same as in Alternatives 2a and 2b, except that Stage C would be repeated after each notching and Stage D would not be reached until the entire dam would be removed. In addition, in Alternatives 3a and 3b, less material would be eroded the first year, because the dam would be still in place. However, after year 3 the total amount eroded from the reservoir is similar to Alternative 2a and 2b.

Temporary Stabilization (4b): In the Temporary Stabilization Alternative, a 100-foot wide channel would be formed through the reservoir sediments and the banks of the channel would be temporarily stabilized with 3 to 7 feet of revetment. Therefore, the initial channel formation stage has already taken place. The erosion would occur when the water elevations exceed the revetment height and erode the banks of the channel. The channel slopes would be 3:1 and therefore erosion should initially occur as surface erosion. After the banks near the channel have been eroded, steeper slopes may result and mass failure of banks would occur.

The removal of the temporary structures could be staged so that the sediment would be removed gradually. Once the temporary structures would be removed, there would be an additional sediment supply. This sediment would be easily eroded from the riverbanks. During a flood, the peak flows would erode sediment from the banks and make it available for transport.

Table 9.1. Erosion from Reservoir for Each Alternative.

	Year			
Location	**1**	**3**	**10**	**50**
No Action	-40,000	-530,000	-1,300,000	-3,400,000
Alternative 1	0	0	0	0
Alternative 2a	1,000,000	2,200,000	2,200,000	2,200,000
Alternative 2b	2,000,000	3,200,000	3,200,000	3,200,000
Alternative 3a	770,000	1,400,000	2,200,000	2,200,000
Alternative 3b	1,200,000	2,300,000	3,200,000	3,200,000
Alternative 4a	0	0	0	0
Alternative 4b	500,000	1,000,000	2,200,000	2,200,000

Matilija Canyon (Reach 6b)

Matilija Hot Springs is located just downstream of the dam. The facilities are located on a terrace that is just above the 100-yr flood plain. The terrace is at an elevation of 970 feet and the 100-yr flood elevation at a section near the facility is 965 feet. Based on a comparison between the 1970 survey and the 2001 survey, there has been approximately 4 feet of erosion of the thalweg in this reach. However, the bed elevation at the USGS gage 11115500, which is at RM 16.0 and just downstream of Matilija Hot Springs, has remained relatively constant (based on personal communication with CMWD). The USGS gage is a concrete sill structure and acts as a control on the channel elevations. In addition, there is some uncertainty in the comparison between the 1970 survey and the 2001 survey because the exact location of the 1970 cross sections is not known. Therefore, it is likely that while some erosion has occurred in this reach, it is not severe and 4 feet is an upper estimate of its value. As sediment is re-supplied to this reach (regardless of the alternative) some aggradation of sediment may be possible, and the Hot Springs facility would be at an increased flood risk. However, the increase in risk is dependent upon the alternative.

No Action: Matilija Canyon would remain essentially stable until coarse sediment begins to pass over the top of the dam (in approximately 40 years). It would take much longer before sediment that is coarse enough to cause bed aggradation start to pass over the dam. It is estimated that approximately 100 years would pass before equilibrium elevations may be obtained in this reach. The numerical modeling indicates that this reach may slightly degrade in the next 50 years. However, the model does not take bedrock control such as occurs at the USGS gage. Therefore, it is expected that the reach would remain essentially unchanged in the next 50 years.

Mechanical Removal/Permanent Stabilization (1, 4a): The model predicts that there would be approximately 0.5 feet of deposition on average throughout this reach. The deposition should occur within the first several floods. After this time, the reach would remain relatively stable.

Natural Transport with Removal of Reservoir Fines (2a, 3a): The model predicts that their would be over 2 feet of deposition on average in this reach by year 3. The majority of the deposition should be in the lower reaches. The maximum deposition occurs 3 years after dam removal with the majority of deposition occurring below the USGS stream gage at RM 16. By year 50, the deposition decreases to an average of 0.6 feet.

Natural Transport with No Removal of Reservoir Fines (2b, 3b): The model predicts over 5 feet of deposition in this reach the first year. The large amount of deposition is due to its proximity to the dam and the sudden increase in sediment loads. The upper part of reach 6b (RM 16.47 to RM 16) would return to current elevations after 50 years. The lower portion of reach, however, would stabilize with several feet of deposition.

Temporary Stabilization (4b): The model predicts a deposition of approximately 1 foot on average in this reach the year after dam removal. The deposition would be slightly less than Alternative 2a because a wide channel was constructed through the reservoir sediments and the initial over supply of sediment would be less than in Alternative 2a. Following the first year, the reach would remain relatively stable.

Matilija Canyon to Robles Diversion (Reach 6b)

Based on the sediment removal records of CMWD, there is approximately 13,300 yd^3/yr (8 ac-ft/yr) deposited on average behind Robles Diversion Dam. A single event, such as the 1978 flood, can deposit as much as 91,000 yd^3 (56 ac-ft). The main source of sediment upstream of Robles is currently North Fork Matilija Creek, with a drainage area of 16 mi^2. There are also tributaries on the east and west sides of the Ventura River that enter upstream of Robles Dam that have a combined drainage area of 2.0 mi^2. The channel may also be a source of sediment.

Based on the deposition of sediment of gravel size and larger behind Matilija Dam, Matilija Creek supplies approximately twice the amount of coarse sediment as North Fork Matilija Creek. This number is also supported by an analysis of the drainage areas of the two watersheds. The drainage area of North Fork and tributaries between the start of the Ventura River and Robles Diversion Dam is 18 mi^2 and the drainage area of Matilija Creek is 55 mi^2, or approximately 3 times greater. Because North Fork Matilija Creek and the tributaries are much steeper than Matilija Creek, they may supply a greater amount of coarse load per area, but Matilija Creek is expected to supply at least twice as much sediment as North Fork in total.

Because all the alternatives would pass Matilija Creek sediments to the downstream reach, the deposition at Robles would be significantly increased. The expected deposition is expected to be approximately twice the current deposition or 26,600 yd^3/yr (16 ac-ft/yr) under equilibrium conditions. Currently, operation of the Robles Diversion becomes difficult once 40,000 yd^3 is deposited behind the diversion dam (CMWD, personal communication, 2003). This volume is presently only exceeded for the floods with a return period at least as long as 20 years, but with the re-supply of sediment from Matilija Creek, this volume may be exceeded for smaller floods as well.

The 1991 flood, which is an average annual flood, deposited approximately 20,000 yd^3. Under equilibrium conditions, it is expected that approximately 40,000 yd^3 would have been deposited for the 1991 flood.

The 1995 and 1998 floods correspond to floods with return intervals of 7.5 and 15 years respectively, while the 1991 flood corresponds to a flood with a return period of 3 to 4 years. Under equilibrium conditions, it is estimated that floods with a return period larger than 3 to 4 years would deposit 40,000 yd^3 or more of material behind Robles Diversion.

<u>No Action</u>: No change to the current deposition is expected for approximately 40 years. After that time, coarse sediment would gradually start to spill over the top of the dam and the deposition at Robles would gradually increase. Equilibrium of sediment supply and transport in the reservoir and reach upstream of Robles is expected to occur in approximately 50 to 70 years. After that time, the deposition at Robles would be approximately twice the current values.

<u>Mechanical Removal/Permanent Stabilization (1, 4a)</u>: The reaches upstream of Robles would reach equilibrium relatively quickly, within 10 years, and therefore the deposition at Robles would reach equilibrium conditions within 10 years. The sediment deposition without a sediment bypass should be approximately twice the current values. The deposition amounts for could be reduced significantly if a sediment by-pass is constructed (see Section 10.3 and Exhibit I).

<u>Natural Transport/Temporary Stabilization (2a, 2b, 3a, 3b, 4b)</u>: The sudden re-supply of sediment to the downstream reach would cause large amount of deposition at Robles. The exact of amount of deposition would be dependent on the flood magnitudes following dam removal. Based on the simulations run using the 1991-2001 hydrology, the full dam removal alternative 2b would deposit over 100,000 yd^3 the first year following dam removal. This amount of deposition could effectively shut down the diversion operations at Robles for that first year. The other full dam Alternative, 2a, is expected to deposit approximately 90,000 yd^3. The deposition amounts for could be reduced significantly if a sediment by-pass is constructed (see Section 10.3 and Exhibit I).

The incremental removal alternatives (3a and 3b) would deposit much less sediment the first year after dam removal. However, after the final notch is complete the deposition amounts would be similar to the first year following dam removal in Alternatives 2a and 2b. Section 10.3 discusses deposition at Robles Diversion in more detail.

Deposition in Reach 5 (Robles Diversion to Baldwin Rd Bridge)

This reach has experienced down cutting of the river channel in the past 30 years and it is located relatively close to Matilija Dam. The erosion in this reach has been due to the removal of sediment at Robles Basin and the trapping of sediment at Matilija Dam. This reach is therefore expected to aggrade significantly following the re-supply of sediment to this reach.

<u>No Action</u>: This reach is expected to continue to degrade over the next 50 years, although at a much slower rate with less than 2 additional feet of erosion by year 50. No significant aggradation in Reach 5 is expected until coarse sediment starts to pass over the dam. The reach would then slowly start to aggrade, finally reaching equilibrium in 70 to 100 years. The equilibrium condition would be near the pre-dam condition.

<u>Mechanical Removal/Permanent Stabilization (1, 4a)</u>: Aggradation in Reach 5 would begin after dam removal and continue for up to 50 years when near equilibrium conditions would be attained. Approximately 2 feet of deposition would occur in the upper portion of Reach 5 with nearly no deposition occurring in the lower portion of Reach 5.

<u>Natural Transport with Removal of Reservoir Fines/Temporary Stabilization (2a, 3a, 4b)</u>: Significant deposition is expected from RM 15 to RM 13 (Robles Diversion is at RM 14.15). The amount of deposition would decrease in the downstream direction. Significant erosion has occurred in the reach downstream of Robles Diversion and it is expected that alternatives 2a and 3a would initially supply enough sediment to this reach to aggrade rapidly the cross sections back to the pre-dam conditions. The Temporary Stabilization Alternative would also supply adequate sediment to aggrade this reach, though the process is expected to take longer because the stabilization structures would delay the sediment erosion process. Approximately 4 feet of deposition would occur in the upper portion of Reach 5 with nearly no deposition occurring in the lower portion of Reach 5.

There are several properties immediately downstream of Robles Diversion on the left side of the main channel. It is estimated that the channel elevation would rise approximately 4 feet from RM 14.2 to 13.7, and this amount of aggradation may cause some of the properties on the downstream end to be within the 100-yr flood plain.

<u>Natural Transport with No Removal of Reservoir Fines (2b, 3b):</u>

The deposition for the Natural Transport with No Removal of Reservoir Fines Alternative would be similar to the Alternatives 2a and 3a. On average, there would be slightly more deposition predicted for Alternatives 2b and 3b than for Alternatives 2a and 3a, but the maximum deposition amounts would be very similar between these alternatives.

One potential difference between Alternatives 2b and 2a (as well as 3b and 3a) is the amount of floodplain deposition expected. The amount of deposition reported is for the main channel and would be different for the floodplains. The floodplains in Reach 5 would be subject to the most deposition of any reach. However, the amount of floodplain deposition would be highly dependent upon the flood history. For example, if the first flood to occur after dam removal is a 100-yr flood, then much of the floodplain would be inundated and it is expected that sediment deposition in the floodplain would occur on the receding limb of the hydrograph. However, if several relatively small floods (e.g. 2-yr floods) occur then most of the fine reservoir sediment would be carried all the way to the ocean without an opportunity to deposit on the floodplains.

One-dimensional models do not explicitly predict the deposition that occurs on the floodplain and it is difficult to model the complicated interactions between the main channel and the floodplain. However, some simple estimates can be made. First, floodplain deposition can only occur in areas where there is significant floodplain flow. Second, the deposition elevations would be less than the water surface elevations. Reach 5 would be most subject to floodplain deposition as it is near the dam and has large floodplains that can become active.

Deposition in Reach 4 (Baldwin Rd to San Antonio Creek)

This reach has remained relatively stable since 1971 and should continue to remain relatively stable. The main concern in this reach is the levee that protects the town of Live Oaks. The Santa Ana Blvd bridge is located downstream of the Live Oaks Levee and it can constrict flows causing water to back up against it and then overtop the levee. Even though the reach as a whole is relatively stable, the reach immediately upstream and downstream of Santa Ana Blvd may not be stable because the bridge severely constricts the river. The bridge is a site of active channel maintenance and the county has a maintenance program to maintain the channel invert at the bridge to an elevation of 393.5 feet. The 100-yr flow is passed if the sediment is excavated to 393.5 feet. The October 2001 and the March 2002 surveys measured the bed elevation at Santa Ana to be 396 feet.

<u>No Action</u>: The erosion that has taken place in reach 5 may progress down into this reach, with approximately 1 to 2 feet of erosion.

<u>Mechanical Removal/Permanent Stabilization (1, 4a)</u>: The re-supply of sediment to this reach would prevent any more erosion occurring in this reach. There would be some slight deposition in this reach. Depending upon location within the reach, 0 to 2 feet of deposition is predicted. Because of the re-supply of sediment to the river channel, the sediment excavation at the Santa Ana Blvd Bridge would need to be increased. There is no long-term record of sediment excavation at this site so it is difficult to estimate the increase in excavation required. If Santa Ana Bridge is replaced, no sediment excavation would be required and the flood risk would be decreased. A bridge replacement is recommended for the Mechanical Removal and Permanent Stabilization Alternatives.

<u>Natural Transport/Temporary Stabilization (2a, 2b, 3a, 3b, 4b)</u>: This reach would aggrade approximately 2 feet on average in the next 50 year. Because of the additional sediment being deposited in this reach, the County may not have an opportunity to remove sediment before a large flow occurs. The 100-yr water surface elevation (wse) would then force pressure flow at the bridge and cause water to overtop the levee. If the bridge is re-designed, the wse would drop and the increase in wse would be less. A bridge replacement is recommended for the Natural

Transport and Temporary Stabilization Alternatives instead of increased maintenance because it would be difficult to guarantee that maintenance occurs before the 100-yr flood. Even a small flood preceding the 100-yr event could deposit material and prevent excavation before the 100-yr event arrives.

Deposition in Reach 3 (San Antonio Creek to Foster Park)

Casitas Springs Levee protects the town of Casitas Springs from RM 6.84 to 7.85. It is expected some aggradation would occur *regardless* of the alternative (including No-Action) in this reach with the majority of deposition occurring in the upper portion of the reach, in the area of Casitas Levee.

<u>No Action</u>: There is some aggradation expected due to the natural sediment loads from the upstream river channel and San Antonio Creek. Approximately 2 feet of deposition is expected in the area of Casitas Springs Levee during the next 50 years.

<u>Mechanical Removal/Permanent Stabilization (1, 4a)</u>: Due to the increases in sediment loads, the deposition would be approximately 3 feet in the area of Casitas Springs Levee during the next 50 years.

<u>Natural Transport/Temporary Stabilization (2a, 2b, 3a, 3b, 4b)</u>: In the Casitas Springs area, it is expected that there would be approximately 4 to 6 feet of sediment deposition at the end of 50 years.

Deposition in Reach 2 (Foster Park Bridge to Main St Bridge)

The reach from RM 5.5 to RM 2 has experienced the most erosion of any reach on the Ventura River. The erosion is likely due to constriction of the channel by bridges, and the trapping of sediment by Casitas Dam. For all alternatives, the erosion is expected to continue from RM 5 to RM 3, but at different rates. Downstream of RM 3, deposition is predicted for all alternatives.

<u>No Action</u>: There would be up to 4 more feet of additional erosion in this reach over the next 50 years (Figure 19.137). The erosion would primarily occur between RM 5 and RM 3. Downstream of RM 3, the channel may aggrade approximately 2 to 4 feet. The deposition in the lower portion of reach 2 is affected by the over-prediction of deposition in the estuary reach, which is discussed below.

<u>Mechanical Removal/Permanent Stabilization (1, 4a)</u>: There would be up to 3 more feet of additional erosion in the reach from RM 5 to RM 3 over the next 50 years (Figure 19.146). Downstream the channel may aggrade between 4 to 6 feet. The deposition in the lower portion of reach 2 is affected by the over-prediction of deposition in the estuary reach, which is discussed below.

<u>Natural Transport/Temporary Stabilization (2a, 2b, 3a, 3b, 4b)</u>: There would be up to 2 more feet of additional erosion in the reach from RM 5 to RM 3 over the next 50 years (Figure 19.139). Downstream the model is showing aggradation of between 4 to 9 feet, however, this is likely an overestimate. The deposition in the lower portion of reach 2 is affected by the over-prediction of deposition in the estuary reach, which is discussed below.

Deposition in Reach 1 (Estuary Reach)

The model is predicting between 8 feet of deposition for the No-Action alternative and 10 feet of deposition for Alternative 2b. The relative differences between the alternatives are likely correct, but the magnitudes of depositions are over predicted. The over prediction of deposition was also shown in the comparison with historical data in section 8.5 “Testing of GSTARS-1D using Historical Data”. In the comparison with historical data, the actual data from 1970 to 2001 showed that the reach from RM 1 to RM 0 has remained relatively stable, but the model showed up to 7 feet of deposition. The reasons for the discrepancies of the model likely are caused by the model’s inability to represent the hydraulics correctly in this reach. In addition, the model does not represent the fact that the ocean currents carry away sediment from the beach. Wave action at the beach erodes the delta of sediment deposited by the Ventura River and this is not represented in the numerical model. In assessing impacts, it is recommended that the No Action Alternative be assumed to cause no deposition in the Estuary Reach. For the other alternatives, the deposition difference between the No Action and that alternative should be used to assess impacts. In the following tables, the No Action Alternative is assumed to have no deposition in Reach 1. The values of deposition for the other alternatives are the computed difference in deposition between the selected alternative and No Action.

Summary Tables of Deposition Impacts

The tables below contain the average deposition by reach of each alternative for specific years as predicted by GSTARS-1D. Numbers in parenthesis are the lower and upper bounds on the deposition throughout the reach. The average deposition by reach was computed by computing the average the thalweg change of every cross section within the reach.

Table 9.2. Summary Table for No Action Alternative. Results are from 50-yr simulation.

	Year			
Location	**1**	**3**	**10**	**50**
Average Deposition – Reach 1 (ft)	0.9	1.8	3.2	8.3 (4 to 9)
Average Deposition – Reach 2 (ft)	0.2	0.2	0.3	1.5 (-3 to 6)
Average Deposition – Reach 3 (ft)	0.2	0.3	0.6	1.9 (1.5 to 2)
Average Deposition – Reach 4 (ft)	0.0	0.0	-0.1	-0.2 (-1 to 2)
Average Deposition – Reach 5 (ft)	0.1	0.1	-0.1	-1.6 (-3 to -1)
Average Deposition – Reach 6a (ft)	0.0	-0.1	-0.4	-1.9 (-2 to -1)
Average Deposition – Reach 6b (ft)	-0.2	-0.7	-1.3	-2.0 (-3 to -1)

Table 9.3. Summary Table for Mechanical Removal/Permanent Stabilization (1, 4a). Results are from 50-yr simulation.

	Year			
Location	**1**	**3**	**10**	**50**
Average Deposition – Reach 1 (ft)	0.1	0.5	0.8	0.5 (0 to 2)
Average Deposition – Reach 2 (ft)	0.2	0.3	0.6	2.2 (-3 to 6)
Average Deposition – Reach 3 (ft)	0.3	0.5	0.9	2.6 (2 to 3)
Average Deposition – Reach 4 (ft)	0.1	0.1	0.1	0.7 (0 to 2)
Average Deposition – Reach 5 (ft)	0.4	0.5	0.5	0.6 (-2 to 2)
Average Deposition – Reach 6a (ft)	0.5	0.9	1.4	4.7 (4 to 6)
Average Deposition – Reach 6b (ft)	0.5	0.5	0.5	0.5 (-2 to 4)

Table 9.4. Summary Table for Alternative 2a. Results are from 50-yr simulation.

	Year			
Location	**1**	**3**	**10**	**50**
Average Deposition – Reach 1 (ft)	0.0	1.6	2.0	1.2 (0 to 2)
Average Deposition – Reach 2 (ft)	0.2	0.7	1.1	3.4 (-2 to 8)
Average Deposition – Reach 3 (ft)	0.2	1.2	1.9	4.2 (3 to 6)
Average Deposition – Reach 4 (ft)	0.2	0.6	0.7	2.0 (0 to 4)
Average Deposition – Reach 5 (ft)	0.4	1.4	1.7	2.1 (0 to 4)
Average Deposition – Reach 6a (ft)	0.8	3.3	4.1	5.0 (4 to 6)
Average Deposition – Reach 6b (ft)	1.0	2.2	2.0	0.6 (-2 to 4)

Table 9.5. Summary Table for Alternative 2b. Results are from 50-yr simulation.

	Year			
Location	**1**	**3**	**10**	**50**
Average Deposition – Reach 1 (ft)	1.2	2.1	2.4	1.4 (0 to 2)
Average Deposition – Reach 2 (ft)	0.6	1.1	1.5	3.6 (-2 to 9)
Average Deposition – Reach 3 (ft)	1.0	1.5	2.1	4.5 (4 to 6)
Average Deposition – Reach 4 (ft)	0.6	0.8	0.9	1.6 (1 to 4)
Average Deposition – Reach 5 (ft)	1.8	2.1	2.3	1.9 (0 to 4)
Average Deposition – Reach 6a (ft)	4.4	5.1	5.6	5.7 (4 to 8)
Average Deposition – Reach 6b (ft)	5.2	5.5	5.5	3.0 (0 to 6)

Table 9.6. Summary Table for Alternative 3a. Results are from 50-yr simulation.

	Year			
Location	**1**	**3**	**10**	**50**
Average Deposition – Reach 1 (ft)	0	1.6	2.4	1.7 (0 to 2)
Average Deposition – Reach 2 (ft)	0.1	0.7	1.1	3.5 (-2 to 8)
Average Deposition – Reach 3 (ft)	0.1	1.1	1.9	4.2 (3 to 6)
Average Deposition – Reach 4 (ft)	0.0	0.5	0.7	2.2 (0 to 4)
Average Deposition – Reach 5 (ft)	0.1	1.1	1.5	2.0 (0 to 4)
Average Deposition – Reach 6a (ft)	0.0	2.2	3.2	5.8 (4 to 7)
Average Deposition – Reach 6b (ft)	1.0	1.8	1.9	1.1 (0 to 4)

Table 9.7. Summary Table for Alternative 3b. Results are from 50-yr simulation.

	Year			
Location	**1**	**3**	**10**	**50**
Average Deposition – Reach 1 (ft)	1.3	2.0	2.6	1.4 (0 to 2)
Average Deposition – Reach 2 (ft)	0.7	1.1	1.6	3.6 (-2 to 9)
Average Deposition – Reach 3 (ft)	1.0	1.5	2.2	4.5 (4 to 6)
Average Deposition – Reach 4 (ft)	0.6	0.7	1.0	1.6 (1 to 4)
Average Deposition – Reach 5 (ft)	1.6	1.9	2.2	1.3 (-1 to 3)
Average Deposition – Reach 6a (ft)	4.1	4.8	5.5	4.1 (2 to 6)
Average Deposition – Reach 6b (ft)	3.7	4.0	4.2	1.1 (0 to 5)

Table 9.8. Summary Table for Alternative 4b. Results are from 50-yr simulation.

	Year			
Location	**1**	**3**	**10**	**50**
Average Deposition – Reach 1 (ft)	0	1.1	2.3	1.6 (0 to 2)
Average Deposition – Reach 2 (ft)	0.2	0.6	1.2	3.6 (-2 to 9)
Average Deposition – Reach 3 (ft)	0.2	1.0	2.0	4.2 (3 to 6)
Average Deposition – Reach 4 (ft)	0.2	0.5	0.8	2.3 (0 to 4)
Average Deposition – Reach 5 (ft)	0.4	1.0	1.4	2.2 (0 to 4)
Average Deposition – Reach 6a (ft)	0.5	1.5	2.3	6.4 (4 to 8)
Average Deposition – Reach 6b (ft)	0.7	1.0	1.0	0.9 (-2 to 4)

Table 9.9. Summary Table for All Alternatives. Results are after 50 years of simulation.

		Alternatives					
Location	**No Action**	**1, 4a**	**2a**	**2b**	**3a**	**3b**	**4b**
Reach 1 (ft)	0	0.1	0	1.2	0	1.3	0
Reach 2 (ft)	1.5	2.2	3.4	3.6	3.5	3.6	3.6
Reach 3 (ft)	1.9	2.6	4.2	4.5	4.2	4.5	4.2
Reach 4 (ft)	-0.2	0.7	2.0	1.6	2.2	1.6	2.3
Reach 5 (ft)	-1.6	0.6	2.1	1.9	2.0	1.3	2.2
Reach 6a (ft)	-1.9	4.7	5.0	5.7	5.8	4.1	6.4
Reach 6b (ft)	-2.0	0.5	0.6	3.0	1.1	1.1	0.9

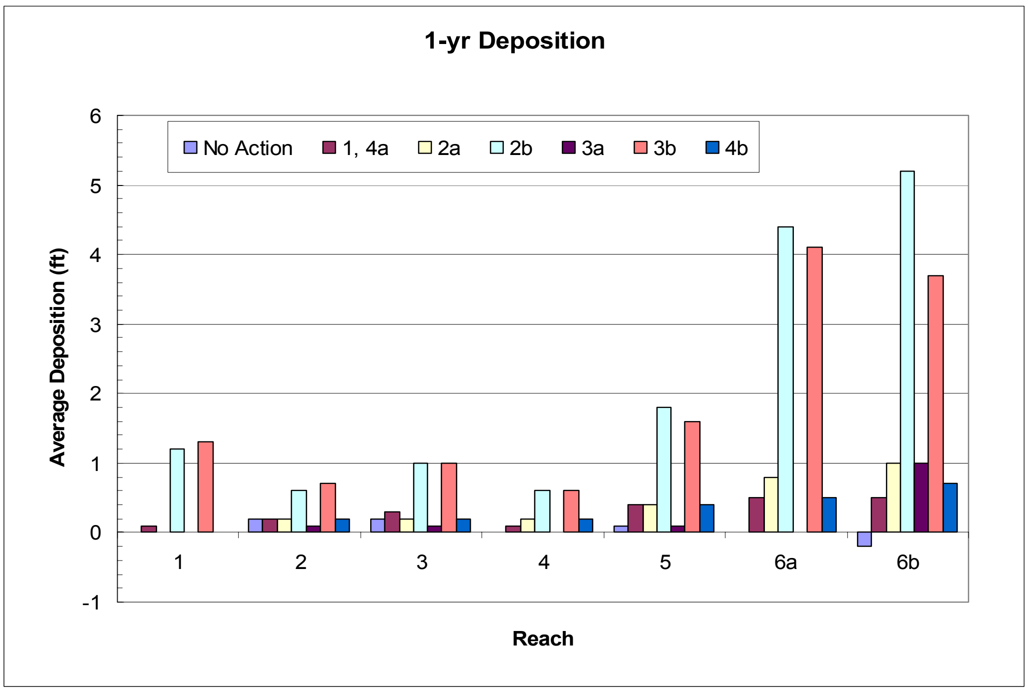


Figure 9.2. Reach Averaged Deposition after 50 years Following Dam Removal for Each Alternative.

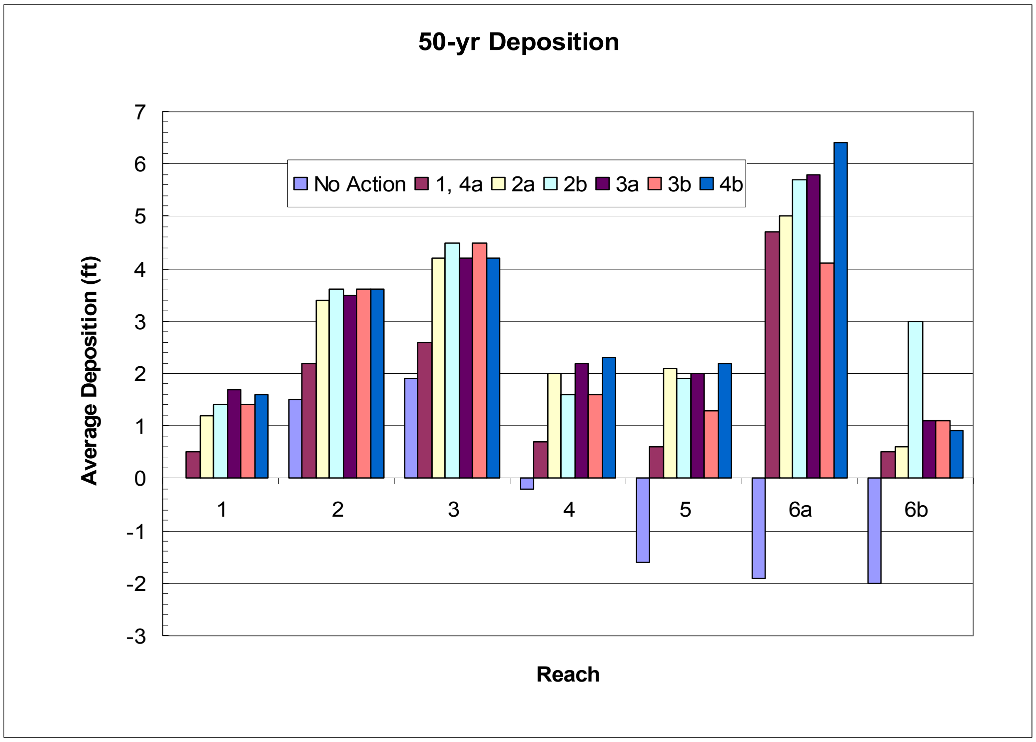


Figure 9.3. Reach Averaged Deposition after 50 years Following Dam Removal for Each Alternative.

9.2. Sediment Concentrations

Currently the majority of fine sediment (silt and clay) that enters Matilija Reservoir passes over the dam when the reservoir spills. Therefore, the downstream reaches currently experience approximately natural concentrations of fine sediment during flood events. As a consequence, the No Action, Mechanical Removal, and Permanent Stabilization Alternatives (1, 4a) have similar fine sediment concentrations.

No Action: Most all the silt and clay that enters Matilija Reservoir passes over the top of Matilija Dam. However, there is still a small amount of silt and clay that is trapped behind Matilija Dam at the lower flows. It is expected that the average fine sediment concentrations downstream of Matilija Dam would increase by approximately 30% after the reservoir is nearly filled with sediment, which is expected to occur in approximately 10 years.

Mechanical Removal/Permanent Stabilization (1, 4a): For the Mechanical Removal, Permanent Stabilization, and No-Action Alternatives, the fine sediment concentrations after the first flood events would not be significantly different from each other. However, before the first flood, the deconstruction of the dam and the mechanical removal of sediment would introduce fine sediment into the river system. In addition, it would be impossible to removal all the fine sediment from the system. There would be residual fine sediment that remains between large cobbles and boulders. The residual sediment would be easily mobilized by the first flows that pass through the reservoir area. These river flows would likely carry high concentrations of sediment until the first flood flow 'cleans' out the reservoir area.

Natural Transport with Removal of Fines in Reservoir Area (2a, 3a):

Two simulations were performed to assess the concentration of fine material for the Natural Transport Alternatives where the fines would be removed from the reservoir (2a, 3a): the 1991 flood and the 1998 flood. The floods of 1998 and 1991 were chosen as representative floods corresponding to the approximate 15 year and 3 to 4 year floods, respectively. For both floods, there would be an initial high concentration (around 50,000 mg/l or greater) of fine sediment that lasts a short period. The period of high concentrations would depend upon the flow rate. If a stream flow diversion is constructed around the reservoir sediments, the period of high concentrations above 10 times natural conditions could be kept to a very short period (perhaps only a few days).

After the first flood peak has past, the concentrations of fine material would quickly decrease, however, they would still be 2 to 3 times larger than natural conditions. Currently, the fine concentrations fluctuate by a factor of 2 or more; so the increases, while real, would be within the range of the natural variability. After a flood with a return period greater than 10 years or after a period of 3 years with average hydrology, which ever comes first, the increase in fine sediment concentration would be expected to reduce to 10 % to 50 % greater than background concentrations. Within 10 years and as early as 5 years following dam removal, the fine sediment concentration would be similar to the No Action Alternative.

The sand concentration would also be increased. The initial concentrations after dam removal of sand would be much less than that of the silt and clay, but as a result, the impact of increased sand concentration would last longer. It is expected that several floods would have to pass through the reservoir area before the sand concentration returns to natural levels.

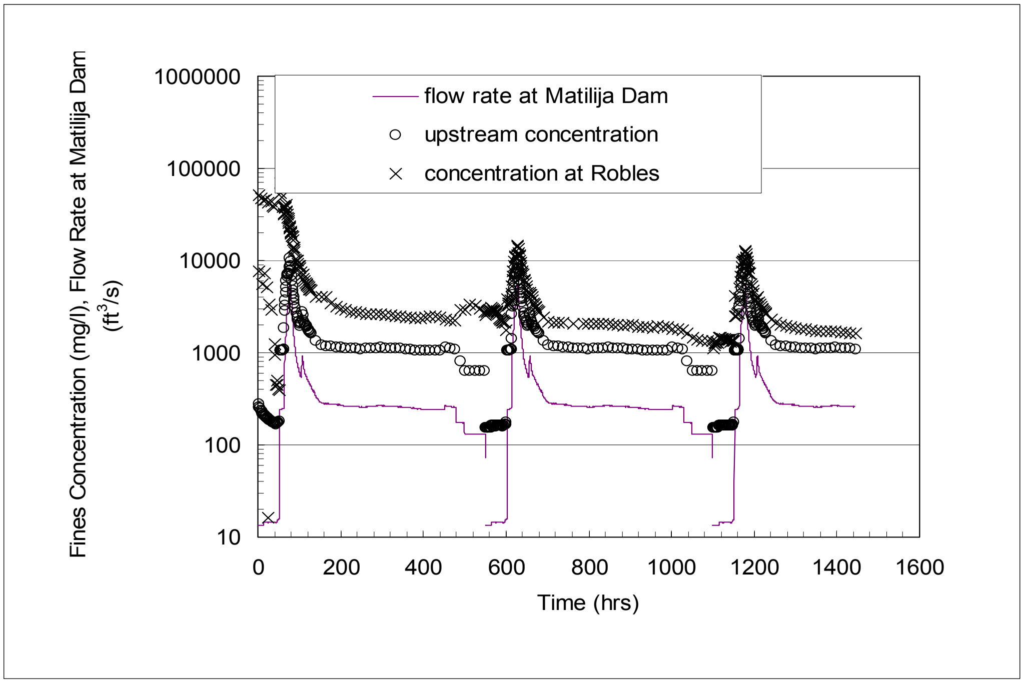


Figure 9.4. Concentration of fine sediment for alternative 2a with three 1991 floods in succession.

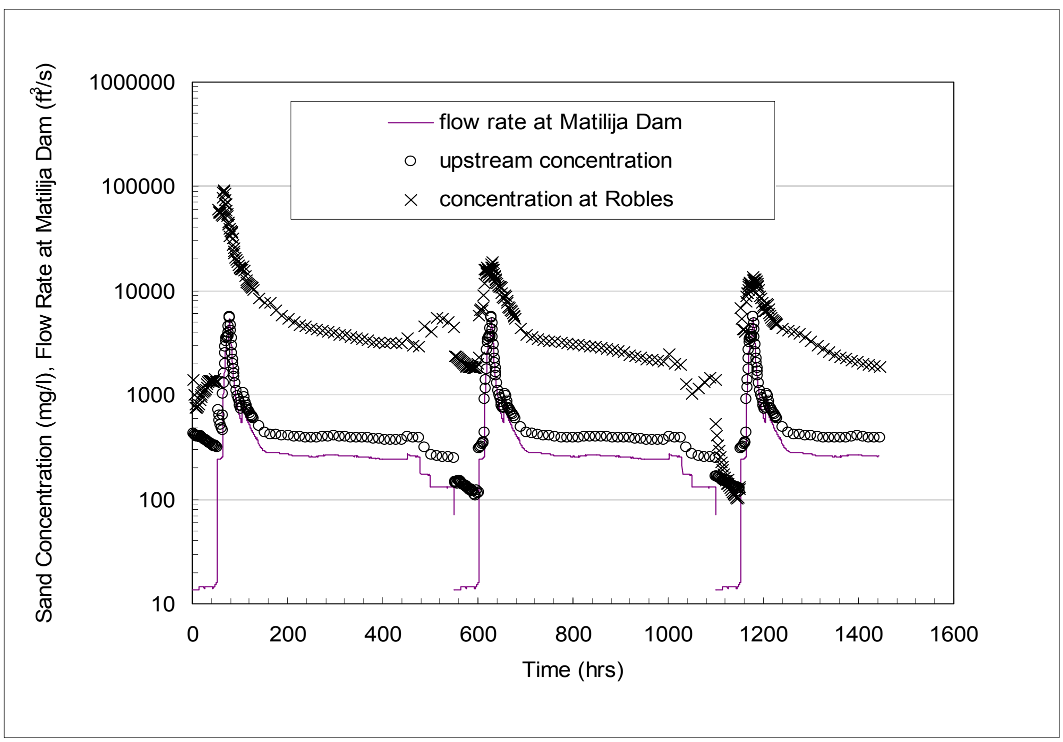


Figure 9.5. Concentration of sand for alternative 2a with three 1991 floods in succession.

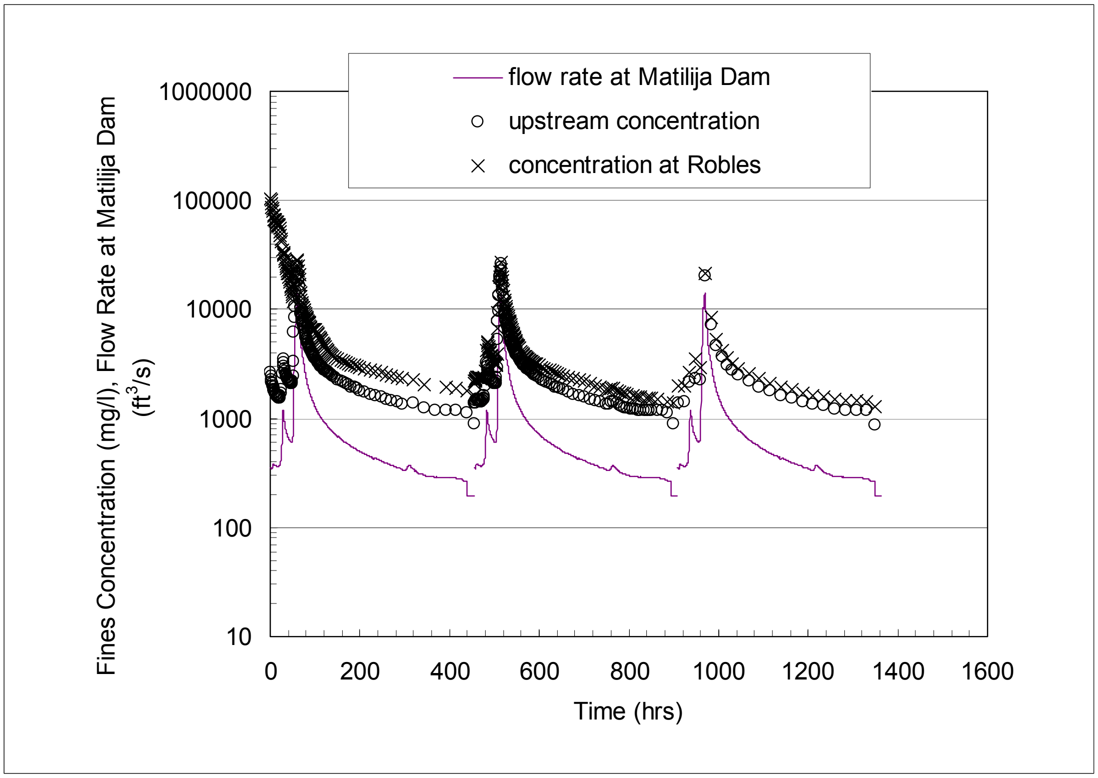


Figure 9.6. Concentration of fines for alternative 2a with three 1998 floods in succession.

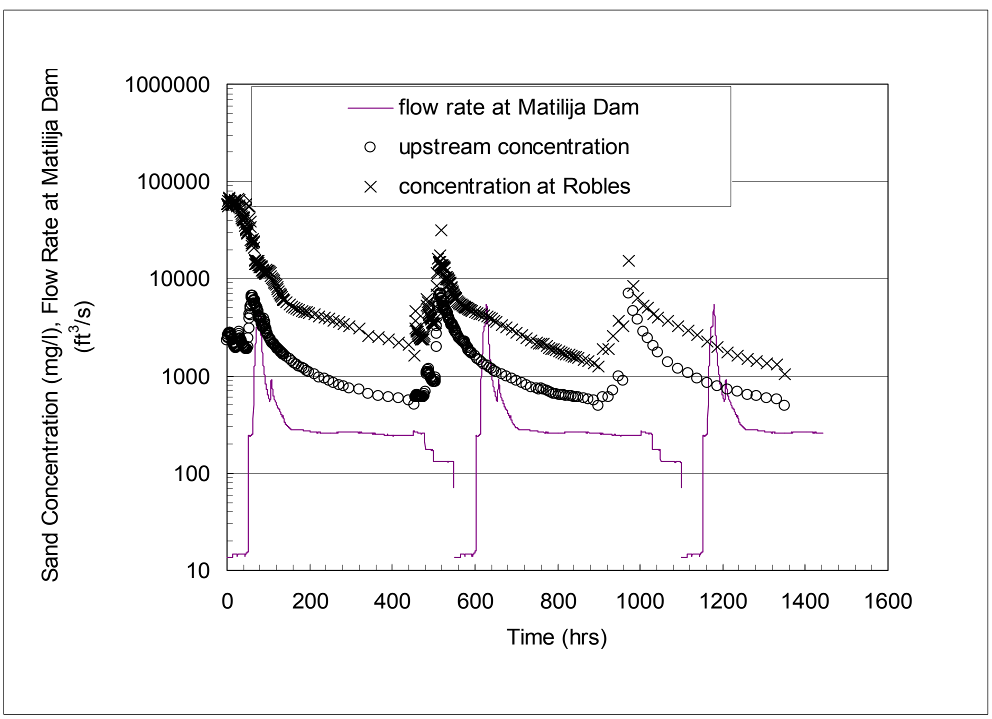


Figure 9.7. Concentration of sand for alternative 2a with three 1998 floods in succession.

Natural Transport with No Removal of Fines (2b, 3b): Four different simulations were performed to analyze the fine sediment concentration for the Natural Erosion Alternatives with no removal of sediment in the reservoir region (2b and 3b): the 1991 flood, the 1998 flood, and the dry period between 1954 until 1960. The 1991 and 1998 floods represent the condition of

Concentrations Resulting from Wet Hydrology for Alternatives 2b and 3b

For the Natural Transport Alternatives with no removal of fine sediment (Alternative 2b and 3b), concentrations would be exceedingly high downstream of Matilija Dam immediately after the introduction of the first flow following dam removal. Concentrations may reach up 100,000 - 200,000 mg/l for a short time during the first flow that erodes the reservoir sediment. These extremely high concentrations may persist days or weeks, depending upon the flow rate. The concentrations will quickly decreases, but may be 10 times higher than with the dam in place for a period up to a two years, depending upon flow (for the natural concentrations, see Figure 8.11). The first several floods would carry much higher than normal concentrations and the high concentration may persist after the peak has subsided. However, each successive flood would carry less sediment. After 10 years of hydrology similar to the 1990s, the concentration for Alternatives 2b and 3b would be not significantly different from the No Action Alternative. This is based upon the fact that no significant erosion in the reservoir occurs after 10 years under wet hydrological conditions (Table 9.1).

As an example, the concentrations of fine sediment for the first two floods following dam removal are shown in the following figures. If the first flood passing through the reservoir sediments after dam removal is large (e.g. a 1998 flood) then the fine sediment concentration decreases more rapidly on the receding limb of the hydrograph. However, if the first flood is smaller (e.g. a 1991 flood) then the concentrations may be still be near 100,000 mg/l for a period after the flood. In addition, the second flood would also have higher sediment concentrations. In the examples presented below, the second 1991 flood had peak concentrations of 60,000 mg/l, while the second 1998 flood had peak concentrations of 30,000 mg/l.

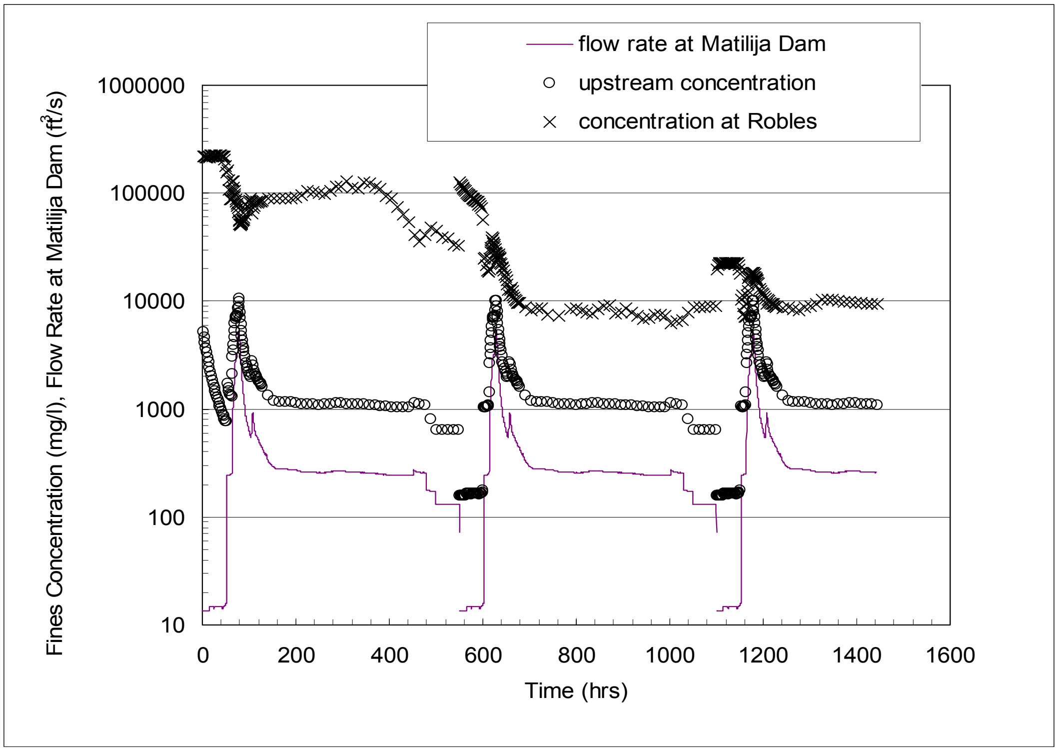


Figure 9.8. Concentration of Fine Sediment for Alternative 2b with Three 1991 Floods in Succession.

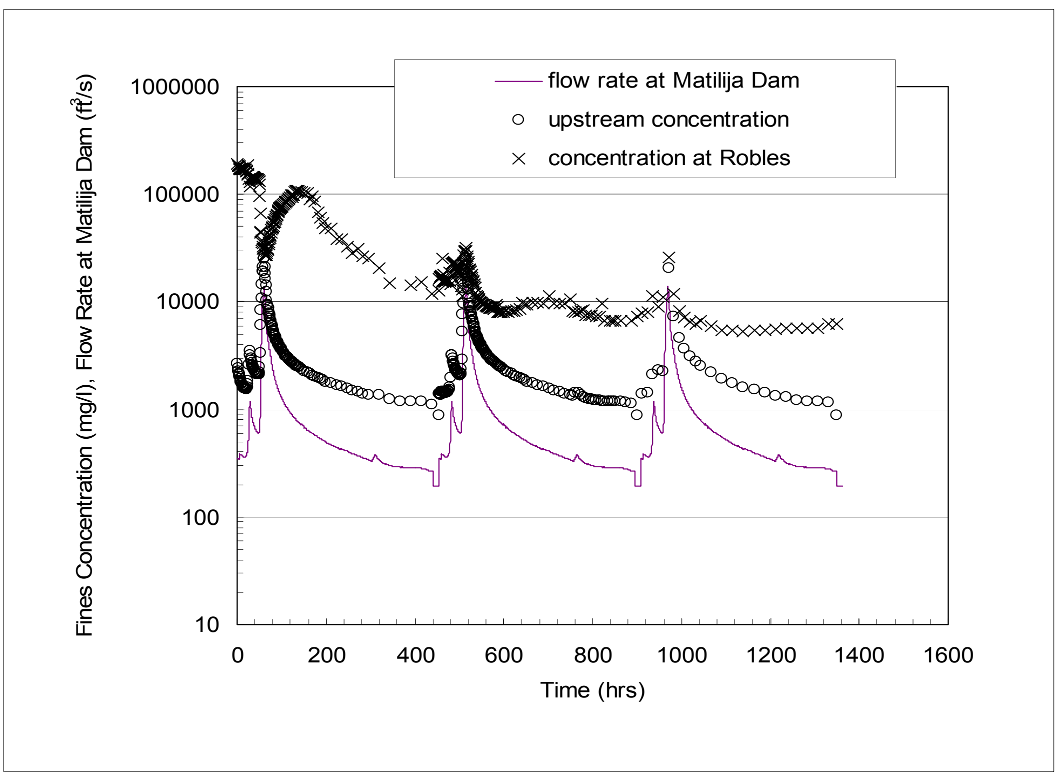


Figure 9.9. Concentration of Fine Sediment for Alternative 2b with Three 1998 Floods in Succession.

Concentration Resulting from Dry Hydrology for Alternatives 2b and 3b

If there is a dry period immediately following dam removal, the high concentrations would persist longer. The hydrology from 1954 until 1960 was used as a representative dry period and Alternative 2b was simulated. The results are shown in Figure 9.10. For most of the first wet season, the concentrations may exceed 100,000 mg/l. For alternative 2b, it is estimated that at least 6,000 acre-ft of water needs to pass through Matilija Reservoir area to remove enough fine sediment to reduce the concentrations below 10,000 mg/l. A volume of 6,000 acre-ft would correspond to a flow of 100 ft^3/s for 1 month.

After the initial flushing of fine material, the concentrations would gradually decrease. Currently, concentrations presently vary between 10,000 mg/l during flood flows (approx. 5000 ft^3/s) to 1,000 mg/l during flows with a magnitude of 100 ft^3/s. In the simulation of the Natural Transport Alternative 2b, the concentrations for a low flow of 100 ft^3/s may be as high as 10,000 mg/l for the years 2 and 3 following dam removal. It is expected that after a flood of average magnitude pass through the reservoir area (i.e. a 2-year flood), the concentrations would return to acceptable levels. The figure below still shows concentrations approximately 20 times higher than background levels after 6 years. However, the model does not adequately model the armoring process that would occur. The model is allowing the fines to be eroded from a bed that in reality would be armored after short period. Therefore, it is suspected that the model is showing more fines eroded from the bed than in reality would occur. Additional improvements to the sediment transport model would need to be made to correct this problem.

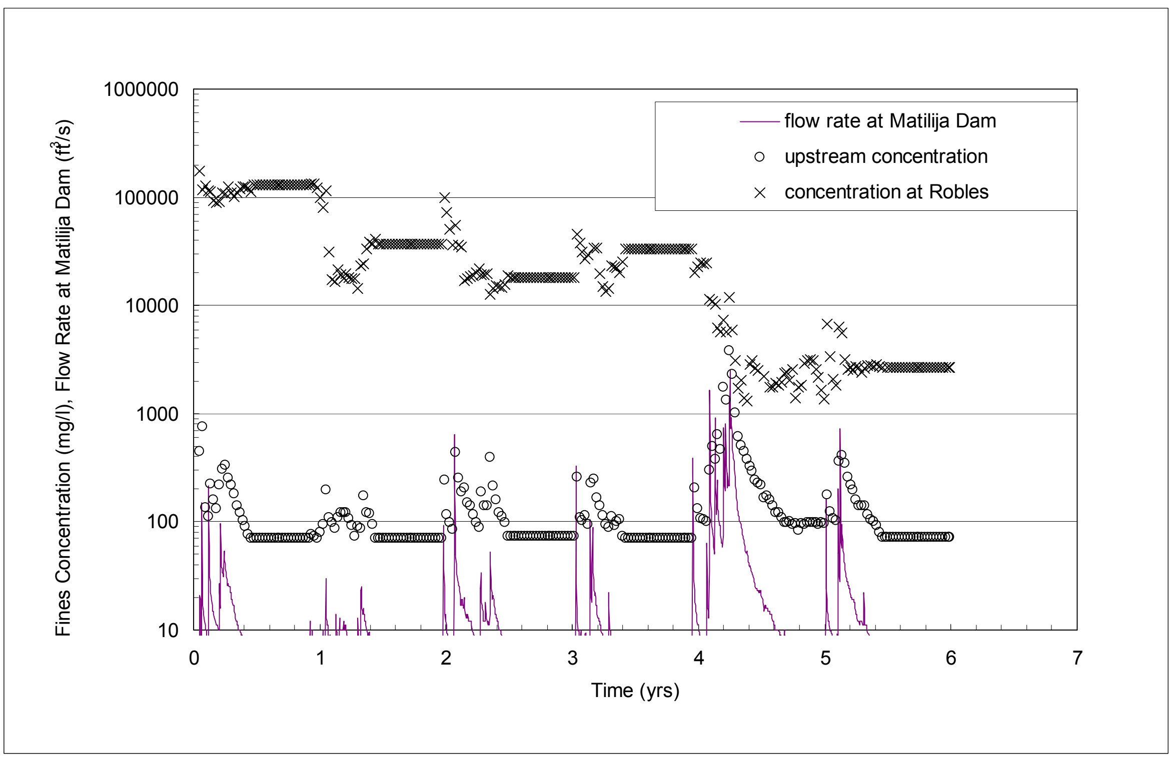


Figure 9.10. Concentration of Fine Sediments (silt and clays) at Robles during dry years for alternative 2b.

<u>Temporary Stabilization Alternative (4b)</u>:

This alternative constructs a channel that would allow the low flows to pass down stream without picking up sediment. However, high flows and following removal of the bank revetment, there may be fine sediment mobilized. Because there would be multiple removals of stabilization structures, there would be multiple impacts of fine sediment. After each removal, there would be some fine sediment released into the river as the flood flow passes through the area. The fine sediment would be mobilized as the banks are eroded. As the flood recedes, the water elevation would recede from the banks and no longer erode the fine sediment. Therefore, the increases in turbidity would be mostly confined to the flood events and the lows flows would not experience large increases in turbidity. The magnitude of the sediment concentration increases would most likely be about 2 to 4 times greater than natural conditions before the removal of the first revetment. After the first revetment is removed, the concentrations may temporarily increase between by a factor of 2 to 10 times the current condition. Each subsequent removal of revetment would produce similar increases in turbidity. The time required for the sediment to be transported downstream would be a function of the amount of sediment eroded from the banks. The physical processes would be quite complicated and difficult to simulate accurately. After the final removal of revetment, the turbidity levels should stabilize at natural conditions after one or two floods of average size pass through the reservoir area.

9.3. Robles Diversion Impacts

Both fine sediment and coarse sediment may affect the water supply and/or water quality that is diverted from the Ventura River at Robles Diversion.

Fine sediment can adsorb nutrients and these can be transported to Casitas Reservoir through the diversion at Robles Diversion Dam. Natural flood runoff from Coyote and Santa Ana Creeks, and the historic diversion from the Ventura River have caused algal production in Casitas Reservoir. The water quality data show nitrate concentrations less than 1 mg/L as nitrate. This concentration is low and should not cause algae problems; however, there was no indication of flow conditions and the concentration would be higher during higher runoff conditions. Phosphorus is the other nutrient associated with algal production and phosphorus data was available in the water quality data from the natural runoff. Since algae production has occurred, one would have to suspect that enough phosphorus is in the runoff to support short-term algal production. One composite fine sediment sample from Matilija Dam was leached and the leachate was analyzed for phosphorus. The concentration was 0.18 mg/L which is high enough to cause algal problems in a lake if the algae are blue-green that are capable of fixing nitrogen from the atmosphere. Better nutrient data is needed from the natural flood runoff and from seepage from the fine sediment in Matilija Reservoir. The alternatives that prevent or minimize the clay and silt from Matilija Dam from entering Casitas Reservoir would minimize the water quality impacts there. Usually much of the phosphorus loading would be adsorbed on the fine sediment and would settle out in a reservoir or sediment basin with the sediment. The more clay and silt from Matilija Reservoir that gets into Casitas Reservoir the greater the chance for increased algal production. The sediment would also fill in the storage volume and reduce the available storage. However, the loss in storage in Casitas Reservoir due to the removal of Matilija Dam must be less than the volume of fine material stored behind Matilija Dam. There is approximately 1700 ac-ft of silt and clay material, so the loss in storage in Casitas Reservoir must be less than 1700 ac-ft, which is 0.7% of the original Casitas reservoir capacity.

Coarse sediment would cause problems of deposition in the basin behind Robles Diversion or in the canal linking the Ventura River with Casitas Dam. Matilija is still trapping sand size and larger material. Therefore, the main source of coarse sediment to Roles Diversion is North Fork Matilija Creek. Matilija Creek transports at least twice as much gravel and cobble material as North Fork Matilija Creek. Eventually, the coarse sediment from Matilija Creek would begin to reach Robles Diversion; however, the time required for coarse sediment from Matilija Creek to reach Robles Diversion is very different between alternatives.

Based on the clean out records of CMWD, the current average coarse sediment deposition is approximately 13,300 yd^3/yr (see Table 1.4, p. 71). However, the deposition is highly variable from year to year and varies between practically zero during dry years to over 90,000 yd^3 for events such as the 1978 flood or the 1969 flood. Under the equilibrium condition, the deposition is expected to be twice the current deposition or 26,600 yd^3/yr. The deposition is expected to be twice current amounts because Matilija Creek supplies up to 3 times as much sediment as North Fork, which means that the sediment inflow would be increased by a factor of three. However, it is also expected that the trap efficiency of Robles Diversion would decrease by a factor of two because of the increased deposition that reduces the volume available for trapping sediment. Therefore, the net increase in deposition would be approximately twice the current deposition.

To compute the amount deposited behind Robles Diversion, a combination of the results from GSTARS-1D and the estimate for equilibrium deposition were used. GSTARS-1D did not have the capability to represent the sediment removal activities that are conducted in Robles Basin after it fills with sediment. Therefore, after the basin fills the model does not predict the correct amount of deposition because no clean out activities are represented. The following procedure was therefore used to predict the deposition behind Robles,

1. It was assumed that the GSTARS-1D correctly represents the deposition in year 1 to 3.

2. From year 1 to year 3, the deposition rates were assumed the same for year 1, except for alternatives 3a and 3b where the deposition volumes at year 3 were assumed the same as for 2a and 2b, respectively.

3. From year 3 to year 50, the equilibrium deposition rates were assumed.

The expected deposition for each alternative is given in Table 9.10, p. 258.

No Action Alternative

Matilija Reservoir would continue to fill with sediment. It is expected that the reservoir pool would be almost completely gone in 10 years. After the reservoir pool is gone, there would be no trapping of fine material. Coarse sediment would begin to pass over the top of the dam around 2040. The increase in sediment loads may affect the diversion at Robles Diversion through three possible mechanisms.

1. Deposition in Robles Basin – Deposition at the entrance to the canal may prevent some of the water from entering the diversion canal. The coarse sediment loads would be expected to reach equilibrium around 2040. Under equilibrium conditions, it is estimated that any flood with a return period larger than 3 to 4 years would deposit 40,000 yd^3 or more of material behind Robles Diversion. When 40,000 yd^3 or more of sediment deposit behind Robles Diversion, CMWD has problems continuing diversion.

2. Increase in turbidity – Currently, there is only a small amount of trapping of fine material behind the dam. After the reservoir pool fills with sediment, the fine sediment load would be expected to increase by approximately 30% under equilibrium conditions. Fine sediment concentrations vary by a factor of two or more under natural conditions, so an increase of 30% is not considered large.

3. Deposition in Robles Canal and/or at Fish Screen – The excessive quantities of sand may not be transported through the fish screen area. Sand generally travels as suspended load in the river and it would be possible that large quantities of sand are transported into the canal. Once they reach the fish screen, it is possible that they would deposit due to the reduced velocities there. The increase in sand loads would cause increased maintenance. Because the fish screen facility is new, its ability to function under high sediment load is

difficult to determine. The sand loads are expected to increase until they reach equilibrium values at around 2040.

Mechanical Removal/Permanent Stabilization (1, 4a):

The increase in sediment loads due to the re-supply of sediment may affect the diversion at Robles Diversion through three possible mechanisms.

1. Deposition in Robles Basin – Deposition at the entrance to the canal may prevent some of the water from entering the diversion canal. The Mechanical Removal and Permanent Stabilization Alternatives allow the natural sediment loads from Matilija Creek to pass down Ventura River and the coarser fractions of the sediment (coarse sand, gravel, and cobbles) may deposit behind Robles Diversion. The sediment deposition is expected to increase by a factor of 2 to 3 over current conditions. Under equilibrium conditions, it is estimated that any flood with a return period larger than 3 to 4 years would deposit 40,000 yd^3 or more of material behind Robles Diversion. When 40,000 yd^3 or more of sediment deposit behind Robles Diversion, CMWD has problems continuing diversion. A sediment bypass would limit the amount of deposition at the Robles Diversion and would limit the possibility of the diversions at Robles being affected. The deposition amounts with a high flow bypass should be similar to the current condition.

2. Increase in turbidity – before the first flood, the deconstruction of the dam and the mechanical removal of sediment would introduce fine sediment into the river system. In addition, it would be impossible to removal all the fine sediment from the system. The residual sediment would be easily mobilized by the first flows that pass through the reservoir area. These river flows would likely carry high concentrations of sediment until the first flood flow 'cleans' out the reservoir area. However, the turbidity increase should be relatively minor and of short duration. After the first flood, the sediment concentrations would be near equilibrium concentrations. The equilibrium concentrations may be approximately 30% higher than current concentrations. Fine sediment concentrations vary by a factor of two or more under natural conditions, so an increase of 30% is not considered large.

3. Deposition in Robles Canal and/or at Fish Screen – The excessive quantities of sand may not be transported through the fish screen area. Sand generally travels as suspended load in the river and it is possible that large quantities of sand would be transported into the canal. Once they reach the fish screen, it is possible that they would deposit due to the reduced velocities there. The increase in sand loads would cause increased maintenance. Because the fish screen facility is new, its ability to function under high sediment load is difficult to determine. However, no significant water loss is expected for Alternatives 1 and 4a.

Based on the analysis of these three factors, if a sediment bypass is installed, the diversion capability of CMWD should not be adversely affected.

Natural Transport Alternatives with Removal of Reservoir Fines (2a, 3a):

The increase in sediment loads due to the release of sediment would affect the diversion at Robles Diversion through three possible mechanisms.

1. Deposition in Robles Basin – Deposition at the entrance to the canal may prevent some of the water from entering the diversion canal. The Natural Transport Alternatives release significant amounts of sediment downstream and the coarser fractions of the sediment (coarse sand, gravel, and cobbles) may deposit behind Robles Diversion. The first floods that occur after dam removal would cause excessive deposition at Robles Diversion. For example, under alternative 2a, the 1991 flood deposits approximately 90,000 yd^3 of material behind Robles Diversion (Table 9.10). Alternative 3a is expected to cause much less deposition with the first notch, but the final notch would produce as much deposition as Alternative 2a. A sediment bypass would limit the amount of deposition at the Robles Diversion.

2. Increase in turbidity – An increase in turbidity may cause water quality problems in Lake Casitas and may increase water treatment costs. Based on the average detention time of water in the reservoir (approximately 8 yrs) it is expected that most of the silt and sand sized sediment would deposit near the outlet of Robles Canal into Casitas Reservoir and would not reach the intakes for the treatment plant. However, some small amounts of clay and organic matter may stay in suspension indefinitely in Casitas reservoir. Further study into the clay material deposited in Matilija reservoir is needed to assess this. Clays account for approximately 5% of the material in the delta of Matilija Reservoir. For Alternatives 2a and 3a, the duration of excessive turbidity is expected to be a matter of days as soon as flow is returned to the reservoir area. See section 9.2 Sediment Concentrations for descriptions of the sediment concentrations. If the temporary increase in turbidity is deemed unacceptable, a de-silting basin could be constructed that would settle out fine sediment. However, because the increase in fine sediment concentration expected to be confined to the first few floods, a permanent de-silting basin may not be justified.

3. Deposition in Robles Canal and/or at Fish Screen – The excessive quantities of sand may not be transported through the fish screen area. Sand generally travels as suspended load in the river and it is possible that large quantities of sand would be transported into the canal. Once they reach the fish screen, it is possible that they would deposit due to the reduced velocities there. If a flood similar to the 1991 flood occurs immediately after dam removal for alternative 2a, the amount of sand entering the canal would be approximately 10 times the amount under equilibrium conditions (Figure 10.6). The increase in sand loads would cause increased maintenance at the fish screen facility. Because the fish screen facility is new, its ability to function under high sediment load is difficult to determine. However, the water loss due to excessive deposition in the fish screen area should be temporary if sufficient means to excavate sediment from the fish screen facility are available.

Based on the analysis of these three factors, if a sediment bypass is installed, the diversion capability of CMWD should not be adversely affected. However, because of the large sand loads, the increased maintenance required to keep the fish screen area clear may be significant.

Natural Transport Alternatives with Natural Erosion of Reservoir Fines (2b, 3b):

The increase in sediment loads due to the release of sediment would affect the diversion at Robles Diversion through the same three mechanisms listed above. However, the magnitude of the impacts would be greater. In particular, the increase in turbidity would be much greater.

1. Deposition in Robles Basin – Deposition at the entrance to the canal may prevent some of the water from entering the diversion canal. The Natural Transport Alternatives release significant amounts of sediment downstream and the coarser fractions of the sediment (coarse sand, gravel, and cobbles) may deposit behind Robles Diversion. The first floods that occur after dam removal would cause excessive deposition at Robles Diversion. For example, under alternative 2b, the 1991 flood deposits approximately 100,000 yd^3 of material behind Robles Diversion (see Table 9.10). Alternative 3b there would be expected to be 70,000 yd^3 of deposition. A sediment bypass is recommended as a measure to limit the amount of deposition at the Robles Diversion.

2. Increase in turbidity – An increase in turbidity may cause water quality problems in Lake Casitas and may increase water treatment costs. Based on the average detention time of water in the reservoir (approximately 8 yrs) it is expected that most of the silt and sand sized sediment would deposit near the outlet of Robles Canal into Casitas Reservoir and would not reach the intakes for the treatment plant. However, some small amounts of clay and organic matter may stay in suspension indefinitely in the reservoir. Further study into the clay material deposited in Matilija reservoir is needed to assess this. Clays account for approximately 30% of the material in the reservoir region. The increase in turbidity would be of long duration. Sediment concentrations may be higher than 100,000 mg/l during the first events that occur after dam removal. It is expected that after 2 to 3 floods, the sediment concentrations would still be higher than equilibrium conditions but only between 2 to 10 times higher. A de-silting basin would reduce sediment loads before they enter Lake Casitas.

3. Deposition in Robles Canal and/or at Fish Screen – The excessive quantities of sand may not be transported through the fish screen area. Sand generally travels as suspended load in the river and it is possible that large quantities of sand would be transported into the canal. Once they reach the fish screen, it is possible that they would deposit due to the reduced velocities there. If a flood similar to the 1991 flood occurs immediately after dam removal for alternative 3b, the amount of sand entering the canal would be approximately significantly higher than under equilibrium conditions. The increase in sand loads would cause increased maintenance at the fish screen facility. In addition, the first few floods may cause such excessive deposition, that diversion would be impossible. The diversion under these conditions may have to be shut down.

Without mitigation measures, it is most likely that diversion would not be possible for the first 2 to 3 floods after removal for both Alternative 2b and 3b. With both a sediment bypass and a desalting basin, diversions may be possible, but it is also possible that deposition in the fish screen area would be so excessive that diversion would be impossible.

To plan for the missed diversions that would be possible following dam removal, the safe yield of Casitas Reservoir may be reduced. Entrix (2002) determined the safe yield starting in 2009 to be 21,500 ac-ft/yr. If no mitigation measures are constructed, it is possible that CMWD would not be able to divert water until most of the sediment is eroded from the reservoir. Based on the computer simulations of the removal, it would take one very large flood, or several smaller floods to erode the reservoir sediments. If these missed diversions are imposed on the safe yield estimates of Entrix, it is possible to calculate the reduction in the safe yield of Casitas Reservoir.

The revised safe yield is developed by using the period starting in 1944, as this is the driest period for which there are stream flow measurements. Beginning in 1944, it is assumed that diversions are missed from 1944 through 1952. There were only three years of significant diversions during this period: 1945, 1946, and 1952. It is assumed that after 1952, most of the reservoir sediments would be eroded and that CMWD would be able to restore diversions during the wet seasons. To meet the same safe yield criteria as was used in the Entrix report, the yield from Casitas Reservoir would have to be reduced by 6,000 ac-ft each year for a period of 8 years, for a total of 48,000 ac-ft (Figure 9.11).

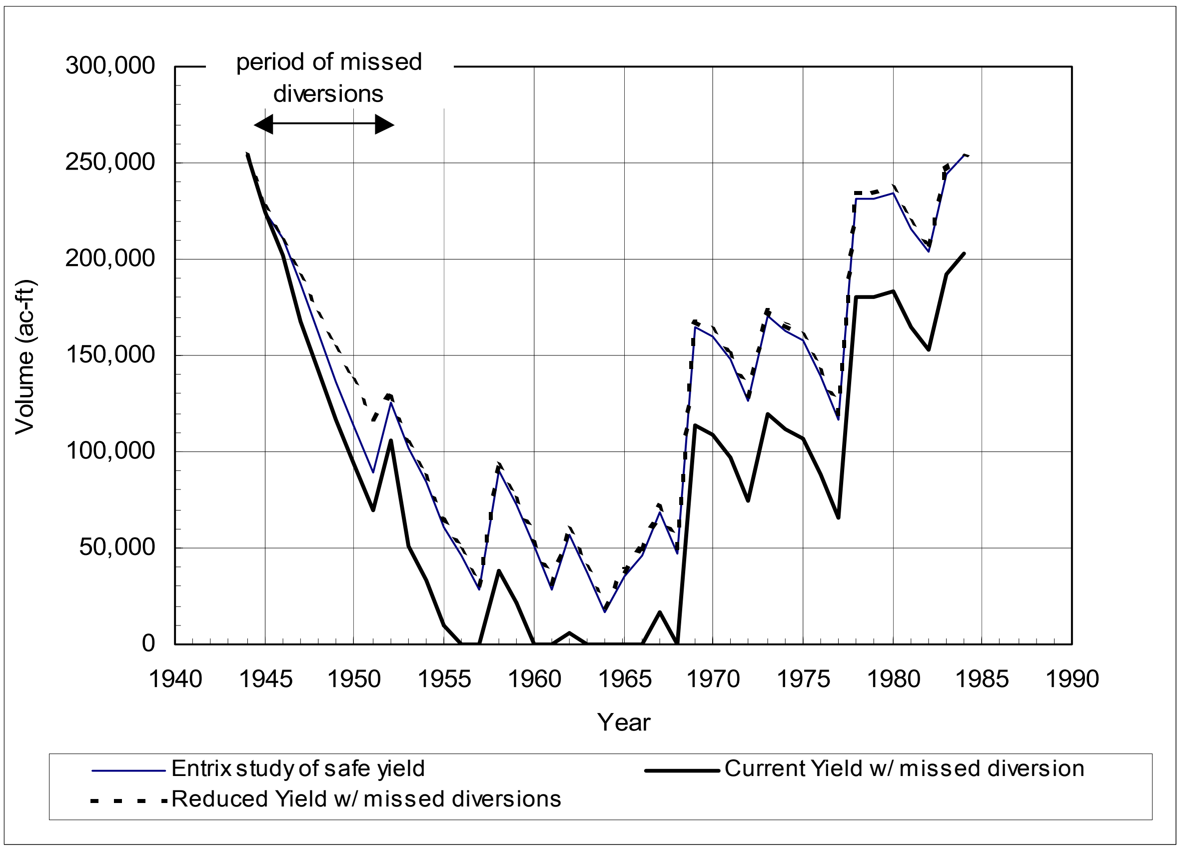


Figure 9.11. Estimate of Casitas Reservoir Volume during drought period for Natural Transport Alternatives 2b and 3b. The thin line is the reservoir volume under the current conditions and with the current safe yield. The dashed line is the volume in Casitas Reservoir during the Natural Transport Alternative 2b or 3b if the safe yield is reduced by 6,000 ac-ft/yr for a period of eight

years. The thick solid line is the reservoir volume during the Natural Transport Alternative 2b or 3b if the safe yield is not reduced.

Temporary Stabilization of Sediments (4b):

The increase in sediment loads due to the release of sediment would affect the diversion at Robles Diversion through three possible mechanisms.

1. Deposition in Robles Basin – Deposition at the entrance to the canal may prevent water from entering the diversion canal. The temporary stabilization alternative releases significant amounts of sediment downstream and the coarser fractions of the sediment (coarse sand, gravel, and cobbles) may deposit behind Robles Diversion. A high flow diversion is recommended as a measure to limit the amount of deposition at the Robles Diversion.

2. Increase in turbidity – An increase in turbidity may cause water quality problems in Lake Casitas and may increase water treatment costs. Based on the average detention time of water in the reservoir (approximately 8 yrs) it is expected that most of the silt and sand sized sediment that enters the reservoir would deposit before entering the intakes for the treatment plant. However, some small amounts of clay and organic matter may stay in suspension indefinitely if it has become bonded to organic material in the reservoir. Further study into the clay material deposited in Matilija reservoir is needed to assess this. Clays account for approximately 5% of the material in the delta of Matilija Reservoir. The magnitude of the sediment concentration increases would most likely be about 2 to 4 times greater than natural conditions before the removal of the first revetment. After the first revetment would be removed, the concentrations may temporarily increase between by a factor of 2 to 10 times the current condition. If the temporary increase in turbidity is deemed unacceptable, a de-silting basin could be constructed that would settle out fine sediment. However, because the increase in fine sediment concentration expected to be confined to the first few floods and following removal of revetment, a permanent de-silting basin may not be justified. The increase in fine sediment concentrations is not expected to affect water supply to Lake Casitas.

3. Deposition in Robles Canal and/or at Fish Screen – Excessive quantities of sand may not be transported through the fish screen area. Sand generally travels as suspended load in the river and it is possible that large quantities of sand would be transported into the canal. Once they reach the fish screen, it is possible that they would deposit due to the reduced velocities there. The increase in sand loads would cause increased maintenance at the fish screen facility. Because the fish screen facility is new, its ability to function under high sediment load is difficult to determine. However, no significant water loss is expected for Alternative 4b.

Based on the analysis of these three factors, if a sediment bypass is installed, the diversion capability of CMWD should not be adversely affected. However, because of the large sand loads, the increased maintenance required to keep the fish screen area clear may be significant.

This alternative constructs a channel that would allow the low flows to pass down stream without picking up sediment. However, high flows and following removal of the bank revetment, there may be fine sediment mobilized. Because there would be multiple removals of stabilization structures, there would be multiple impacts of fine sediment. After each removal, there would be some fine sediment released into the river as the flood flow passes through the area. The fine sediment would be mobilized as the banks are eroded. As the flood recedes, the water elevation would recede from the banks and no longer erode the fine sediment. Therefore, the increases in turbidity would be mostly confined to the flood events and the lows flows would not experience large increases in turbidity. The magnitude of the sediment concentration increases would most likely be about 2 to 4 times greater than natural conditions before the removal of the first revetment. After the first revetment is removed, the concentrations may temporarily increase between by a factor of 2 to 10 times the current condition. Each subsequent removal of revetment would produce similar increases in turbidity. The time required for the sediment to be transported downstream would be a function of the amount of sediment eroded from the banks. The physical processes would be quite complicated and difficult to simulate accurately. After the final removal of revetment, the turbidity levels should stabilize at natural conditions after one or two floods of average size pass through the reservoir area.

Summary of Robles Diversion Effects

Table 9.10 summarizes the estimated deposition at the diversion at Robles. Alternatives 2a, 2b, 3b, and 4b cause more than 40,000 yd^3 of deposition the first year. This amount of deposition could potentially reduce water diversion volumes. Alternative 3a would cause more than 40,000 yd^3 of deposition following complete removal of the dam. For Alternatives 1 and 4a, the deposition would be less than 40,000 yd^3; however, the risk to water supply would be still significantly increased. Floods larger than a 3 to 5 year flood could affect the ability to divert at Robles Diversion. Therefore, a sediment bypass would be recommended for all alternatives to limit the deposition rates and to reduce the risk to the water supply at Robles Diversion.

Table 9.11 summarizes the expected water loss at Robles Diversion for various alternatives. With no mitigation, it would be possible that deposition under the equilibrium conditions would increase at Robles and approximately every 5 years, deposition would exceed 40,000 yd^3. This amount of deposition may hinder their diversion capability. As a rough estimate, this deposition could cause of loss of 6,000 ac-ft on an annual basis. Installing a sediment bypass would be expected to eliminate the water loss due to deposition behind Robles Diversion. However, for alternatives 2b and 3b, the excessive silt and clay loads that occur after dam removal would still create missed diversions. This missed diversion would be expected to be of limited duration and the total water loss would be approximately 48,000 ac-ft. It would be uncertain if a desilting basin would eliminate water loss for alternative 2b or 3b because of problems at the fish screen facility. This would be because of the possibility of the high sediment loads clogging the fish screen area. The water loss due to excessive deposition would be less than 48,000 ac-ft.

Table 9.10. Summary of Estimated Deposition at Robles Diversion. Results are from 50-yr simulation. These numbers are without a sediment bypass.

	Year			
Location	**1**	**3**	**10**	**50**
No Action	20,000	39,900	133,000	665,000
Mechanical Removal/Permanent Stabilization (1, 4a)	34,000	94,600	266,000	1,330,000
Alternative 2a	70,000	210,000	396,200	1,460,200
Alternative 2b	80,000	240,000	426,200	1,490,200
Alternative 3a	27,000	210,000	396,200	1,460,200
Alternative 3b	70,000	240,000	426,200	1,490,200
Alternative 4b	70,000	210,000	396,200	1,460,200

Table 9.11. Expected Water Loss at Robles Diversion for Each Alternative with Various Mitigation Measures.

Alternative	No Mitigation	With High Flow bypass	With High Flow bypass and De-silting Basin
No Action	0	0	0
Alternative 1, 4a	~ 6,000 ac-ft/yr	0	0
Alternative 2a, 3a, 4b	~ 6,000 ac-ft/yr	0	0
Alternative 2b, 3b	~ 6,000 ac-ft/yr	48,000 ac-ft	0 to 48,000 ac-ft

9.4. Foster Park Diversion Impacts

Currently, the diversion at Foster Park is a combination of surface diversion and subsurface wells. ENTRIX (1997) states that on average 2,500 ac-ft of surface water and 3,900 ac-ft of groundwater is diverted at Foster Park annually. The surface diversion is actually a combination of an above ground surface diversion and an intake that is approximately 4 feet below the riverbed. The subsurface wells are approximately 50 feet deep. As shown in Figure 1.12, the surface diversion decreased after 1993 and more water is taken from the groundwater wells. The surface bed material at Foster Park is generally large cobbles with a small amount of sands. Therefore, the hydraulic conductivity of the bed material is very large and the subsurface diversions are not limited by the infiltration rates into the bed.

Because Foster Park is located approximately 10 miles from the dam, the present effect of Matilija Dam on the sediment loads there is small. Both North Fork Matilija Creek and San Antonio Creek enter the Ventura River between Matilija Dam and Foster Park. In addition, there is a large sediment supply from the banks of the riverbed between Robles Diversion and San Antonio Creek. Therefore, there are presently very high sediment concentrations that occur at Foster Park. The City of Ventura and the non-profit group Surfriders have collected turbidity samples at Foster Park (Figure 9.12).

The City of Ventura presently discontinues surface diversion when the turbidity rises above 10 NTU in the Ventura River. The data from flow duration curve was used along with Figure 9.13 to compute the total fraction of time the City of Ventura Water Treatment Plant cannot divert from its surface diversion. The computation is shown in Table 9.12. Under current conditions and for the average year, the surface diversion is shut down approximately 4.6% of the time, or about 17 days per year on average.

The City of Ventura provided the daily average flows for the period from 1984 until 2002 for the shallow intake and the period from 1991 until 2000 for the above ground surface diversion. The maximum recorded daily diversion at the shallow intake was 8.60 MGD (13.3 cfs), and 8.64 MGD (13.4 cfs) at the surface diversion. The average diversion for the shallow intake was 1.2 MGD (1.8 cfs) and was 1.8 MGD (2.9 cfs) for the surface diversion. To estimate the amount of water not diverted due to high turbidity, the 90% percentile of diversion flow was calculated. The 90% percentile was used as a representative diversion during the high flows that are linked to high turbidity. The 90% percentile diversion is 2.5 cfs for the shallow intake and 4.6 cfs for the surface diversion.

Table 9.12. Computation of Fraction of Time Turbidity would exceed 10 NTU under current conditions.

C1 Flow (cfs)	C2 % non-exceed	C3 fraction in flow bin	C4 Fraction of Readings exceeding 10 NTU	C5 Fraction of Time > 10 NTU (C3 * C4)
0 to 1	0	0.378	0.000	0.0000
1 to 5	37.8	0.185	0.005	0.0010
5 to 10	56.3	0.117	0.020	0.0023
10 to 30	67.9	0.140	0.007	0.0010
30 to 100	82.0	0.106	0.115	0.0123
100 to 300	92.6	0.036	0.212	0.0077
300 to 1000	96.2	0.025	0.441	0.0110
1000 to 3000	98.7	0.010	0.750	0.0072
3000 to 30000	99.7	0.003	1.000	0.0035
Total				**4.6%**

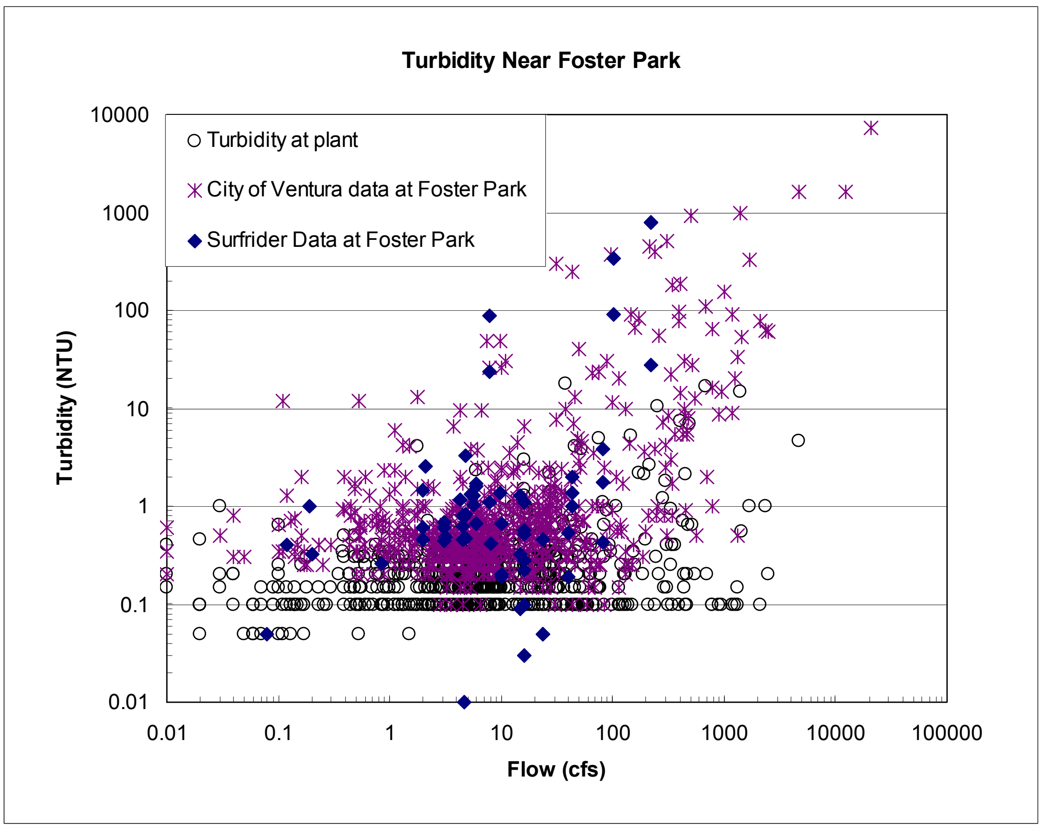


Figure 9.12. Current Turbidity and Sediment Concentration in the Ventura River at Foster Park.

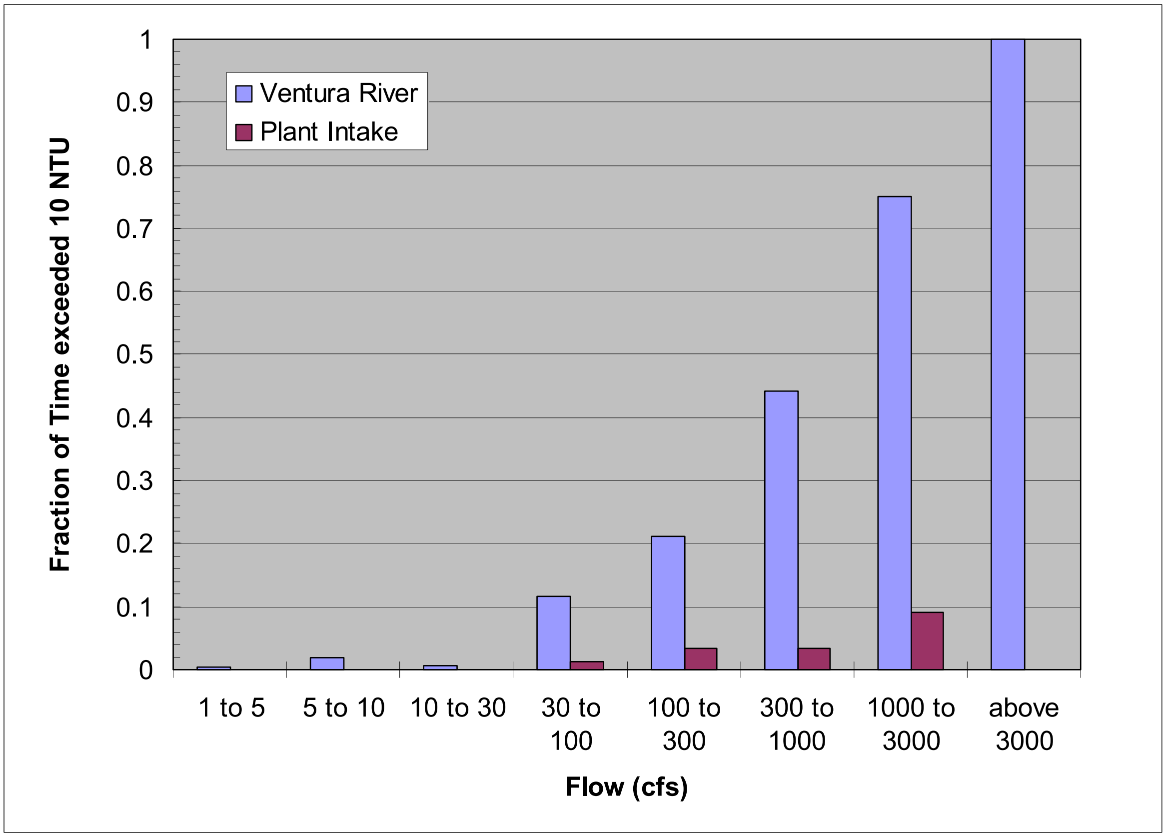


Figure 9.13. Fraction of Time 10 NTU Criteria is Exceeded in Ventura River and at City of Ventura Water Treatment Plant Intake under Without Project Conditions.

The increase in sediment loads due to the release of sediment would affect the diversion at Foster Park Diversion through two possible mechanisms.

1. Increase in Fine Sediment Concentrations – An increase in fine sediment concentration would increase the turbidity and increase the time at which they are unable to divert. The sediment concentration is related to turbidity, but the relation may not be linear. Therefore, doubling the fine sediment concentration may more than double the turbidity.

2. Decrease in Infiltration Rates – If large sediment concentrations exist at low flows (less than 50 cfs), it is possible that as the water is pumped from the subsurface wells, and the riverbed may become clogged with sediment. This could only occur until the next high flows mobilize the sediment, but during this period, the yield from the subsurface wells may be reduced. For this to occur, however, the infiltration throughout the entire Ventura River would have to be reduced. This is not deemed possible, for as soon as this occurs surface flow would occur that then erodes the fine material from the bed. Therefore, infiltration into the bed would always occur. The aquifer is connected to the River throughout its entire length and there is a groundwater dam just downstream of the diversion that forces water to the surface.

The impact to the diversion for each alternative is discussed below. For each alternative, a range of potential impact was estimated to capture the uncertainty in the estimates. Only one estimate was calculated for the No Action Alternative as it was assumed that only the differences from the No Action Alternative were critical to this study. The lower bound of the impact is given in Table 9.13 and an upper bound in given in Table 9.14.

To calculate the impact, the concentration was assumed to increase over the current condition by some multiple. The multiple of concentration increase was based upon the model results presented in Section 9.2 and Section 19. The concentrations simulated before and after flood events were compared against the results for the No Action Alternative for a variety of floods. The simulations were performed for a series of floods occurring back to back, so that the effect of the decrease in concentrations following successive floods could be estimated. Once the multiple of concentration increase was determined, it was assumed that the relative concentration increase would be proportional to the increase in turbidity.

It is estimated that floods with a peak flow of over 3000 cfs would be sufficient to move significant amounts of sediment from the reservoir. Such floods occur every 2.7 years on average and therefore it would be assumed that floods occur every 3 years for the following tables. As previously discussed, a representative diversion rate is 2.5 cfs for the shallow intake and 4.6 cfs for the surface diversion. Therefore, for every day of missed surface diversion, approximately 14 ac-ft of water is not diverted.

<u>No Action</u>

In the future, the fine sediment concentrations would not be significantly different from the present condition. For a period of 15 years, the estimated amount of missed surface diversion would be 3600 ac-ft.

<u>Mechanical Removal/Permanent Stabilization (No Action, 1, 4a)</u>:

For the Mechanical Removal, Permanent Stabilization, the fine sediment concentrations after the first flood events would not be significantly different from the present condition. Therefore, it would be expected that the period of no surface diversion would remain at approximately 17 days per year over the long term. However, before the first flood, the deconstruction of the dam and the mechanical removal of sediment would introduce fine sediment into the river system. In addition, it would be impossible to removal all the fine sediment from the system. The residual sediment would be easily mobilized by the first flows that pass through the reservoir area. These river flows would likely carry high concentrations of sediment until the first flood flow ‘cleans’ out the reservoir area. Therefore, as a lower bound on this impact it is assumed that the turbidity before the first flood would be elevated by a factor of two. As an upper bound, it is estimated that the turbidity would increase by a factor of four for the first three years. This gives an estimated volume of missed surface diversions of 3870 to 4410 ac-ft, or 270 to 810 ac-ft more than the No Action Alternative.

As part of Alternative 1 and 4a, there may be fine material place in the flood plain upstream of Foster Park. If this material would be allowed to erode, the impact to Foster Park would be affected.

Natural Transport Alternatives with Removal of Reservoir Fines (2a, 3a):

The delta region consists of approximately 25 % silt and 5 % clay. This material would be allowed to travel downstream. The first floods would carry increased fine sediment concentrations. See section 9.2 Sediment Concentrations for descriptions of the sediment concentrations. The sediment concentrations would be expected to be two to three times higher than current conditions following the first few floods after dam removal. During this period of impact, the time at which surface diversions do not occur because of high sediment concentrations would increase. However, after this time, the sediment concentrations would decrease to current levels.

Based on the modeling, the turbidity is estimated to be between 2 to 10 times higher before the first flood peak passes through the reservoir area. After the first flood, the turbidity would be 2 to 4 times larger than natural conditions.

To estimate the upper bound of the impacts it would be first assumed that the turbidity would be 10 times higher before and after the first flood. The turbidity would then decrease to 4 times higher than current conditions until two more floods pass through the reservoir area. Then after approximately 12 years, the turbidity decrease to twice the current levels. After 15 years, the turbidity would be back to current levels.

As a lower bound of impacts, it would be estimated that the turbidity would be 4 times higher before the first flood peak arrives. After the first flood peak arrives the turbidity decreases to twice the current levels for a period of six more years. Then, by year 10, the turbidity levels would be at current levels.

The upper and lower bounds on the volume of missed surface diversions are 7710 and 4680 ac-ft, respectively.

Natural Transport Alternatives with Natural Erosion of Reservoir Fines – One Notch (2b):

The increase in sediment loads due to the release of sediment would affect the diversion at Robles Diversion through the same two mechanisms listed above. However, the magnitude of the impacts would be greater. In particular, the increase in turbidity would be much greater. Because there would be over 2 million yd^3 (1240 ac-ft) of fine sediment available for transport, it would be possible impacts would last much longer. In section 9.2 "Sediment Concentrations" the sediment concentrations were up to 100,000 mg/l for the period immediately following dam removal. It is expected that until a flood occurs, diversion at Foster Park would not be possible when the flow in Ventura River is above 30 cfs. A flow of 30 cfs is chosen because that is the expected capacity of a low flow diversion around the fine sediments of Matilija Dam. If this low flow diversion around the Matilija sediments continues to operate until the first floods occur, then the flows below 30 cfs would be relatively free of sediment. A flow of 30 cfs is exceeded 12 % of the time at Matilija Dam. Including the possibility that flows below 30 cfs are naturally above 10 NTU at times, gives an estimate that the diversion at Foster Park is expected to be not possible 18 % of the time (66 days) following dam removal. The upper bound estimate assumes that high turbidity continues shut down surface diversions for a period of 6 years when the flow is above 30 cfs. The lower bound estimate assumes that this period lasts 3 years.

For the lower bound estimate, it is assumed that the after the first flood the turbidity levels would be approximately 10 times the current condition. After the second flood, the turbidity levels decrease to twice the current condition. After the third flood, the turbidity would be back to current levels.

For the upper bound estimate, it is assumed that the turbidity levels would be 10 times the current condition after the second flood. They are not assumed to decrease to current conditions until year 15.

The upper and lower bounds on the volume of missed surface diversions are 7380 and 10050 ac-ft respectively.

<u>Natural Transport Alternatives with Natural Erosion of Reservoir Fines – Two Notches (3b)</u>:

The impacts of Alternative 3b would be similar to Alternative 2b, except that high turbidity would occur after each notching. Therefore, it is expected that after the first and second floods, the turbidity would cause the surface diversion to be shut down 18% of the time. The 30 cfs diversion would be still assumed to be operating. The return to equilibrium conditions would occur in approximately the same time. As a lower bound estimate, the turbidity levels would return to current conditions after the third flood. As an upper estimate, the turbidity levels would return to current conditions after the 5 floods, or approximately 15 years.

<u>Temporary Stabilization of Sediment (4b)</u>:

The delta region consists of approximately 30 % silt and clays. This material would be allowed to travel downstream whenever the temporary stabilization structures would be overtopped or removed. It is estimated that there would be four separate removals of revetment. In addition, it is assumed that one flood would pass through the reservoir before any revetment would be removed. The residual sediment that would be left in the constructed channel and in the areas that would be unprotected may increase the turbidity before the first flood.

As a lower bound on the impact, the turbidity levels are assumed approximately twice-current levels until a flood passes through the area after the final removal of revetment. This would mean that approximately 15 years would pass before the turbidity levels decrease to current levels.

As an upper bound on the impact, the turbidity levels are assumed to increase by a factor of 10 for a period of 9 years following dam removal. After year 9, it is assumed that the turbidity levels decrease to approximately four times the current levels until year 15. After year 15, the turbidity levels decrease to current conditions. The large difference between the lower and upper bounds is justified based upon the uncertainty associated with the bank erosion mechanics in the reservoir area as well as the uncertainty of the hydrology. For example, a large short flood may erode a large portion of the bank but not carry this sediment all the way past Foster Park. Smaller flows may then erode this sediment and prolong the turbidity impact. The increase in sediment concentration would be controlled by the rate at which the revetment would be removed. The upper and lower bounds on the volume of missed surface diversions are 8820 and 4950 ac-ft, respectively.

Summary of Foster Park Diversion Impacts

It is estimated that floods with a peak flow of over 3000 cfs would be sufficient to move significant amounts of sediment from the reservoir. Such floods occur every 2.7 years on average and therefore it would be assumed that floods occur every 3 years for the following tables. As previously discussed, a representative diversion rate is 2.5 cfs for the shallow intake and 4.6 cfs for the surface diversion. Therefore, for every day of missed surface diversion, approximately 14 ac-ft of water would be not diverted.

Table 9.13. Lower Bound Estimated Annual Surface Water Loss for Each Alternative at Foster Park Diversion for a period of 15 years.

	Annual Water Not Diverted due to High Turbidity (ac-ft/yr)					
Alternative	**Years 1 to 3**	**Year 4 to 6**	**Years 7 to 9**	**Years 10 to 12**	**Years 13 to 15**	**TOTAL (ac-ft)**
No Action	240	240	240	240	240	3600
Alternative 1, 4a	330	240	240	240	240	3870
Alternative 2a, 3a	420	330	330	240	240	4680
Alternative 2b	950	700	330	240	240	7380
Alternative 3b	950	700	330	240	240	7380
Alternative 4b	330	330	330	330	330	4950

Table 9.14. Upper Bound of Estimated Annual Surface Water Loss for Each Alternative at Foster Park Diversion.

	Annual Water Not Diverted due to High Turbidity (ac-ft/yr)					
Alternative	**Years 1 to 3**	**Year 4 to 6**	**Years 7 to 9**	**Years 10 to 12**	**Years 13 to 15**	**TOTAL (ac-ft)**
No Action	240	240	240	240	240	3600
Alternative 1, 4a	420	330	240	240	240	4410
Alternative 2a, 3a	700	700	420	420	330	7710
Alternative 2b	950	950	700	420	330	10050
Alternative 3b	950	950	950	700	420	11910
Alternative 4b	700	700	700	420	420	8820

9.5. Impacts to Groundwater Use

There is approximately 6 million yd^3 of sediment behind Matilija Dam. For all alternatives, the sediment transport modeling shows that the release of this material would not substantially change the composition of the Ventura River Bed. The silts and clays would not deposit onto the river bed. The only material that would deposit on the river bed is cobble, gravel and some sand sized sediment. The Ventura River has a large capacity to transport sediment because of its steep slope (over 1%) and high flows. In fact, the Ventura River transported over 4,000,000 yd^3 of sediment in less than 1 month in 1969 (Figure 5.2). More detailed discussion of the impacts to groundwater for each alternative is given below.

For all alternatives, the removal of Matilija Reservoir would not affect the production from the wells upstream of Matilija Dam. The well elevations are much above the elevation of the Matilija reservoir (Table 3.1). Therefore, the wells not supplied by water infiltrating from Matilija Reservoir but are supplied by infiltration from Matilija Creek.

No Action: This alternative leaves the dam in place and continues to trap sediment. Fine sediment is already passing over the top of the dam and would continue to pass over the top. However, the fine sediment being transported in the Ventura River does not deposit in the river bed. The groundwater supply in the future is expected to be the same as under current conditions.

Mechanical Removal/Permanent Stabilization (1, 4a): This alternative removes the dam, but does not return the reservoir sediments to the river system. Fine sediment is already passing over the top of the dam and this alternative would also allow the natural supply of fine sediment to pass downstream. The groundwater supply in the future is expected to be the same as under current conditions.

Natural Transport (2a, 3a): Of the 3.9 million yd^3 of sediment allowed to travel downstream, approximately 1 million yd^3 is silt and clay, 1.8 million yd^3 is sand, and 1 million yd^3 is gravel and cobble. The silt and clays are mixed in with the coarser material. All this sediment would be eroded by natural flows as soon as the dam is removed. The sediment transport modeling to date shows that the gradual release of this material would not substantially change the composition of the Ventura River Bed. The silts and clays would not deposit onto the river bed. Therefore, silt and clay would not enter into the groundwater aquifer.

The ability of the river to transport large amounts of fine sediment is also evidenced by the fact that there is currently almost no silt and clay present in the bed of the Ventura River despite large amounts of fine sediment being transported by the river. The current bed material composition is given in Section 5.3. The sediment concentrations on the main stem of the Ventura River have been measured at over 20,000 mg/l during flood events and are commonly over 10,000 mg/l (Figure 8.11). These high concentrations are evidence of a large supply of fine sediment in the watershed, even with Matilija Dam in place. In addition, because the Matilija Reservoir is almost full, most of the fine sediment that enters the reservoir from the upstream end passes over the dam. While the release of additional sediment would increase the natural sediment loads, the river has a large capacity to transport this fine sediment all the way to the ocean. Also, because the preferred alternative only releases fine sediment during flood flows, the low flows would not carry additional fine sediment.

The disposal sites would not affect the percolation of water from the bed of the Ventura River into the Upper Ventura Aquifer. As mentioned above, the primarily supply of water to the aquifer is percolation of water from the Ventura River. It is estimated that no significant recharge to the aquifer occurs from rain falling on the floodplain and then percolating into the aquifer. The average rainfall in this area is approximately 20 inches of rain per year, but can be highly variable. Some of the rain that falls onto the disposal site would run off into the river because the infiltration rate of the disposal site is small. The rain the infiltrates into the disposal sites would most likely eventually evaporate. Based upon measurements of pan evaporation in the Santa Clara River Basin the evaporation potential is more than 60 inches per year (United Water Conservation District, 2001), which is much larger than the annual rainfall.

The slurry disposal sites would be composed of primarily silt and clay, it would not allow rainwater falling on the disposal site to infiltrate into the groundwater. The fine sediment in the disposal sites would act as a seal on top of the aquifer preventing water from entering the aquifer from the disposal. Therefore, anything that is present in the disposal site sediment would not enter the aquifer below. Furthermore, there has already been extensive testing of the reservoir sediment.

<u>Natural Transport - No Removal of Fines (2b, 3b)</u>:

This alternative returns all the stored sediment to the river system. In addition, the natural supply of sediment is restored to the downstream reach. The large and sudden increase in sediment load would cause the downstream channel to aggrade quickly. Within a few years, 50 years of degradation would be reversed by the sudden over-supply of sediment. Because of the aggradation in the river channel, most of the sediment coarser than gravel stored behind the dam would not reach the ocean. However, most of the sands, silts, and clays would reach the ocean.

Because low flows would be eroding the fine sediment, it would be possible the some fine sediment would be temporarily stored in the slow velocity areas of the Ventura River. The sediment in these slow moving areas may temporarily cover the coarser bed material. However, the fine sediment would not deposit in the main current of the stream. Therefore, the overall recharge to the Upper Ventura Aquifer would not be significantly affected. Additional studies on the current percolation rates would be required to assess the impacts further.

<u>Temporary Stabilization (4b)</u>:

Of the 3.9 million yd^3 of sediment allowed to travel downstream, approximately 1 million yd^3 is silt and clay, 1.8 million yd^3 is sand, and 1 million yd^3 is gravel and cobble. The silt and clays are mixed in with the coarser material. All this sediment would be gradually eroded by large floods as the temporary revetments would be removed. The sediment transport modeling to date shows that the gradual release of this material would not substantially change the composition of the Ventura River Bed. Plots of the d_{16}, d_{50} and d_{50} are given in Exhibit G, Section 19.4.5. The d_{16} is the diameter of which 16% of the sediment in the bed is finer than. The release of sediment from behind the dam does cause the bed to become slightly finer, but the bed still remains coarse and composed primarily of cobbles and gravel. In addition, the bed would eventually return to very near current conditions. The d_{16} would be greater than 6 mm for all times after dam removal in all reaches upstream of River Mile 2. In most reaches the d_{16} would be above 10 mm for all times above River Mile 2. The d_{35} would be above 35 mm for all reaches above River Mile 2 for all times after dam removal. The d_{50} remains above 60 mm for all reaches above River Mile 2 for all times after dam removal. The silts and clays would not deposit onto the river bed. Therefore, silt and clay would not enter into the groundwater aquifer.

The ability of the river to transport large amounts of fine sediment is also evidenced by the fact that there is currently almost no silt and clay present in the bed of the Ventura River despite large amounts of fine sediment being transported by the river. The current bed material composition is given in Section 5.3. The sediment concentrations on the main stem of the Ventura River have been measured at over 20,000 mg/l during flood events and are commonly over 10,000 mg/l (Figure 8.11). These high concentrations are evidence of a large supply of fine sediment in the watershed, even with Matilija Dam in place. In addition, because the Matilija Reservoir is almost full, most of the fine sediment that enters the reservoir from the upstream end passes over the dam. While the release of additional sediment would increase the natural sediment loads, the river has a large capacity to transport this fine sediment all the way to the ocean. Also, because the preferred alternative only releases fine sediment during flood flows, the low flows would not carry additional fine sediment.

The disposal sites would not affect the percolation of water from the bed of the Ventura River into the Upper Ventura Aquifer. As mentioned above, the primarily supply of water to the aquifer is percolation of water from the Ventura River. It is estimated that no significant recharge to the aquifer occurs from rain falling on the floodplain and then percolating into the aquifer. The average rainfall in this area is approximately 20 inches of rain per year, but can be highly variable. Some of the rain that falls onto the disposal site would run off into the river because the infiltration rate of the disposal site is small. The rain the infiltrates into the disposal sites would most likely eventually evaporate. Based upon measurements of pan evaporation in the Santa Clara River Basin the evaporation potential is more than 60 inches per year (United Water Conservation District, 2001), which is much larger than the annual rainfall.

The slurry disposal sites would be composed of primarily silt and clay, and they would not allow rainwater falling on the disposal site to infiltrate into the groundwater. The fine sediment in the disposal sites would act as a seal on top of the aquifer preventing water from entering the aquifer below the disposal areas. Therefore, anything that would be present in the disposal site sediment would not enter the aquifer below. Furthermore, there has already been extensive testing of the reservoir sediment. Several 2 inch cores were extracted from the full depth of the reservoir sediments and no contaminants above background levels were found.

9.6. Delivery of Sediment to the Ocean

Each alternative would supply different amounts of sediment to the ocean. This may affect the habitat at the beach and the littoral transport in that region. In terms of the sediment delivery to the ocean, the one-notch and two-notch alternatives would be assumed the same. A summary of the sediment delivery over the next 50 years is given in Figure 9.14 and Table 9.15. Yearly volumes are given in “Exhibit H. Sediment delivery to ocean”. The sediment delivery was calculated using the historical data and using the measured sediment volumes behind Matilija Dam. The estimates did not rely solely upon the results of GSTARS-1D. This is because the model did not represent the hydraulics and sediment dynamics in the estuary properly; therefore, the amount of sediment being transported to the ocean was not correct.

It is important to realize the channel as acted as a major source of sediment over the last 50 years. Many channel banks have been cut back and the channel elevation has been lowered in many reaches. It is estimated that approximately 16% of the sediment delivered to the ocean has been eroded from the Ventura River Channel (Table 5.15). However, the channel has a limited supply and it would gradually provide less and less sediment if Matilija Dam remains in place and continues to trap sediment.

Another important point is that a significant amount of material may remain in the reservoir area until a large flood occurs. It would take potentially several floods greater than the 20-yr floods to mobilize all the sediment in the reservoir area. Therefore, over the next 50 years, up to half of the material behind the dam may remain there. It is expected that eventually all the sediment stored behind the dam would be eroded, but the timing for this is somewhat uncertain. Because a 100-yr flood may cause significant hillslope erosion and the resulting flood would almost fill the entire valley where Matilija Reservoir lies, it is expected that in less than 100 years most of the sediment would be eroded from behind the dam.

The sediment supply is divided into fines (silts and clays), sands, gravels, and cobbles. Of the estimated volumes in Table 9.15, the most certain are those of the fines and sands. There is ample data at the USGS stream gage at Foster Park on the Ventura River to give accurate measurements of the current loads of fines and sands in the stream. The gravel and cobbles estimates have not been verified by data and are subject to more uncertainty. Because large gravels and cobbles move only during the very largest floods, it is difficult to obtain measurements.

The total amount of additional sediment that would be introduced into the Ventura River system relative to the No Action Alternative is given in Table 9.16. The numbers were derived by first calculating the amount of sediment trapped behind Matilija Reservoir that would be returned to the river system for each alternative. This amount was then added to the amount of sediment that would eventually be trapped behind Matilija Dam under the No Action Alternative.

If it is assumed that the eventual equilibrium condition of the Ventura River is the same regardless of the alternative, then it is possible to compute the total potential for sediment supply to the ocean. The numbers in Table 9.16 represent the maximum additional sediment that would reach the ocean over the long term for each alternative. This assumption is valid if the time scale is at least 100 years and the sediment is of sand size and smaller. However, it is possible that significant amounts of sediment, particularly cobble size and greater, remain in the river system for much longer than that and perhaps remain there several hundred years.

No Action: This alternative leaves the dam in place and continues to trap sediment. An additional 3.3 million yd^3 may be stored behind the dam. It is expected that coarse material would not start to pass over the dam until 2038. After 2038, the degradation in the downstream river channel is expected to slow then eventually reverse to aggradation once coarse sediment would be re-supplied to the downstream reaches. Extrapolating the modeling results, it estimated that the Ventura River would take at least an additional 60 years after coarse sediment starts to spill over the dam to reach the equilibrium supply of sediment to the ocean.

The delivery of sediment for the No Action alternative was calculated using historical sediment data collected at the USGS stream gage on the Ventura at Foster Park (USGS gage #11118500). The sediment rating curves developed at this location are given in Section 5.5. It was assumed that the channel currently provides 16% of the sediment that is delivered to the ocean (see Table 5.15). The channel supply is assumed zero by year 20. After year 20, the channel is assumed to stop contributing sediment. The contribution of the channel is linearly interpolated in time between these points. At year 40, Matilija Creek is assumed to start contributing sediment at equilibrium rates.

Mechanical Removal/Permanent Stabilization (1, 4a): This alternative removes the dam, but does not return the reservoir sediments to the river system. Therefore, the river channel would act as a sink of sediment for a period until equilibrium is obtained. Based on the modeling results, it is estimated that the channel and ocean sediment supply would come to equilibrium in approximately 50 years.

Natural Transport (2a, 3a): This alternative returns most of the coarse sediment to the river system, but not the silts and clays within the reservoir. The removal of silts and clays would be accomplished by mechanical means, possibly by slurry line. In addition, the natural supply of sediment would be restored to the downstream reach. The large and sudden increase in sediment load would cause the downstream channel to aggrade quickly. Within a few years, 50 years of degradation would be reversed by the sudden over-supply of sediment. Because of the aggradation, most of the coarse sediment stored behind the dam would not reach the ocean.

The sediment supply to the ocean would come to equilibrium quickly. Under the alternative 2a and 3a, it is expected to occur within 10 years. The equilibrium sediment supply is listed in Table 9.17.

Natural Transport - No Removal of Fines (2b, 3b): This alternative returns all the stored sediment to the river system. In addition, the natural supply of sediment would be restored to the downstream reach. The large and sudden increase in sediment load would cause the downstream channel to aggrade quickly. Within a few years, 50 years of degradation would be reversed by the sudden over-supply of sediment. Because of the aggradation in the river channel, most of the sediment coarser than gravel stored behind the dam would not reach the ocean. However, most of the sands, silts, and clays would reach the ocean.

The sediment supply to the ocean would come to equilibrium quickly. Under the current hydrological alternative, it is expected to occur within 10 years. The equilibrium sediment supply is listed in Table 9.17.

Temporary Stabilization (4b): The total 50-yr sand delivery to the ocean for Alternative 4b would be expected to be similar to Alternative 2a and 3a. However, because the sediment is temporarily stabilized, the time required for the Temporary Stabilization Alternative to reach equilibrium sediment supply would be greater. The revetment would eventually be removed, but the exact time that it would be removed would be dependent upon the hydrology and the criteria used to indicate when it should be removed. It is estimated that equilibrium sediment supply would occur approximately 10 years after the last removal of the revetment.

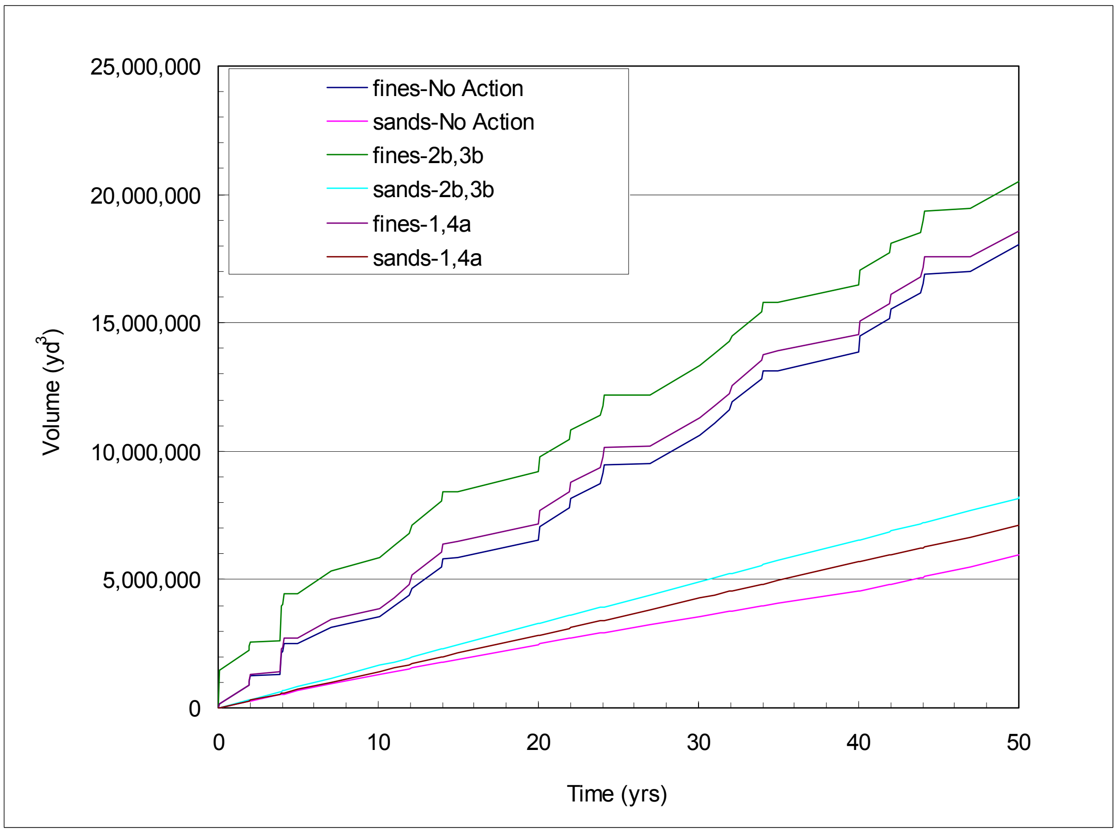


Figure 9.14. Ocean delivery for 50-yr period of simulation.

Table 9.15. Summary of delivery of sediment to the ocean for alternatives. The sediment includes both that trapped in the reservoir and that supplied from the watershed. The number in parenthesis is the percent increase in sediment load for that size fraction relative to the no-action alternative.

	No Action	Alternatives 1, 4a	Alternatives 2a, 3a	Alternatives 2b, 3b	Alternative 4b
Time to reach equilibrium transport to beach (yrs)	~100	~50	~ 10	~ 10	10 - 20
Net 50-yr Fine Transport (yd^3)	18,000,000	18,600,000 (3 %)	19,000,000 (6 %)	21,000,000 (17 %)	19,000,000 (6 %)
Net 50-yr Sand Transport (yd^3)	5,900,000	7,100,000 (20 %)	7,800,000 (32 %)	8,100,000 (37 %)	7,800,000 (32 %)
Net 50-yr Gravel Transport (yd^3)	410,000	490,000 (20 %)	570,000 (32 %)	570,000 (32%)	570,000 (32 %)
Net 50-yr Cobble Transport (yd^3)	23,000	27,000 (20 %)	32,000 (32 %)	32,000 (32%)	32,000 (32 %)

Table 9.16. Total Additional Sediment Supply Potential to the Ocean.

	No Action	**Alternatives 1, 4a**	**Alternatives 2a, 3a**	**Alternatives 2b, 3b**	**Alternative 4b**
Potential Fine Transport (yd3)	0	780,000	1,770,000	3,500,000	1,770,000
Potential Sand Transport (yd3)	0	1,320,000	3,000,000	3,400,000	3,000,000
Potential Gravel Transport (yd3)	0	580,000	1,320,000	1,330,000	1,320,000
Potential Cobble Transport (yd3)	0	310,000	710,000	710,000	710,000

Table 9.17. Total delivery of sediment to the ocean for current condition and equilibrium prediction.

	yd^3/yr of sediment delivered				
type	**fines**	**sand**	**gravel**	**cobbles**	**total**
Current	311,000	136,000	9,400	530	457,000
Equilibrium Estimation	373,000	164,000	11,300	630	548,000

Table 9.18. Fraction within each size class for current condition and equilibrium estimation.

Fraction	**Class**	**Size Range in mm**

0.680	silt-clay	< 0.062
0.090	very-fine sand	.062 - .125
0.094	fine sand	.125 - .25
0.090	sand	.25 - .5
0.019	coarse sand	.5 - 1
0.006	very coarse sand	1 - 2
0.003	very fine gravel	2 - 4
0.000	fine gravel	4 - 8
0.004	gravel	8 - 16
0.009	coarse gravel	16 -32
0.004	very coarse gravel	32 - 64
0.001	small cobble	64 - 128
0.00002	cobble	128 - 256

9.7. Impacts to Channel Hydraulics

Deposition in the channel downstream of the dam would cause changes to the hydraulic characteristics of the channel. The impacts again would be most noticeable in the reaches nearest the dam (reaches 6, 5 and 4). In general, for all the alternatives, the channel would tend towards a pre-dam morphology. However, the rate at which it returns to that pre-dam condition would be significantly different. The following sections discuss the relationships between the width to depth ratio and the flow rate. The width to depth ratio is the total width of the water divided by the average depth of the water at a specific flow rate. It is computed within HEC-RAS for the current condition and for each alternative. The area inundated by the 10-yr flood is also discussed.

The width to depth ratios can be used to evaluate habitat suitability for aquatic life. The area inundated by the 10-yr flood is important in evaluating the riparian habitat.

9.7.1. WIDTH TO DEPTH RATIOS

Because of the limitations of the channel survey, it is difficult to predict accurately the width to depth ratios for low flows with a depth of less than 1 foot. The survey cannot accurately resolve differences less than 1 foot. In addition, the average particle diameter in the river is near 100 mm (0.3 ft) for most of the river with many boulders larger than 1 foot present. Many rocks would be protruding through the low flows creating a more complicated flow. Despite these shortcomings, the trends and relative differences between the simulations are still valid.

Currently the sediment model does not predict that the Stabilization Alternative would return to pre-dam conditions within 50 years, however, the simulations may not be correct on this point and it is more likely that the width to depth ratios would be relatively similar to the values obtained for the Natural Transport Alternative. This is true for all reaches.

Width to Depth Ratio in reach 8 (Upstream of Reservoir Area)

This reach is upstream of any influence of the project and therefore no significant differences between the current condition and the alternatives are expected in this reach. The average width to depth ratio is relatively constant at around 70 for flows between 100 cfs and 3000 cfs. As mentioned previously, the width to depth ratios for the low flows (below 100 cfs) may be somewhat inaccurate and it is suggested that a value of 70 be used for the low flows as well (Figure 9.15).

Width to Depth Ratio in reach 7 (Reservoir Impacted Region)

The width to depth ratios in this reach would be governed by the particular alternative. For the No action, the width to depth ratios would remain much as they are now because the dam remains in place. For the Stabilization Alternative (4a) the constructed channel in this area would govern them. For the Mechanical Removal Alternative (1), the river would tend towards the pre-dam conditions (Figure 9.16), where the width to depth ratios varied between 40 and 55 for all flows. This reach is slightly narrower than the upstream reach between of the constriction imposed by the narrow canyon opening at Matilija Dam. The Natural Transport Alternatives and Temporary Stabilization Alternative (2, 3, 4b), would also tend toward the pre-dam conditions.

Width to Depth Ratio in Reach 6b (Matilija Canyon)

The No Action alternative would remain much the same as the current condition, until the coarse sediment starts to spill over the top of the dam. However, no significant changes to the width to depth ratios are expected in the Natural Transport alternative for the first 50 years. The current width to depth ratio is larger for low flows than for high flows. This is consistent with the fact that the channel is current incised and degraded. The riverbanks are steep and the width does not change as rapidly as the depth increases. Therefore, the width to depth ratio decreases for larger flows. It is suspected that similar to the upstream reaches, the pre-dam width to depth ratios would be relatively constant for a range of flows in this reach.

The Natural Transport Alternatives and Temporary Stabilization Alternative (2, 3, 4b) would quickly cause the channel to have relatively constant width to depth ratios. The values would range between 50 for 100 cfs to 70 for 300 cfs. This causes the Natural Transport Alternative to have lower width to depth ratios for low flows and higher width to depth ratios for high flows. This trend is consistent throughout the entire river.

Width to Depth Ratio in Reach 6a (Robles Area)

This reach behaves similarly to reach 6b, however, the width to depth ratios are slightly higher because the rivers exits the canyon in this reach.

Width to Depth Ratio in Reach 5 (Robles to Baldwin Rd)

The river becomes much wider in this reach. Consistent with Reach 6, the Natural Transport Alternatives and Temporary Stabilization Alternative (2, 3, 4b) have lower width to depth ratios for the low flows and higher width to depth ratios for the high flows. The transition flow is approximately the average annual flood (approximately 3000 cfs). Some example cross sections in this reach are shown in Figure 9.23.

Width to Depth Ratio in Reach 4 (Baldwin Rd to San Antonio Creek)

In this reach, the width to depth ratio is relatively similar for the lowest flows. The Natural Transport Alternative shows a lower ratio for flows between 100 and 3000 cfs and a higher ratio for flows above 3000 cfs.

Width to Depth Ratio in Reach 3 (San Antonio Creek to Foster Park)

The width to depth ratio is less sensitive to the alternative downstream of San Antonio Creek. For the flows below 1000 cfs, no significant changes to the width to depth ratios from the current conditions are expected. For flows larger than 1000 cfs, the deposition expected in this reach for all the alternatives would cause the width to depth ratios to increase.

The model predicted that the width to depth ratios for the Natural Transport Alternatives and Temporary Stabilization Alternative (2, 3, 4b) would be slightly lower compared to all the other alternatives, but the reasons for this are not clear at present and therefore it is suggested that the width to depth ratios for Alternatives 1 and 4a be used for Alternatives 2, 3 and 4b.

Width to Depth Ratio in Reach 2 (Foster Park Bridge to Main St Bridge)

The width to depth ratios are relatively insensitive to the alternative and are similar to the current condition.

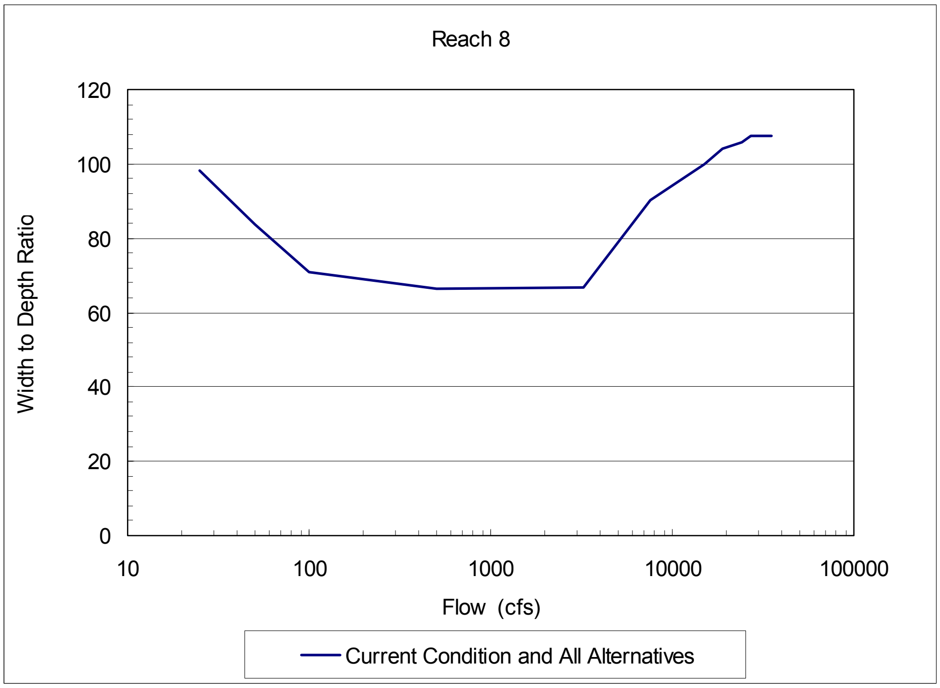


Figure 9.15. Width to Depth Ratios in Reach 8 for all alternatives.

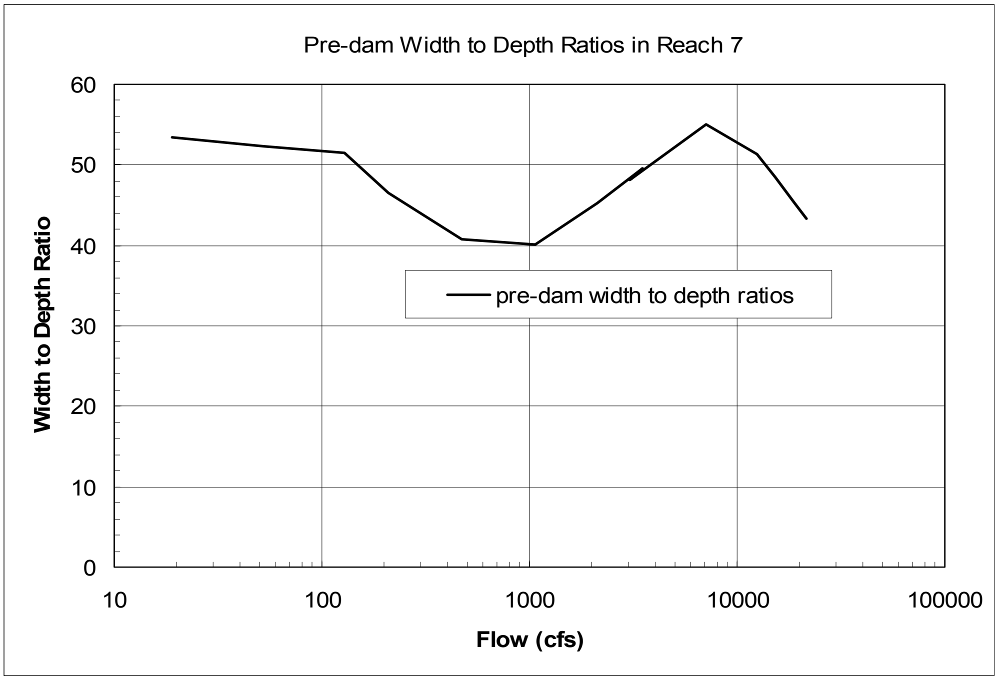


Figure 9.16. Width to Depth Ratios in Reach 7 for pre-dam conditions.

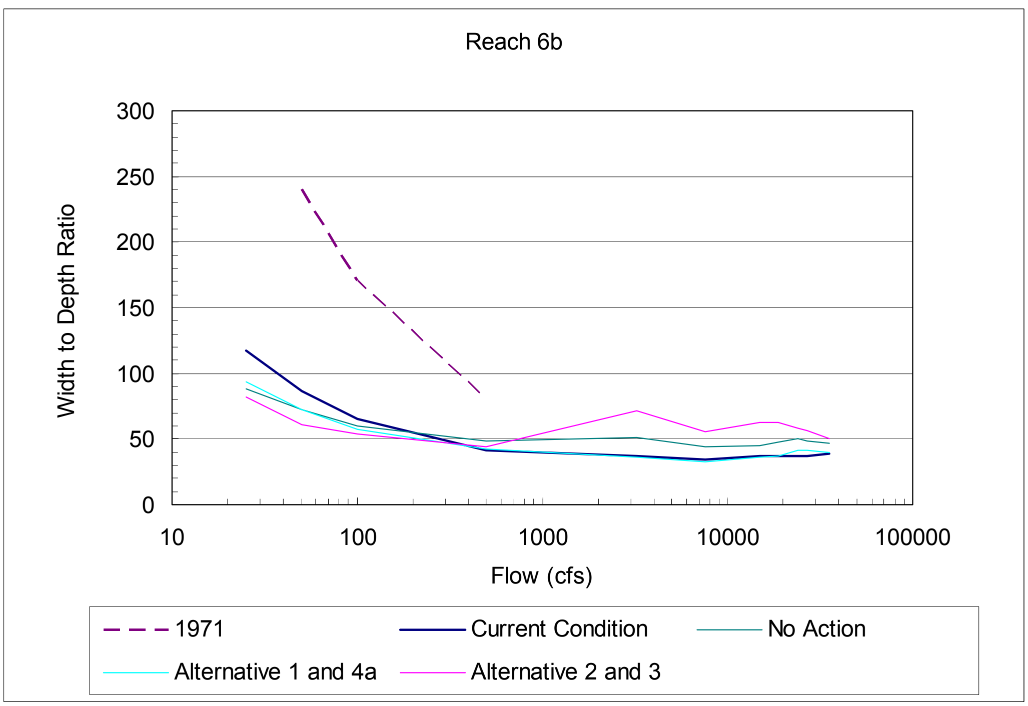


Figure 9.17. Width to Depth Ratios in Reach 6b for all alternatives.

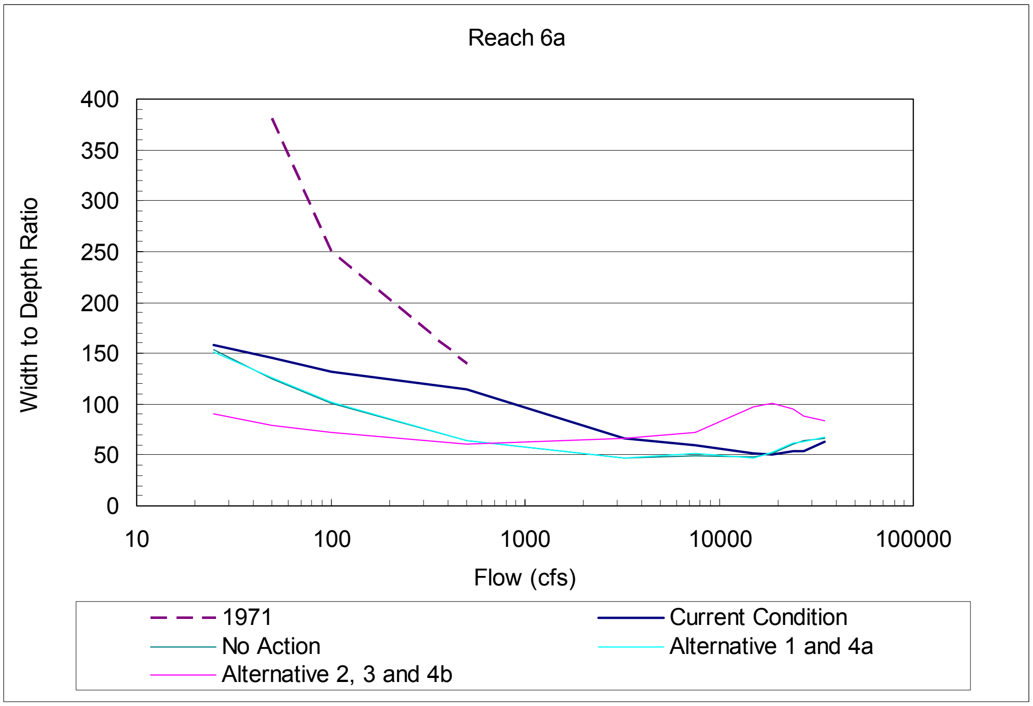


Figure 9.18. Width to Depth Ratios in Reach 6a for all alternatives.

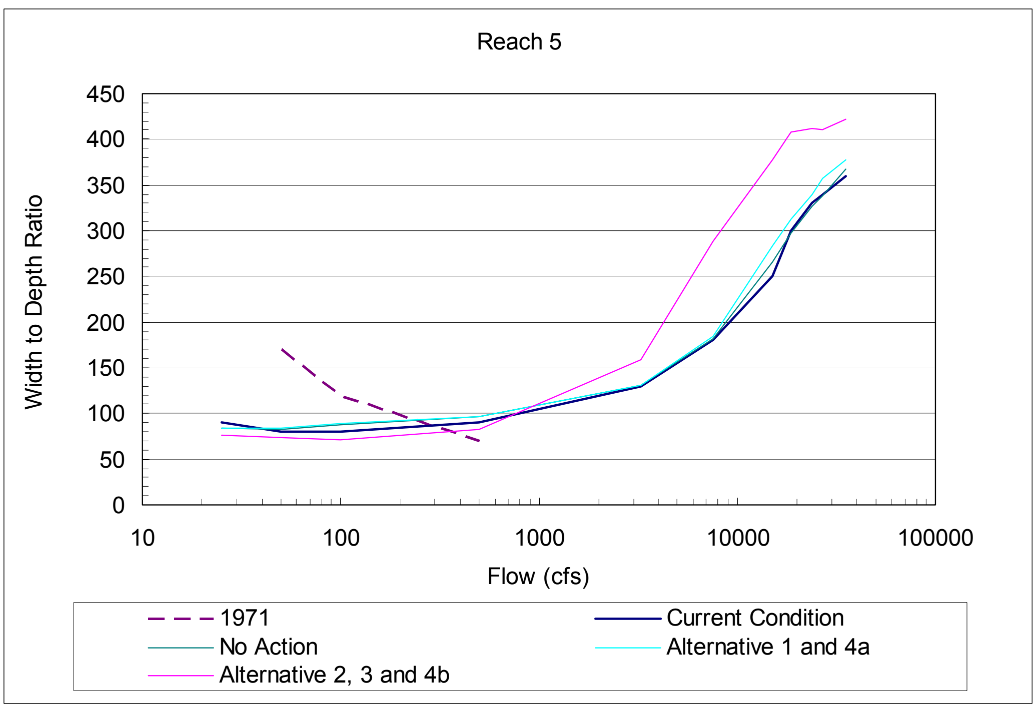


Figure 9.19. Width to Depth Ratios in Reach 5 for all alternatives.

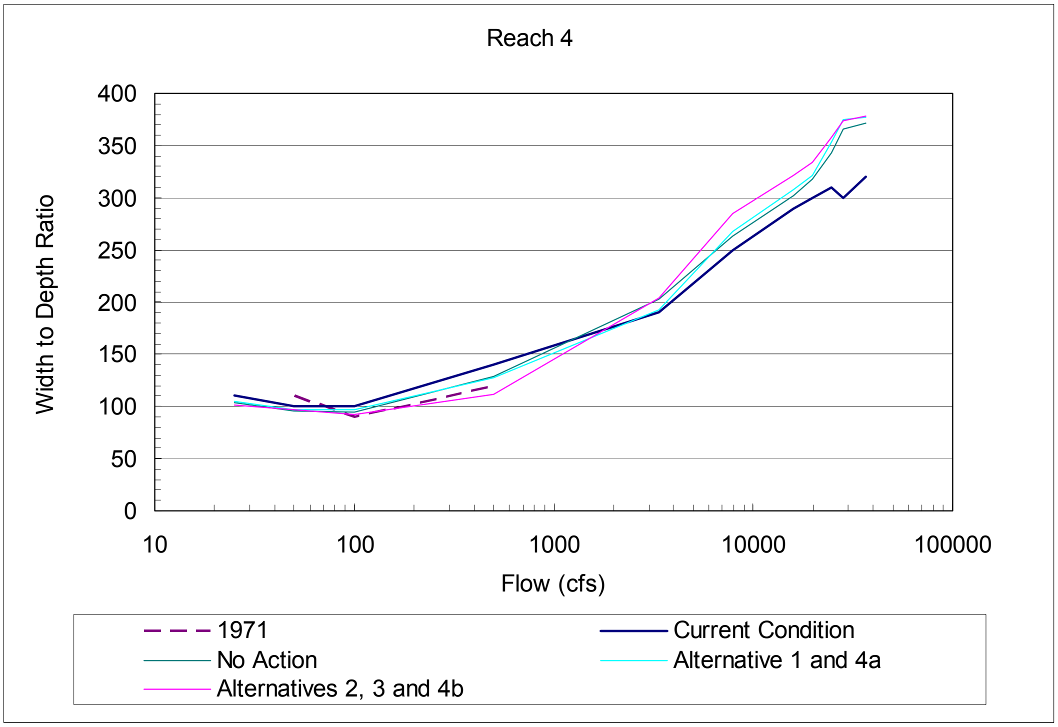


Figure 9.20. Width to Depth Ratios in Reach 4 for all alternatives.

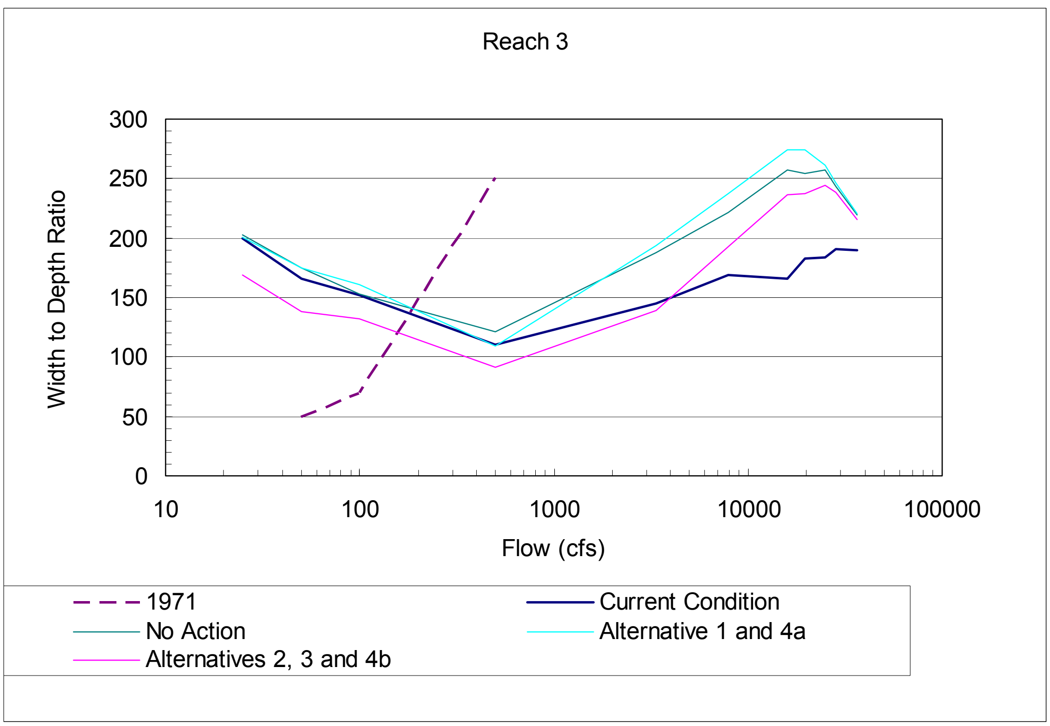


Figure 9.21. Width to Depth Ratios in Reach 3 for all alternatives.

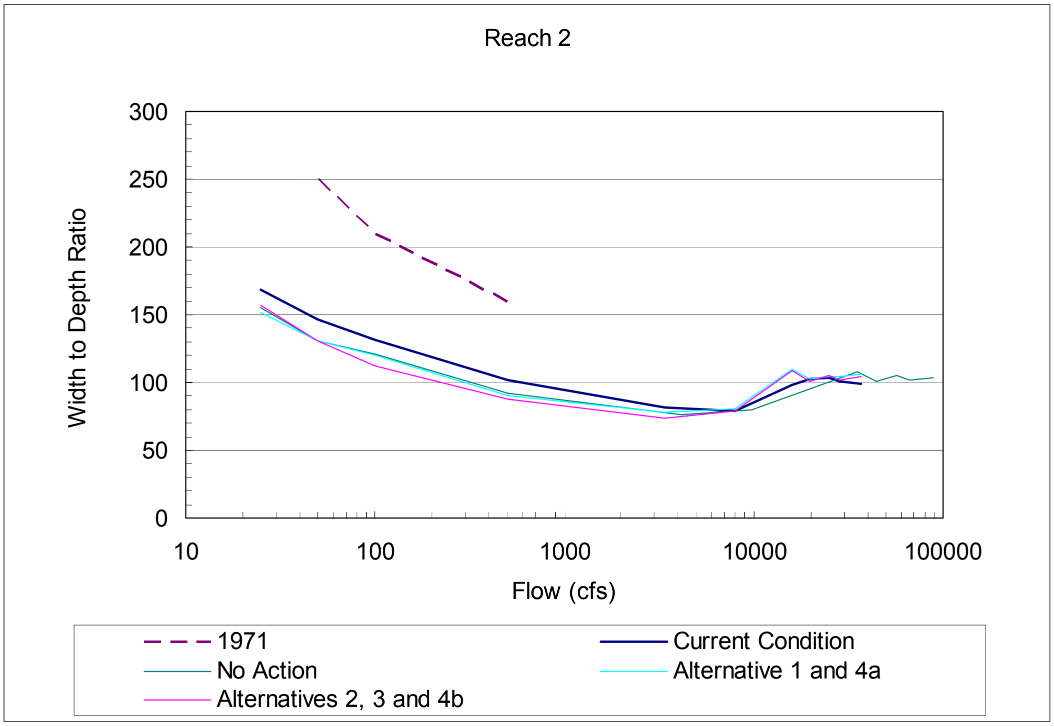
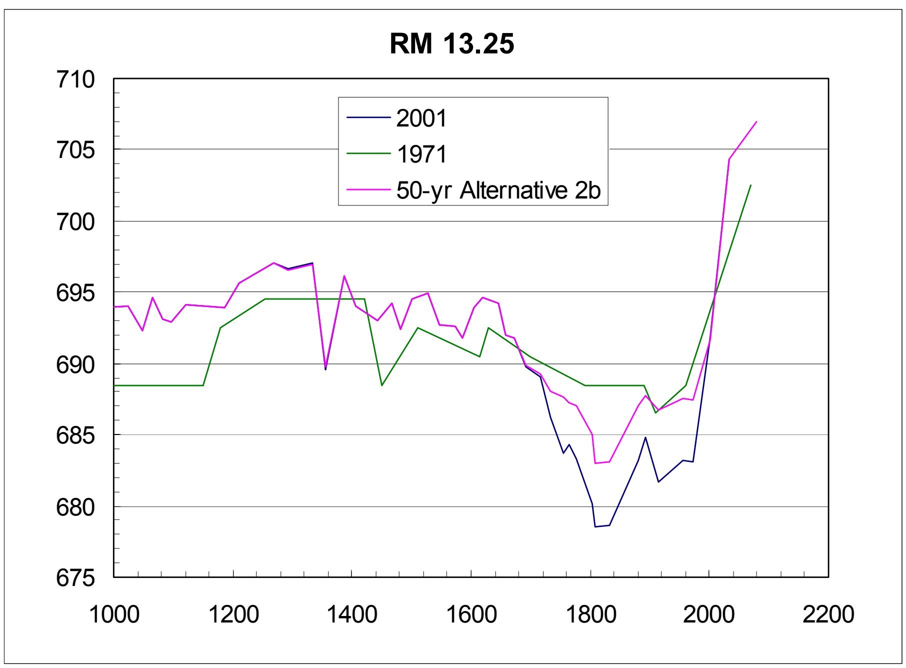


Figure 9.22. Width to Depth Ratios in Reach 2 for all alternatives.



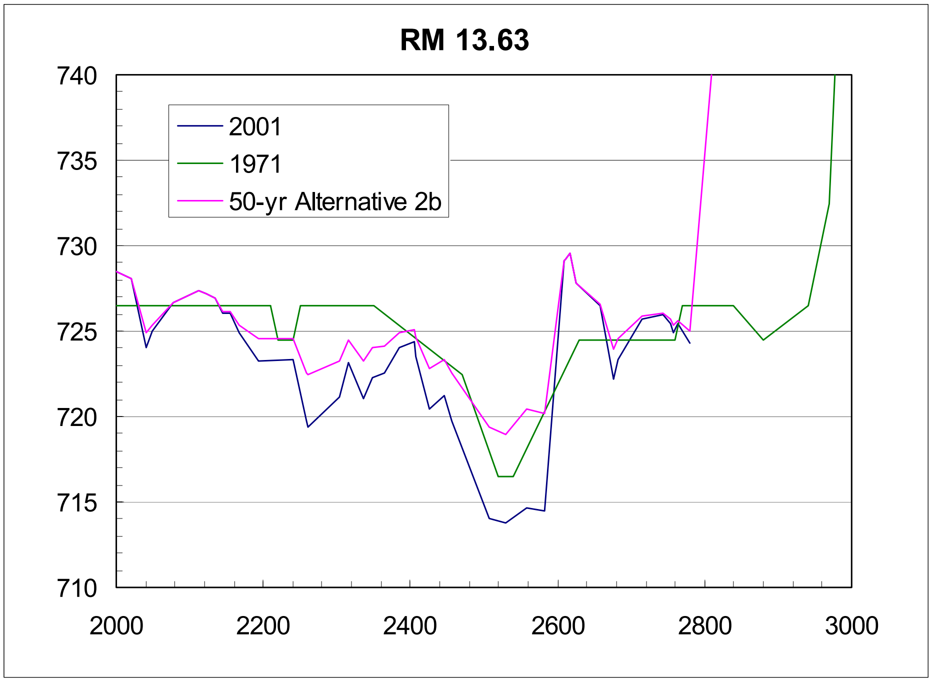


Figure 9.23. Example of changes to the cross sections because of the re-supply of sediment.

9.7.2. CHANNEL AREA INUNDATED BY 10-YR FLOOD

The choice of alternatives would also affect the area inundated by floods. Because of its environmental significance, the 10-yr flood is used to compute the area inundated. No values are reported for the estuary (reach 1), as the beach building and destruction are not modeled explicitly and therefore the area inundated is difficult to predict. In addition, reach 6 is not reported because it is affected by man made disturbances at Robles.

No Action: In the upstream reaches (Reaches 4 and 5) and in the reach 2, there would be a decrease in the area inundated because of the continued degradation in the upstream reaches. The decrease is due to the continued degradation in these reaches. Reach 3 is expected to aggrade slightly and therefore, the area inundated would increase.

Stabilization/Mechanical Removal (1, 4a): There is a slight increase over the No Action alternative for all reaches. The increase in the area inundated is due to the increase in sediment load that causes aggradation in some reaches. The aggradation would tend to cause the flows to overtop the banks more frequently, increasing the area inundated.

Natural Transport Alternatives (2a, 2b, 3a, 3b): In general, the natural transport alternative creates the most area inundated. The largest increase is in reach 5, which is where the largest amounts of deposition occur.

Table 9.19. Total plan area inundated for each alternative for 10-yr flood.

Alternative	Reach	Main Channel Plan Area (acre)	Over bank Area Inundated	
			plan area L (acre)	plan area R (acre)
Current Condition	2	337	38	65
	3	173	4	28
	4	334	34	96
	5	232	57	27
No Action	2	316	42	41
	3	226	36	48
	4	307	35	76
	5	230	57	27
Stabilization/ Mechanical Removal (1,4a)	2	343	46	62
	3	233	40	50
	4	312	36	80
	5	241	65	28
Natural Transport (2, 3)	2	338	45	56
	3	233	40	50
	4	319	36	84
	5	295	100	31

9.8. Fish Passage through Dam Affected Region

Every alternative except the No Action Alternative would eventually restore fish passage in Matilija Creek. The time required to restore fish passage may vary, however. The critical flood required for sediment erosion in the reservoir is assumed the 3-yr flood. This flood was chosen because using GSTARS-1D it was estimated that a flood approximately as large as the 1991 flood would be necessary to move sufficient sediment from the reservoir area (see the results for the 1991 flood in Exhibit G. Model Results for All Simulations).

Once the 3-yr flood was chosen as the critical flood, the cumulative binomial probability distribution (see Bedient and Huber 2002, for example) was used to estimate the probability that a given number of such floods would occur in a certain number of years. The probability of a 3-year flood occurring in any given year is 1/3 or 0.33. The procedure is a relatively simplified approach to estimating fish passage, but is justified considering the large uncertainties associated with sediment dynamics and fish migration.

The number of floods required to restore fish passage was estimated based upon the Alternative. For Alternative 1 it was assumed that a channel sufficient for fish passage would be constructed by year 1. Therefore, fish passage is immediate. The same is true for Alternative 4a and 4b. For Alternative 2a, however, the sediment behind the dam may impede fish passage until sufficient erosion has taken place. It is assumed that two, 3-yr or greater floods would have to occur before fish passage is ensured. Using the cumulative binomial distribution, it is estimated that within 4 to 5 years there is a 50% probability of fish passage. In Alternative 2b, there is an additional 2 million yd^3 of sediment behind the dam and it is estimated that high turbidity would also limit fish passage. Therefore, it is estimated that three, 3-yr floods would have to occur before fish passage would be restored in Matilija Creek. For Alternative 3a, there would be two notchings of the dam. One flood would pass before the second notch would be performed. Then two more floods would be required before fish passage would be restored. The total number of floods required would be therefore three. For Alternative 3b, there would be an additional 2 million yd^3 of sediment and one more flood than for Alternative 3a would be required to ensure fish passage.

Table 9.20. Number of 3-yr Floods Required for Restored Fish Passage. The Number of Years is Measured from the Completion of Dam Removal.

Alternative	Number of 3-yr Floods Required for Fish Passage	Years for 50% Probability of Fish Passage	Years for 90% Probability of Fish Passage
1	0	0	0
2a	2	4 to 5	10 to 11
2b	3	7 to 8	14 to 15
3a	3	7 to 8	14 to 15
3b	4	10 to 11	18 to 19
4a	0	0	0
4b	0	0	0

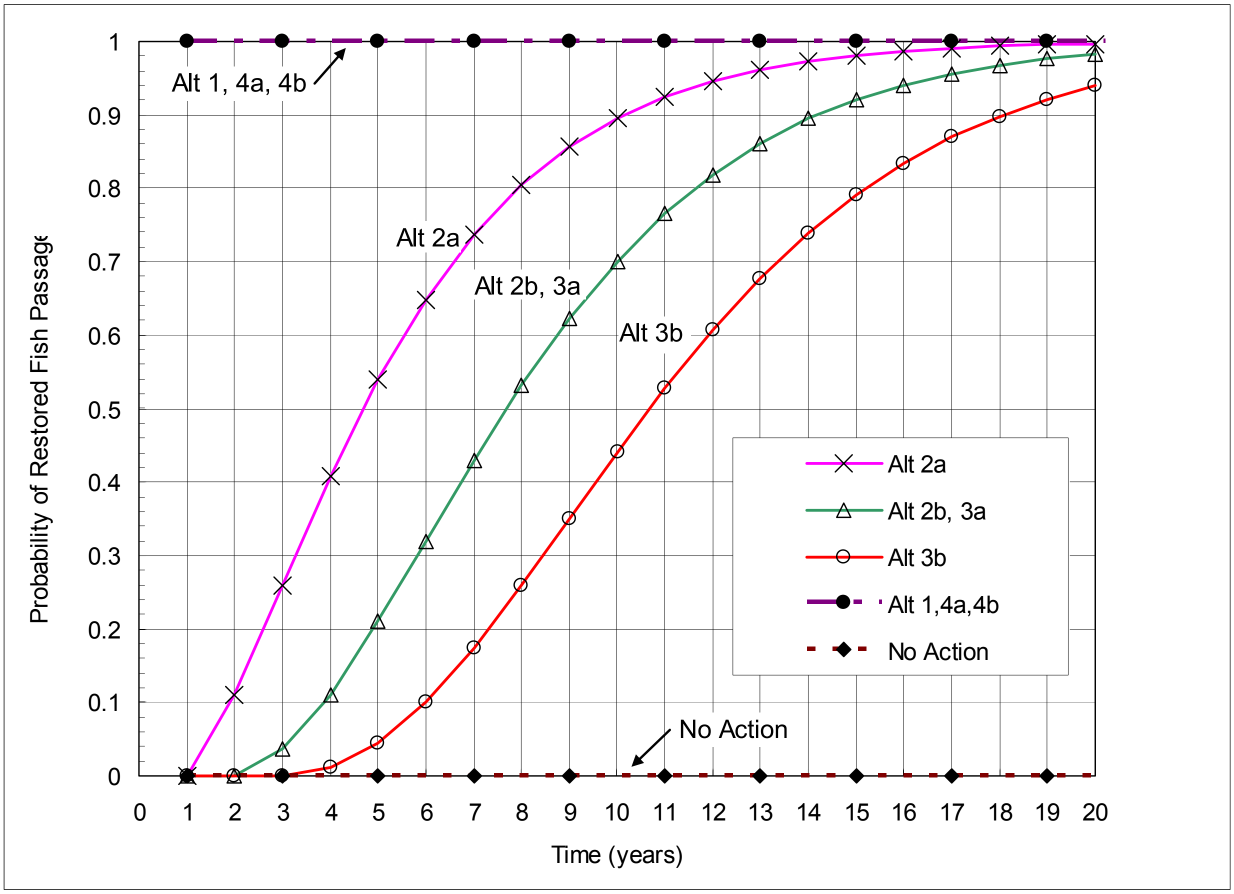


Figure 9.24. Probability that a Given Number of 3-yr Floods have occurred.

10. Mitigation Descriptions

This section describes the mitigation measures proposed for each alternative. There are four main categories of mitigation measures listed here: 1. Flood protection measures, 2. Sediment By-pass at Robles Diversion, 3. Desilting Basin on Robles Canal, and 4. Additional groundwater wells at Foster Park.

In analyzing the alternatives, rough estimates for the flood protection measures were estimated for all the alternatives. After the preferred alternative was chosen, a more rigorous design methodology was performed to design the flood protection measures. Results from each method would be described in the following two sections.

10.1. Initial Estimates of Flood Protection and Levee Construction for all the Alternatives

For the rough estimates performed for all the alternatives, the design of the floodwalls and levees are somewhat conservative and each flood measure would be expected to give protection for floods at least as large as the 100-yr flood. It was assumed that the Alternatives that allow the coarse sediment stored behind Matilija Dam to travel downstream give the same 100-yr flood elevations (i.e. Alternatives 2a, 2b, 3a, 3b, and 4b). The Alternatives that remove or stabilize the coarse sediment (Alternatives 1 and 4a) were assumed to give the same 100-yr flood protection as well.

To compute the flood protection measures for the alternatives that release coarse sediment (Alternatives 2a, 2b, 3a, 3b, and 4b), first the water surface elevation of the 100-yr flood was calculated based on the maximum aggradation predicted during a 50 year simulation of the natural erosion alternative 2b. In general, the maximum amount of aggradation corresponded to the aggradation at the end of the simulation. An additional check was made using the analytical model of Section 7, and if the aggradation predicted by the numerical model was less than the analytical model, the analytical model results were used. A minimum levee height was then calculated by adding approximately 4 feet to the water surface elevations in the upper reaches (i.e. above Baldwin Rd) and 2 feet in the lower reaches (below Baldwin Rd) to account for the uncertainty associated with the computed deposition elevations. Then 6 feet of free board was added to these values. Therefore, there would be up to 10 feet of additional protection built into some of the levees and floodwalls to allow for the large uncertainties associated with predicting sediment transport after dam removal and the resulting flood levels. The levees and floodwalls recommended in this section should be considered as a very conservative levee design and it is likely that further refinement of the alternatives, particularly the alternatives with removal of sediment or multiple notching of the dam, would require substantially lower levees.

To estimate the flood protection for the alternatives that do not allow the stored coarse sediment downstream (Alternatives 1 and 4a), a similar methodology was followed, except that the maximum aggradation from the 50-year simulation of the Stabilization and Mechanical Removal Alternatives (1 and 4a) was used. The same value of 6 feet of freeboard was added to the levee heights in the upper reaches above San Antonio Creek. Below San Antonio Creek, approximately 4 feet of freeboard was added to the 100-yr floodwater surface elevations.

Reach 6b – RM 16.5-15.0

The former Matilija Hot Springs facility would be at risk during high flow events, particularly those resulting from debris/mud flow activity. Due to its close proximity to the dam site and channel, the narrowness of the canyon, and the issues related to the volume and proximity of this much sediment, there would be no conceivable way of protecting this property under Alternatives 2a, 2b, 3a, 3b and 4b during the 100-yr flood. It would be only realistic to purchase and vacate the property; the facility could be set aside until the sediment has been evacuated, then it could be sold back to its owners after equilibrium had been obtained in the reservoir. Under Alternatives 1 and 4a (the complete sediment removal or stabilization scenario), there would be considerable less sedimentation expected in the immediate downstream vicinity of the dam. However, in a sediment stabilization scenario, because of uncertainties in sediment behavior once the dam would be removed, it is still recommended that this facility be purchased and vacated until the channel system reaches an equilibrium condition in regards to sediment transport and bed elevations.

Reach 6a – RM 15.0-14.15

Reach 6a begins at the canyon mouth and extends downstream to immediately upstream of Robles Diversion Dam.

<u>Camino Cielo</u>: Some structures located near the Camino Cielo Bridge would be subject to inundation by either floodwater and/or sediment during high flow events. Due to their close proximity to the channel, the narrowness of the canyon, and the lack of sufficient room for flood conveyance, even under a without-project future condition, the area cannot be protected by reasonable means. The location and constricted nature of the Camino Cielo Bridge require its demolition and restoration of the channel cross section. Removal of the bridge and approaches would improve conveyance through this reach and prevent backwater effects, particularly during high sediment-loaded events.

<u>Meiners Oaks Area</u>: There are numerous structures located along Oso Road and North Rice Road between RM 14.4 and 14.15 (at Robles Diversion). All of these structures are constructed at grade, with no significant first floor elevation above the floodplain. A levee/floodwall approximately 5,023 feet long, extending from approximately RM 14.4 to 13.45, and tying into high ground at either end would protect these properties. The levee/floodwall would be up to 17 feet high above the existing bank.

<u>Robles Diversion:</u> Robles Diversion Dam crosses the channel and is situated within the 100-year floodplain under current conditions. However, if a high-flow bypass is constructed at the Robles Diversion, it is expected that the 100-yr flood elevations would decrease under all alternatives. Therefore, no flood protection measures would be required at the facility.

Reach 5 – RM 14.15 – 11.27

<u>Live Oak Acres:</u> There are at least fifty residences located on the north bank of the river between RM 10.4 and 9.4. They are currently protected by a small levee approximately 3 to 4 feet high at the upstream end and a newer 5-foot levee and floodwall extending down to Santa

Ana Bridge at RM 9.4. Under Alternatives 2a, 2b, 3a, and 3b, a levee would function in the upstream portion of this reach, but due to the close proximity of houses to the channel, only a floodwall could adequately protect the downstream-most portion of this site. A levee/floodwall approximately 6512 feet long and approximately 13 feet high at its maximum would be needed. The height of the levee is large partially because it would be expected that a 100-yr flood would cause pressure or weir flow at Santa Ana Bridge. This dramatically raises the water surface upstream of the bridge and causes the need for a large levee.

Reach 4 – RM 11.27 – 7.93

As mentioned above, Santa Ana Bridge is a severe constriction on the flow. Replacement of the Santa Ana Bridge would be required under all alternatives, with the exception of the "No-Action" Alternative. It presently can only pass the 100-yr discharge if the County continues its maintenance program to excavate sediment from under the bridge. Backwater effects under heavy bedload conditions, which may occur in a 25-year or larger flood event, would cause inundation of many properties on the north side of the channel unless surrounded by an unacceptably high floodwall/levee.

A bridge replacement would reduce the need for high levees for the Natural Erosion Alternatives and the levee cost would be significantly reduced. However, the hydraulic calculations have not yet been performed for the re-designed bridge.

Reach 3 – RM 7.93-5.86

Casitas Springs: There are at least fifty homes in close proximity to the channel at RM 7.85. A levee at the upper end, with a floodwall adjacent to the mobile home park, and a levee extending downstream from this point, would protect this site. A levee/floodwall approximately 5,260 feet long, approximately 15 feet high at its maximum would be needed.

Reach 2 – RM 5.86-0.60

Cañada Larga: Further downstream at Cañada Larga, there are residences, a school, the City of Ventura Water Filtration Plant, and a gasoline refinery located on the south side of the channel. The project would not affect the flood risk in this area.

10.1.1. SUMMARY OF FLOOD CONTROL MITIGATION

The following table summarizes the levees necessary for flood protection for each alternative.

Table 10.1. Flood Protection Measures Necessary for Each Alternative

Location	"No-Action" Alternative	Alternatives 1 and 4a	Alt. 2a, 2b, 3a, 3b, 4b
Matilija Hot Springs	none	Purchase Real Estate	Purchase Real Estate
Camino Cielo Structures/Bridge	none	Purchase Real Estate or Rebuild Structure	Purchase Real Estate or Rebuild Structure
Camino Cielo Hwy 33 Protection 968' Floodwall	none	Floodwall 0.1 to 6.6' Real Estate Purchase	Floodwall 4.1 to 10.6' Real Estate Purchase
Meiners Oaks / Robles Levee/Floodwall/ Levee	none	Levee 0.0' to 1.4'. Floodwall 1.4' to 12.0'. Levee 12.0' to 5.1'. Real Estate Purchase	Levee 0.0' to 6.4' Floodwall 6.4' to 17.0' Levee 17.0' to 10.1' Real Estate Purchase
Live Oaks Levee/Floodwall	none	No levee at upstream end. Floodwall 0.0' to 6.8' No Real Estate Purchase Santa Ana Bridge Replacement	Levee 5.2' to 4.3' Floodwall 4.3' to 12.8' Real Estate Purchase Santa Ana Bridge Replacement
Casitas Levee/Floodwall/ Levee	none	Levee 6.7' to 5.5'. Floodwall 5.5' to 7.4'. Levee 7.4' to 1.2' Real Estate Purchase	Levee 12.7' to 11.5'. Floodwall 11.5' to 13.4'. Levee 13.4' to 7.2' Real Estate Purchase

10.2. Revised Flood Protection for the Preferred Alternative (Alternative 4b)

Alternative 4b has been identified as the Locally Preferred Plan (LPP). Initial incremental analysis has also identified Alternative 4b as the National Economic Development Plan (NER). A risk and uncertainty analysis was conducted to determine recommended levee heights throughout the study reach. The following procedure was followed to determine the risk and uncertainty of the flood impacts.

There were five locations identified in the feasibility study that would be required to be protected. They are the same as identicated in Table 10.1 of the previous section, except that flood protection of Highway 33 near Camino Cielo was found to be unnecessary. The highway is elevated enough at this location to not require protection.

The procedure was based upon the COE reference "EM 1110-2-1619 - Risk-Based Analysis For Flood Damage Reduction," which is available from the Washington D.C. office of the COE.

1. An appropriate index location was chosen in each reach where flood protection was required. The index location was chosen at a cross section where the project indicated deposition that could be considered typical of the reach.

2. The hydrologic uncertainty was computed by determining the discharge-frequency relationship for each index location. The discharge-frequency relationship was taken from Table 2.2. The equivalent years of record was based upon the length of stream gage record used to determine the discharge-frequency relationships.

3. The hydraulic uncertainty was determined. The hydraulic uncertainty is governed by two major factors: the uncertainty in the future deposition or erosion in the Ventura River and the uncertainty in the roughness values used in the HEC-RAS model. The uncertainty in the future deposition caused by the project was determined by computing a low, mean, and high bed geometry.

   a. For the "low" bed geometry the existing condition, 2001, cross sections were used.

   b. The "mean" bed geometry was determined by simulating the 50-yr period with GSTARS-1D starting with the 2001 cross sections. The best estimates of the sediment transport parameters were used. The model parameters used to estimate the mean bed case were the same as for the simulations used to compare alternatives.

   c. The "high" bed geometry was determined by simulating the 50-yr period with GSTARS-1D starting with the 2001 cross sections, as in the mean bed case. However, the erosion rate from the reservoir was tripled. In addition, the inflowing sediment loads from Matilija Creek and North Fork Matilija Creek were doubled. The same sediment transport formula was used as in the "mean" bed case. Another transport formula was also tried, the Meyer-Peter-Muller. However, it was found that the Meyer-Peter-Muller formula did not predict higher deposition for the 50-yr simulations. The Manning's Roughness Coefficient was increased in 0.055 in the main channel as the higher Manning's Roughness generally results in more deposition. The computed deposition was added to the maximum of the 2001 and 1970 bed elevations. As another check on the "high" bed profile, it was checked against the deposition resulting from two 100-yr floods occurring back to back. At all the index locations used in this analysis, the deposition computed from the 50-yr simulations was higher than the deposition predicted from the 100-yr flood.

4. HEC-RAS was then used to compute the 10-, 50-, 100-, and 500-yr frequency events for the following 3 conditions:

1) The "low" bed profile with the low-estimate of n values.

2) The "mean" bed profile with the best-estimate n values.

3) The "high" bed profile with the high-estimate of n values.

5. The stage-discharge relationship at each index location for each flow was computed

6. The standard deviation (SD) of the WSE for each index location was computed by taking the difference between the high value (WS elevation) and the low value (WS elevation) and dividing by four (4).

The results from the above procedure are found in Table 10.2.

At Live Oaks, the deposition values for the "high" bed case were unreasonable high after the 50 yr simulation due to the severe constriction at this location. It is expected that the County would continue to excavate sediment at this location and the model does not account for this. Therefore, to estimate the "high" bed water surface elevation, two feet was added to the elevation predicted in the "mean" bed case. A value of two feet was used because that would be the deposition expected after two 100-yr floods occurring immediately after dam removal as modeled by GSTARS-1D. Therefore, the "high" bed assumption at Live Oaks assumed that the 100-yr flood deposition occurred on top of the "mean" bed deposition. The above procedure gives a 100-yr water surface elevation for the "high" bed that is 5.1 feet higher than existing conditions.

As another check on the values of deposition at Live Oaks, the deposition upstream and downstream of the Santa Ana levee was computed. The "high" bed 100-yr water surface elevations increase approximately 5.6 feet relative to the existing conditions from RM 9.0 to RM 9.3. From RM 10.2 to RM 10.5, the water surface elevations increase approximately 5.3 feet. The final values used for the levee design and those in Table 10.2 are those computed using the procedure of the previous paragraph.

Another exception to the procedure was at the Hot Springs Area. Here the "high" bed estimate gave less deposition than the "mean" bed estimate. This was probably due to the difficultly in simulating the bed mixing in bedrock controlled areas. Therefore, results from the analytical model described in Section 7 were used. The difference between the "best" estimate and upper estimate of deposition at RM 16 from the analytical model was 4 feet. The 4 feet was applied to the “mean” bed geometry to obtain the “high” bed geometry.

Table 10.2. Risk and Uncertainty Summary Data at Index Locations.

Location Description	Hot springs	Camino Cielo	Meiners Oaks	Live Oak	Casitas Springs
HEC-RAS Stationing at Index Location (RM)	16.1932	15.5303	13.7311	9.5644	7.3844
Extents of Reach (RM)	16.1 to 16.3	15.4 to 15.6	13.5 to 14.1	9.4 to 10.3	6.8 to 7.9
Equivalent Record (yrs)	73	68	68	68	68
Levee Heights (ft)	NA	NA	NA	417.2	289.9
Bank Elevation (ft)	961	882	734.5	417.2	289.9
10-yr flow (cfs)	12,500	15,000	15,000	16,000	35,200
20-yr flow (cfs)	15,200	18,800	18,800	19,800	44,400
50-yr flow (cfs)	18,800	24,000	24,000	24,800	56,600
100-yr flow (cfs)	21,600	27,100	27,100	28,300	66,600
500-yr flow (cfs)	27,900	35,200	35,200	36,700	89,000
Existing Conditions					
10-yr wse (ft)	962.4	882.7	730.2	412.7	288.7
20-yr wse (ft)	963.7	883.7	730.5	413.3	289.4
50-yr wse (ft)	964.6	884.9	731.5	414.2	290.1
100-yr wse (ft)	965.3	885.6	731.9	414.9	290.6
500-yr wse (ft)	966.7	887.0	734.3	415.5	290.6
Low Bed Geometry					
10-yr wse (ft)	962.4	882.8	729.8	412.0	288.1
20-yr wse (ft)	963.7	883.8	730.5	412.7	288.7
50-yr wse (ft)	964.6	884.9	731.5	413.6	289.2
100-yr wse (ft)	965.3	885.5	731.9	414.2	289.6
500-yr wse (ft)	966.7	886.8	734.5	415.5	289.6
Mean Bed Geometry					
10-yr wse (ft)	966.5	888.3	733.1	415.7	290.6
20-yr wse (ft)	967.2	889.3	733.8	416.1	291.3
50-yr wse (ft)	968.1	890.5	734.5	417.3	292.1
100-yr wse (ft)	968.7	891.2	734.9	418.0	292.7
500-yr wse (ft)	970.1	892.7	735.7	419.5	292.8
High Bed Geometry					
10-yr wse (ft)	970.5	889.2	738.1	417.7	293.3
20-yr wse (ft)	971.2	890.3	738.8	418.1	293.9
50-yr wse (ft)	972.1	891.6	739.5	419.3	294.7
100-yr wse (ft)	972.7	892.2	739.9	420.0	294.9
500-yr wse (ft)	974.1	893.9	740.7	421.5	295.0
Standard Deviation					
10-yr wse (ft)	2.0	1.6	2.1	1.4	1.3
20-yr wse (ft)	1.9	1.6	2.1	1.4	1.3
50-yr wse (ft)	1.9	1.7	2.0	1.4	1.4
100-yr wse (ft)	1.9	1.7	2.0	1.5	1.3
500-yr wse (ft)	1.9	1.8	1.6	1.5	1.3

[1] The flood frequency at these locations was interpolated between USGS stream gages based on contributing watershed area.

The HEC-FDA Flood Damage Reduction Analysis computer program was used to evaluate recommended levee heights. Additional guidance on developing recommended levee heights was taken from the Corps of Engineers memorandum dated 25 March 1997, Subject: Guidance on Levee Certification for the National Flood Insurance Program". In essence, this memorandum recommends minimum and maximum values of 90 and 95 for the percent chance of non-exceedance for the target one percent chance (100-year) flood.

For this project, the need is to replace, upgrade, or add protection equal to the current existing levels when with project conditions alters the water surface elevations. In addition, the analysis looked at requirements for FEMA 100-Year certification at those mitigation locations.

The results are shown in Table 10.3. The row labeled "Current Level of Protection" contains the approximate level of protection under current conditions and the row labeled "Level of Protection - No Mitigation" contains the level of protection assuming no mitigation measures were constructed. The row below the title "Mitigate Impacts to Current Level of Protection" contains the height requirements for new levees and the additional height requirements for existing levees to maintain their respective level of protection. The following row contains the associated probability that a 100-yr flood would not exceed the height of the levee.

The row below the title "Mitigate Impacts and Provide 100-yr FEMA Level" contains the height requirements for new levees and the height additions to existing (upgrade) levees to have FEMA certification. The following row contains the associated probability that a 100-yr flood would not exceed the height of the levee. The heights are based upon a 95% chance of non-exceedence for the 100-yr flood. The 95% non-exceedence value is used instead of the typical 90% non-exceedence value due to the large uncertainty associated with dam removal.

Note that Live Oak currently has over 100-year protection, so the mitigation levee is greater than the 100-year FEMA requirement levee height. The difference is two feet of levee height. The Camino Cielo site has bank overflow at the 10-year event, however damages to structures/crops do not occur until after the 50-year event for the without project condition.

At Hot Springs and Camino Cielo, preliminary planning and economic screening evaluation indicated that property purchase rather than levee construction would be the most appropriate alternative. Therefore, the alternative of levee construction was dropped from further consideration, and estimate of levee height was prepared for these two locations

Table 10.3. Levee Recommendations Based on Risk and Uncertainty Analysis. Based upon 95 % Non-exceedence Probability for the 100-yr flood.

	Location Description				
	Hot springs	**Camino Cielo**	**Meiners Oaks**	**Live Oak**	**Casitas Springs**
HEC-RAS Stationing	16.1932	15.5303	13.7311	9.5644	7.3844
Current Level of Protection	~100-yr	50-yr	100-yr	> 100-yr	50-yr
Level of Protection - No Mitigation	10-yr	10-yr	50-yr	20 yr	< 10-yr
Extent of Levee Construction	Purchase Property	Purchase Property	New	Upgrade	Upgrade
Mitigate Impacts to Current Level of Protection					
Levee Height (ft)	-	-	5	6	3
Non-Exceedance Probability for 100-yr flood (%)	-	-	95	99.8	60
Mitigate Impacts and Provide 100-yr FEMA Level					
Levee Height (ft)	-	-	5	4	5
Non-Exceedance Probability for 100-yr flood (%)	-	-	95	95	96

10.3. Mitigation at Robles Diversion

The increase in sediment loads downstream of Matilija Dam has the potential to affect adversely the ability of CMWD to divert water at Robles Diversion. The diversion would be impacted through the three mechanisms listed in Section 9.3. These are Deposition in the Robles Basin, Increase in turbidity, and Deposition in Robles Canal and/or Fish Screens.

A possible mitigation for the increase in the sediment loads is to construct a sediment bypass. A sediment-bypass would reduce the risk to diversion by allowing more sediment to travel downstream and by reducing the amount of deposition behind Robles Diversion. Alternative designs for a sediment bypass structure are given in Section 21 titled "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". In that section, four alternative designs are given:

1. A four bay, 120-ft-long radial gate structure on left side of channel
2. A four bay, 120-ft-long radial gate structure on right side of channel
3. A 330-ft long, air bladder operated overshot gated spillway
4. A 120-ft-long, air bladder operated overshot gated spillway

Each structure would reduce the deposition sediment loads by effectively lowering the elevation of the diversion during high flows. This section describes the effect of the sediment bypass on the deposition behind Robles Diversion and quantifies the delivery of sediment into Robles Canal for each alternative.

10.3.1. Deposition at Robles with a Sediment Bypass

The sediment by-pass would decrease the amount of deposition behind Robles Diversion. The current sluice gates have a capacity of 7200 ft^3/s. Flows above this value would pass over the top of the diversion dam, which is at an elevation of approximately 767.5 feet. The opening to the Robles canal is at an elevation of 762.5 feet. To maintain a 500 ft^3/s diversion in Robles canal it is estimated that a minimum depth of 5 feet above the canal invert is necessary.

When flows overtop the diversion dam, large amounts of sediment would deposit behind Robles Diversion and the flow would potentially damage the diversion dam. For example, the 1969 flood destroyed the diversion dam and it had to be rebuilt. A sediment bypass structure would reduce the sediment deposition and prevent the diversion from being destroyed.

Several cases were simulated using GSTARS-1D to determine the effectiveness of the proposed sediment bypass structure. The first was to test its effectiveness if all the sluice gates of the sediment bypass were open. Therefore, no pool was maintained behind Robles Diversion Dam. Results of deposition resulting from the 1991 flood are presented in Table 10.4 for all alternatives. The deposition amounts are the cumulative deposition volumes calculated from the dam face to 700 feet upstream of the dam. For alternative 3a and 3b, it was assumed that the second notching occurs after the first flood. For each alternative, the deposition amounts were computed assuming that the sediment bypass structure is open throughout the entire flood. However, for the No-Action Alternative it is assumed that Robles Diversion is still in place at its current elevation. Therefore, the deposition amounts for the No-Action Alternative would be larger than the other alternatives. Based on the simulation results, when all the gates of the sediment by-pass structure are open, only minimal amounts of deposition would occur for all alternatives.

Further cases were run using the 1998 flood as input and assuming that diversions occur during the entire flood. Therefore, the pool elevation is maintained above the canal invert behind Robles Diversion so that diversions up to 500 cfs may occur. This is done within GSTARS-1D by setting an internal boundary condition at the cross section at the diversion dam. Each alternative was simulated with and without a sediment bypass structure. The total deposition results of the simulations **without** a sediment bypass structure are given in Table 10.5 and the results of the simulations **with** a sediment bypass structure are given in Table 10.6. The difference between the current condition and each alternative with a sediment bypass is given in Table 10.7.

<u>Alternative 1/4a</u>

This alternative is near equilibrium conditions almost immediately. The expected deposition for the 1998 flood without a high-flow bypass is 83,000 yd^3, or an increase of 21,000 yd^3 over the current condition. With a high-flow bypass, the expected deposition is 67,000 yd^3, or an increase of only 5,000 yd^3 over the current condition. Also important is the timing of the deposition. Because the peak flow is allowed to pass through the sediment bypass and does not pass over the top of the dam, deposition does not occur as rapidly during the peak flow. Instead, the deposition occurs more gradually as compared to the current condition. In fact, for Alternative 4a, the time at which 40,000 yd^3 is exceeded is delayed by approximately 24 hours as compared to the current condition (Figure 10.1). The value of 40,000 yd^3 is used because when the deposition exceeds this value problems to the operations at Robles start to appear. Delaying deposition has the benefit of increasing the time available to respond to diversion problems. In the case of the 1998 flood, deposition does not exceed 40,000 yd^3 until the flood has receded. It would be easier to deal with deposition problems during lower flows than high flows.

<u>Alternative 2a</u>

This alternative would allow sediment stored behind Matilija Dam to erode and therefore the deposition amounts at Robles would be initially higher than under Alternative 4a or the current condition. The time at which deposition exceeds 40,000 yd^3, however, is similar to the current condition (Figure 10.2). In addition, after several floods have passed through the reservoir area, the deposition amounts at Robles Dam would approach equilibrium conditions.

<u>Alternative 2b</u>

The deposition amounts at Robles Diversion would be similar to Alternative 2a (Figure 10.3).

<u>Alternative 3a</u>

The deposition amounts for the initial flood (before the second notching) would be similar to that of the current condition. This is because most of the coarse material would not make it over the dam because the fine material in the reservoir was removed and created a reservoir to trap sediment. However, the deposition amounts for the flood following the final notching would be similar to Alternative 2a.

Alternative 3b

The deposition amounts for the initial flood (before the second notching) would be greater than that of Alternative 3a because the sediment in the reservoir was not removed and sediment is able to pass over the top of the dam.

Alternative 4b

This alternative would allow sediment stored behind Matilija Dam to erode and therefore the deposition amounts in Robles would be initially higher than under Alternative 4a. The time at which deposition exceeds 40,000 yd^3, however, it still delayed relative to the current condition (Figure 10.4). In addition, after several floods have passed through the reservoir area, the deposition amounts at Robles Dam would approach equilibrium conditions.

Table 10.4. Deposition at Robles for Each Alternative. Each Alternative is with High Flow By-pass except for No Action Alternative. Sediment Bypass **Completely Open**. Results are for Consecutive 1991 floods.

	1991 Flood (yd^3)	Two 1991 floods (yd^3)	Three 1991 floods (ac-ft)
No Action	17,000	27,500	35,000
Alternative 1	0	0	0
Alternative 2a	10,000	10,000	10,000
Alternative 2b	10,000	11,000	11,000
Alternative 3a	0	1,000	1,000
Alternative 3b	6,000	8,000	10,000
Alternative 4a	0	0	0
Alternative 4b	500	1,500	2,400

Table 10.5. Deposition at Robles for Each Alternative. Each Alternative Assumes **No** Sediment Bypass Structure and Assumes Diversions Occur Throughout the Flood. Results are for Consecutive 1998 Floods.

	1998 Flood (yd^3)	Two 1998 floods (yd^3)	Three 1998 floods (ac-ft)
No Action	62,000	75,000	79,000
Alternative 1	83,000	87,000	87,000
Alternative 2a	90,000	86,000	87,000
Alternative 2b	89,000	86,000	87,000
Alternative 3a	75,000	92,000	91,000
Alternative 3b	89,000	86,000	86,000
Alternative 4a	83,000	87,000	87,000
Alternative 4b	90,000	90,000	90,000

Table 10.6. Deposition at Robles for Each Alternative. Each Alternative Assumes **a Sediment Bypass Structure** and Assumes Diversions Continue throughout the Flood. Results are for consecutive 1998 floods.

	1998 Flood (yd^3)	Two 1998 floods (yd^3)	Three 1998 floods (ac-ft)
No Action			
Alternative 1	67,000	71,000	71,000
Alternative 2a	80,000	78,000	79,000
Alternative 2b	82,000	82,000	82,000
Alternative 3a	50,000	74,000	74,000
Alternative 3b	82,000	82,000	82,000
Alternative 4a	67,000	71,000	71,000
Alternative 4b	76,000	76,000	76,000

Table 10.7. Deposition Difference between All Alternatives with Sediment Bypass and Current Condition for one 1998 Flood.

	1998 Flood (yd^3)
No Action	0
Alternative 1	5,000
Alternative 2a	18,000
Alternative 2b	20,000
Alternative 3a	0
Alternative 3b	20,000
Alternative 4a	5,000
Alternative 4b	14,000

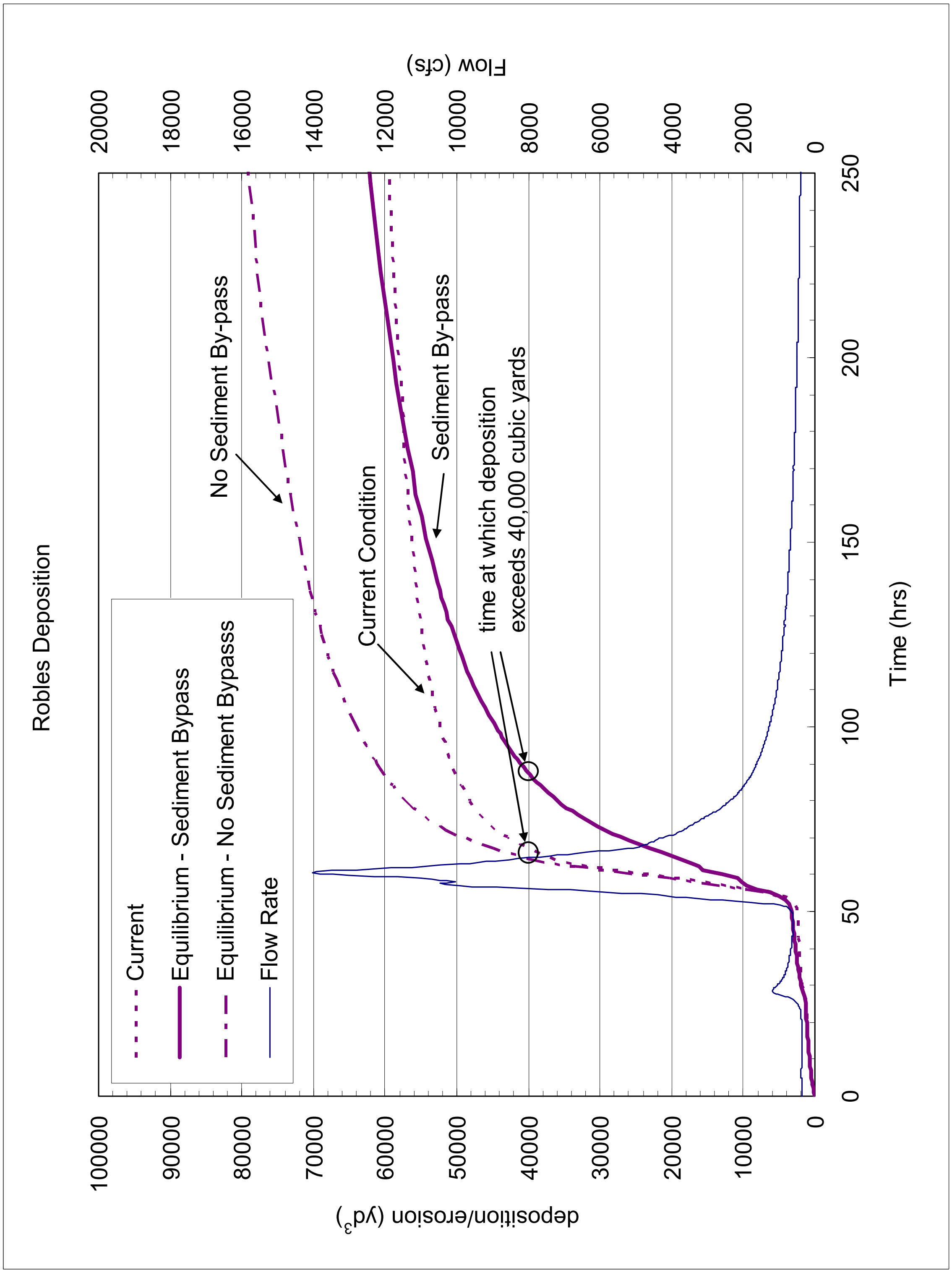


Figure 10.1. Deposition at Robles Diversion under Current Condition and under Alternative 1 and 4a (Equilibrium Condition) with and without Sediment Bypass.

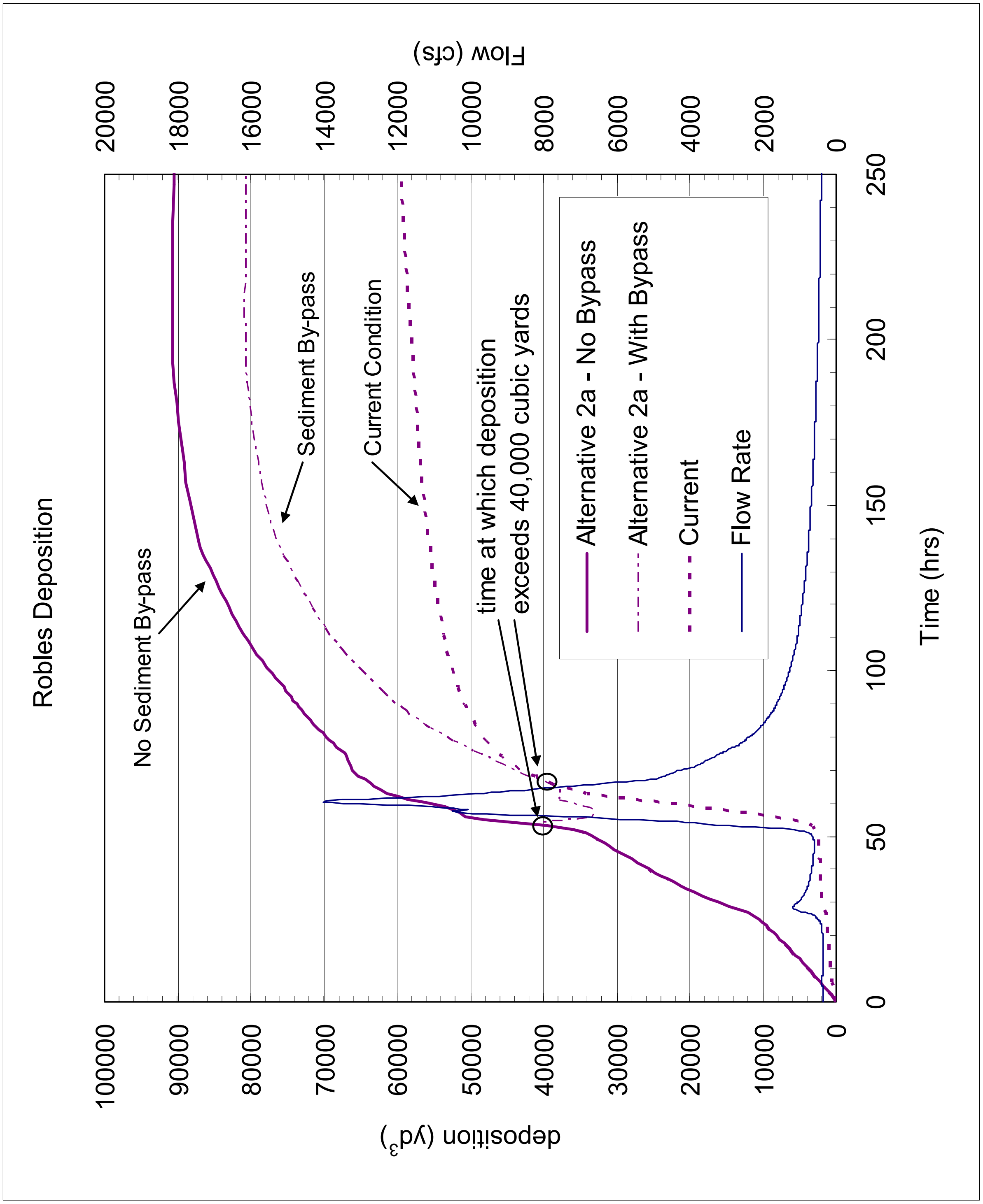


Figure 10.2. Deposition at Robles Diversion under Current Condition and under Alternative 2a with and without Sediment Bypass.

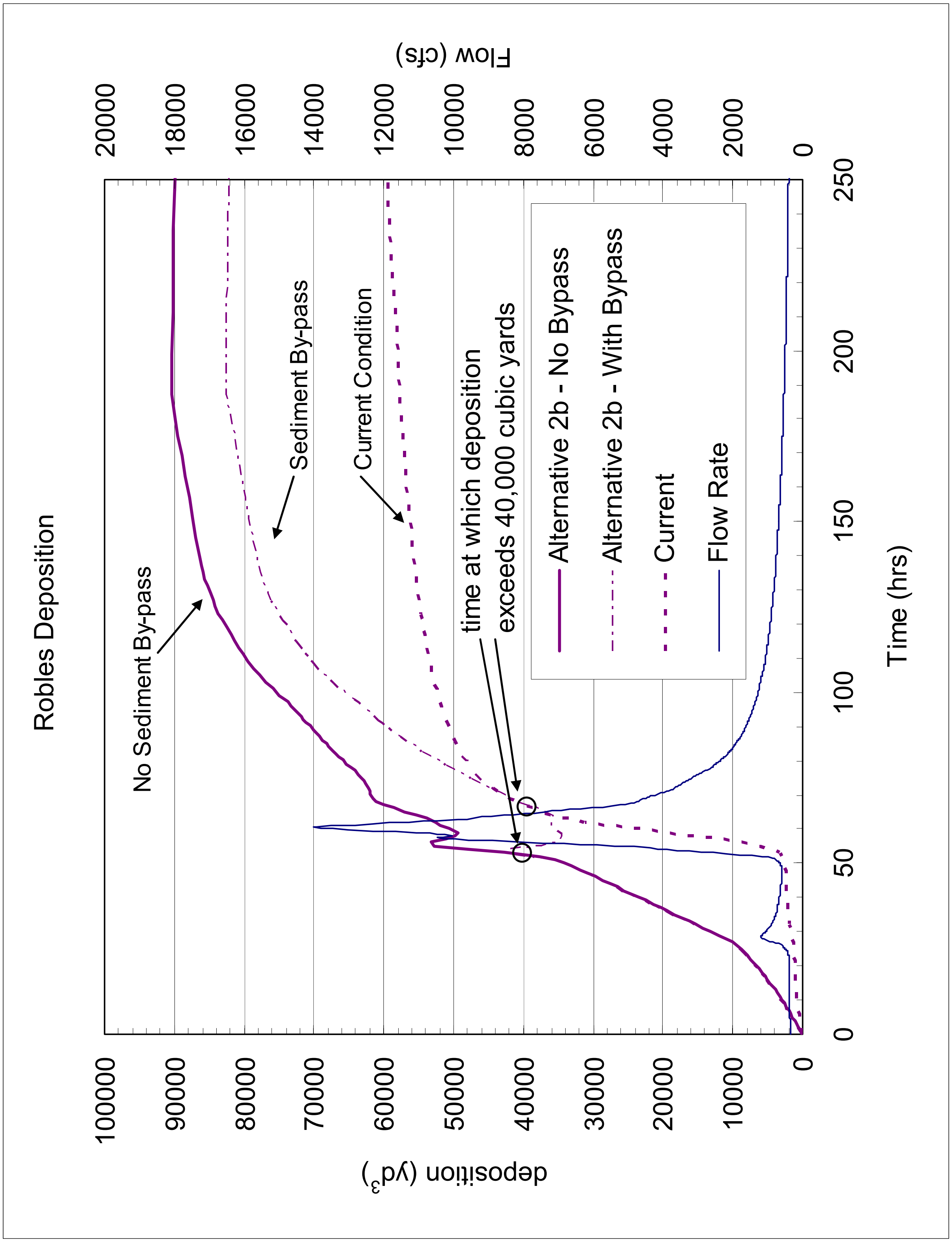


Figure 10.3. Deposition at Robles Diversion under Current Condition and under Alternative 2b with and without Sediment Bypass.

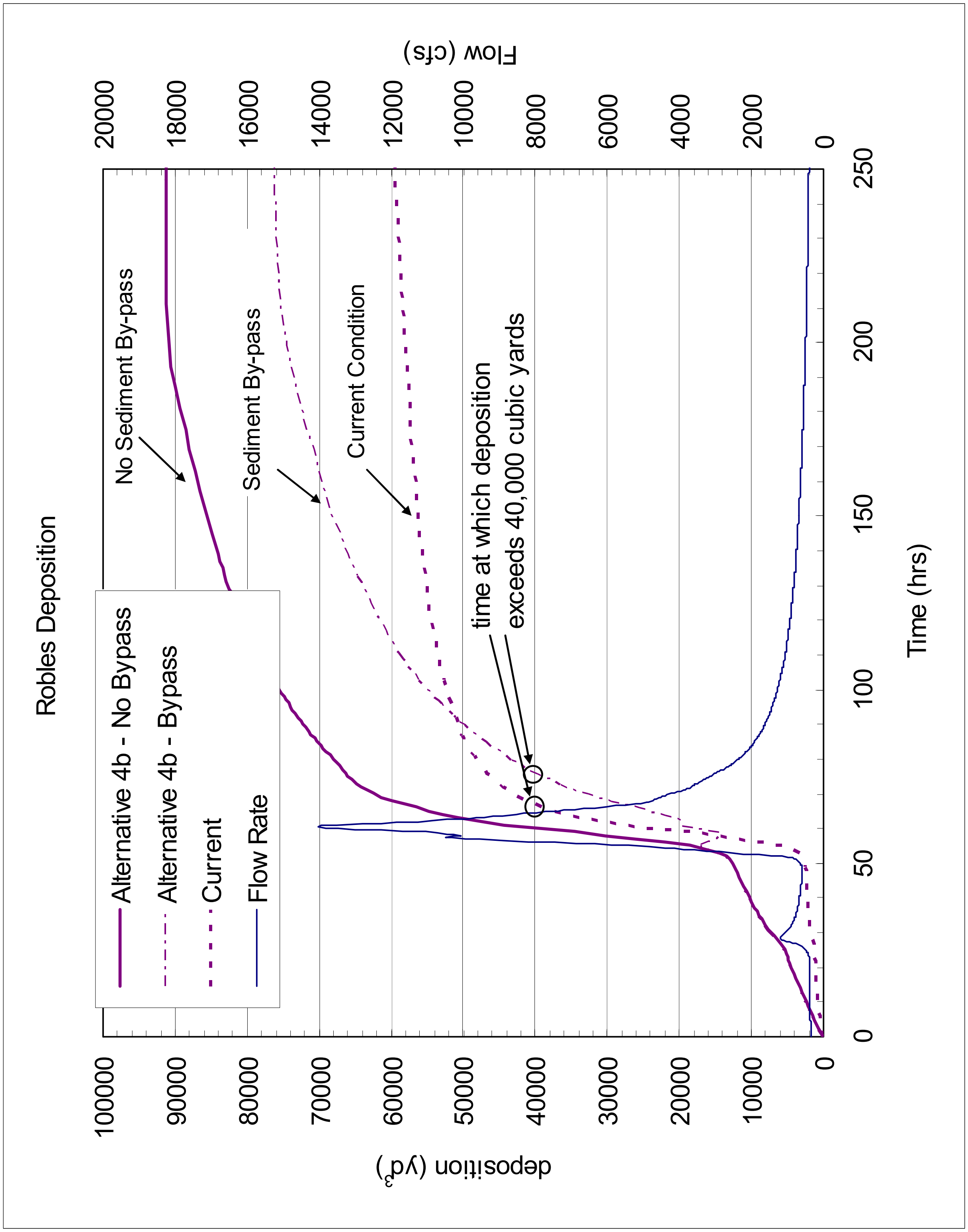


Figure 10.4. Deposition at Robles Diversion under Current Condition and under Alternative 4b with and without Sediment Bypass for 1998 flood.

10.3.2. DELIVERY OF SEDIMENT TO ROBLES CANAL

Sediment from the Ventura River can enter into Robles Canal during flood events. Each alternative would initially deliver different amounts of sediment into the canal. After equilibrium conditions are reached, however, all the alternatives would deliver the same amounts of sediment into the canal.

It was assumed that the silt and clay fractions of sediment were fully mixed in the water. Therefore, the concentration in the river is the concentration that enters the canal. However, for sands, the concentration is much greater near the riverbed than near the water surface. To estimate the sand entering the canal, it was assumed that the concentration distribution is,

$$C = C_a \exp\left(-\frac{w_f(y-a)}{\varepsilon}\right) \qquad \text{Eq 10.1}$$

where w_f is the fall velocity of the sediment particle, ε is the diffusion coefficient, y is the distance from the bed and C_a is the concentration a small distance a from the bed. The canal height is at y_1 relative to the bottom of the channel and then Eq. 9.1 is integrated to find the fraction of the concentration that would enter the canal. The diffusion coefficient is assumed to be $\varepsilon = 0.1u_*h$, where u_* is the friction velocity and h is the depth. Assuming that $a << y_1$ and h, the integration gives:

$$\frac{\tilde{C}}{\overline{C}} = \frac{\exp(-w_f y_1/\varepsilon) - \exp(-w_f h/\varepsilon)}{(1-\exp(-w_f h/\varepsilon))(1-y/h)} = \frac{\exp\left(-\frac{w_f y_1}{0.1u_*h}\right) - \exp\left(-\frac{w_f}{0.1u_*}\right)}{\left(1-\exp\left(-\frac{w_f}{0.1u_*}\right)\right)(1-y/h)} \qquad \text{Eq 10.2}$$

where $\overline{C}$ is the average concentration in the river and $\tilde{C}$ is the concentration in the diversion channel. The fall velocity of the sediment sand size and larger was calculated using Rubey's formula (1933).

As a comparison, the sediment transport by various tributaries is also computed. To compute the sediment transported by Matilija Creek, the data in Section 8.3.1 is used. To compute the delivery of sediment into Lake Casitas by Santa Ana and Coyote Creeks, the ratio of the drainage area of these tributaries to the drainage area of Matilija Creek was used. The drainage area of Santa Ana Creek upstream of Casitas Lake is 9.5 mi^2, or 17% of Matilija Creek drainage area and the drainage area of Coyote Creek upstream of Casitas Lake is 13.4 mi^2, or 24% of Matilija Creek drainage area.

The amount of silt and clay entering the diversion was calculated for all the alternatives with the sediment bypass included. It was assumed that maximum diversions occurred for the duration of the flood. The sediment bypass was assumed to be operating for all alternatives as well, except for the No Action Alternative. In addition, the numbers for the No Action Alternative reflect the first year of the project or current conditions. The results for year 50 of the project would be more near Alternative 1 and 4a, which is near equilibrium conditions. The delivery of sand was calculated only for the equilibrium condition and the current condition. Additional work is required to determine the sand load entering the canal for all the alternatives. The local hydraulics would be important in determining the amount of sand that enters the canal. It may be necessary to modify the bypass operations or design to limit the deposition at the canal entrance and limit the amount of sediment entering the canal.

The delivery of silt and clay under equilibrium conditions is approximately just over 10% higher than under the current condition (Table 10.8). All alternatives would eventually reach the equilibrium condition for fine sediment load. Alternatives 1 and 4a would be immediately at equilibrium conditions for the fine sediment loads. Alternatives 2b and 3b would take at least 2 or 3 floods of average size or larger to reach equilibrium conditions for the fine sediment load. Alternatives 2a and 3a would also take 2 or 3 floods to reach equilibrium conditions, however, their initial impact is much less. The time required to reach equilibrium conditions for Alternative 4b is dependent upon the revetment design. For example, when the final revetment is removed, additional sediment would be exposed to the river and would be available for transport.

The equilibrium sand delivery to the canal is expected to be approximately twice the current values (Table 10.9). The increase is a result of the re-supply of Matilija sediment to the Ventura River. Presently Matilija Reservoir traps most all the sand that enters it. Because some of the sand that enters the canal would deposit in the fish screen area, additional maintenance in the fish screen area is expected over the long term for all alternatives.

Table 10.8. Delivery of Silt and Clay to Robles Canal for Each Alternative. With Sediment By-Pass.

	1991 Flood (ac-ft)	**Two 1991 floods (ac-ft)**	**Three 1991 floods (ac-ft)**
Current Condition	15	30	45
Alternative 1	17	34	51
Alternative 2a	70	100	130
Alternative 2b	800	950	1000
Alternative 3a	55	95	125
Alternative 3b	400	900	1000
Alternative 4a	17	34	51
Alternative 4b	46	85	120
Matilija Creek	48	96	144
Equilibrium Condition	17	34	51
Coyote + Santa Ana Creek	20	39	59

Table 10.9. Delivery of Sand to Robles Canal for Current Condition with No Sediment Bypass and under Equilibrium Conditions with Sediment By-Pass.

	1991 Flood (ac-ft)	Two 1991 floods (ac-ft)	Three 1991 floods (ac-ft)
Current Condition	0.5	1.0	1.5
Equilibrium Condition	1	2	3
Matilija Creek	16	32	58
Coyote + Santa Ana Creek	7	14	21

Current Condition without Sediment Bypass

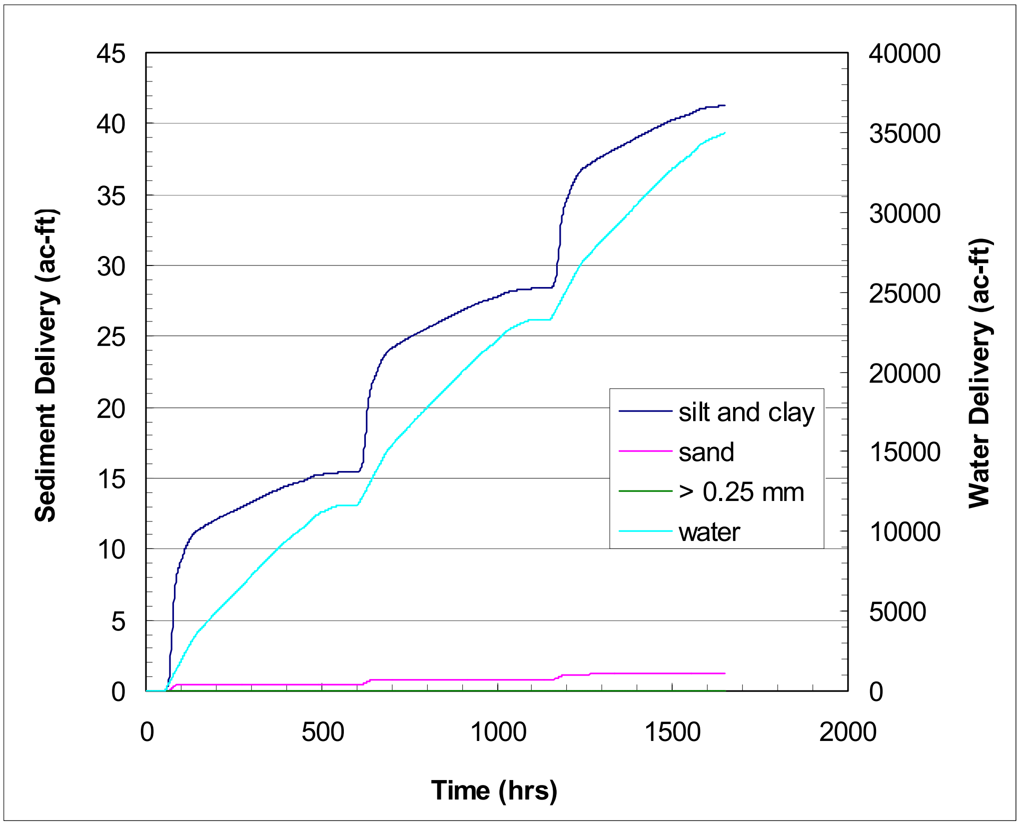


Figure 10.5. Delivery of Sediment into Robles Canal under Current Conditions for 1991 flood. Robles Diversion Dam is at RM 14.15. **No** Sediment bypass.

Equilibrium Condition with Sediment Bypass

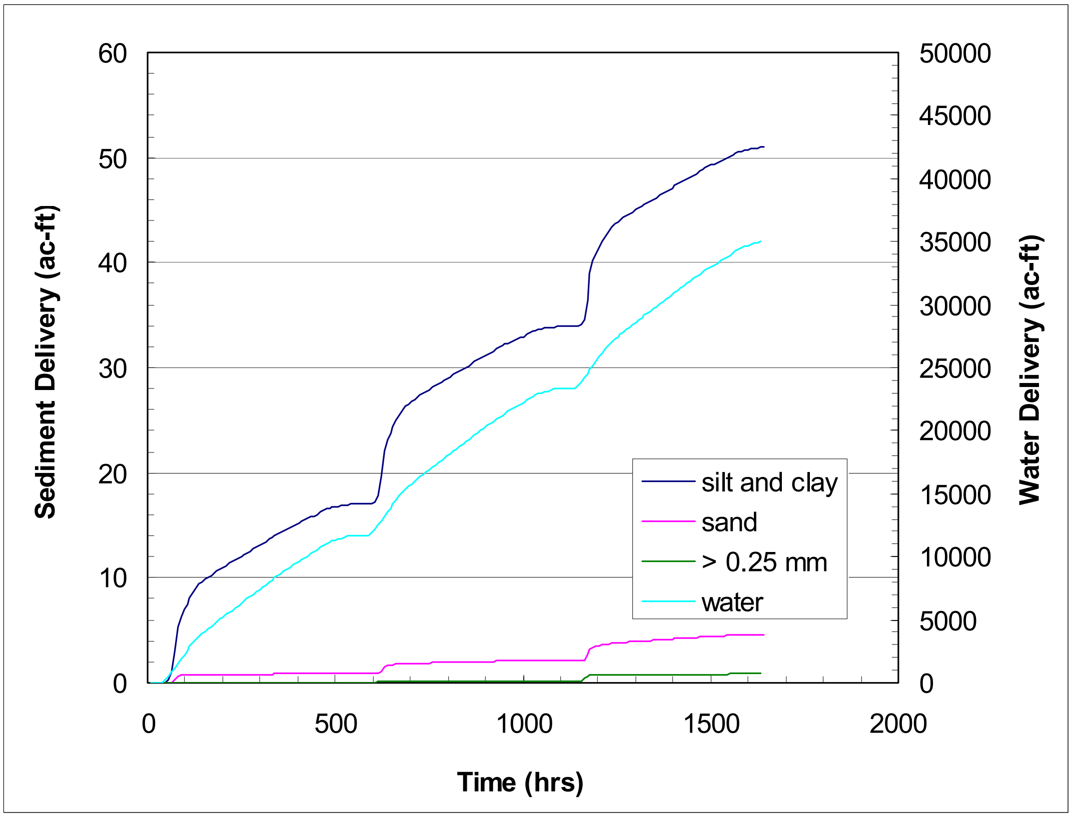


Figure 10.6. Delivery of Sediment into Robles Canal in alternative 2a for 1991 flood. Robles Diversion Dam is at RM 14.15. **With** Sediment bypass.

10.4. Mitigation for Foster Park Diversion

Alternatives 1 and 4a would not affect the fine sediment concentration at Foster Park and therefore there would be no impacts to their diversions for the alternatives. However, for Alternatives 2a, 2b, 3a, 3b and 4b, the additional fine sediment released downstream would impair the ability to capture surface water. It is recommended that subsurface wells be installed to replace the lost water. Increases turbidity loads would not affect the groundwater wells, if those turbidity loads were temporary. In all alternatives, the turbidity levels would return to equilibrium conditions within approximately 10 years. Therefore, increases in turbidity would not impede well function.

11. Summary of Sediment Impacts and Suggested Mitigation Measures

No Action Alternative

The No Action Alternative would cause continued deposition behind Matilija Dam. Over 3 million more cubic yards are expected to deposit behind the dam in the next 50 years. A majority of that sediment would be sand. The reservoir capacity would be expected to be 150 ac-ft in 2010 and less than 50 ac-ft by 2020. However, the relatively small reservoir still provides a benefit to the diversion at Robles. At its current capacity of 500 ac-ft, it is estimated to increase the annual water diversion at Robles Diversion by 590 ac-ft. As the reservoir fills with sediment, this benefit to water diversion would decrease until it is not significant. From now until the reservoir completely fills with sediment the total benefit of Matilija Dam to the diversion at Robles would be 5000 ac-ft.

The river would be expected to remain relatively stable from Matilija Dam downstream to Robles Diversion. From Robles Diversion to Baldwin Road, the river would continue to erode for the next 50 years. On average, there should be approximately 2 feet of erosion. From Baldwin Road to San Antonio Creek, the Ventura River would remain relatively stable. Nevertheless, excavation of sediment at Santa Ana Blvd Bridge would be required to maintain adequate flood capacity. Downstream of San Antonio Creek, 2 feet of deposition would be expected in the Casitas Springs area. The reach between Foster Park and Shell Road Bridge has experienced significant erosion in the past and this would be expected to continue for the next 50 years, with a maximum erosion depth of 3 feet in this reach.

Most of the silt and clay that enters Matilija Reservoir passes over the top of Matilija Dam. However, there is still a small amount of silt and clay that is trapped behind Matilija Dam at the lower flows. It is expected that the average fine sediment concentrations downstream of Matilija Dam would increase by approximately 30% after the reservoir is nearly filled with sediment, which is expected to occur in approximately 10 years.

In approximately 40 years, sand and gravel-sized sediment would start to pass over the dam crest, at which time it is estimated that over 9 million yd^3 of sediment would be stored behind the dam. When sand and gravel-sized sediments begin to pass over the dam, abrasion from these coarse particles may damage the concrete surface of dam crest. Once coarse sediment starts to pass downstream, the reaches immediately below the dam will begin to aggrade. There will also be an increase in the deposition that occurs in Robles Diversion area. It is expected that in approximately 100 years, the Ventura River would be in approximate equilibrium, meaning that sediment load entering the river system is in approximate balance with the sediment load exiting the system. The approximately 2.2 million yd^3 of sand that is presently trapped behind the dam would not be supplied to the beach and approximately an additional 2 million yd^3 of sand would be trapped behind the dam in the next 40 years.

There are current flood concerns along the Ventura River. Several residences downstream of Robles Diversion may be at risk of flooding during a 100-yr flood. At the Santa Ana Bridge, the riverbed would require excavation after every flood if it is to maintain 100-yr flood capacity. In addition, the levee along the Ventura River at the town of Casitas does not provide protection against the 100-yr flood. Flooding would continue to be a problem unless additional levees are constructed.

Full Dam Removal/Mechanical Sediment Transport: Dispose Fines, Sell Aggregate Alternative (Alternative 1)

The Full Dam Removal/Mechanical Sediment Transport: Dispose Fines, Sell Aggregate Alternative (Alternative 1) would remove all the sediment stored behind Matilija Dam from the river system. There would be a natural re-supply of Matilija Creek Sediment to the downstream reaches. This natural re-supply of sediment would have noticeable impact on reaches located between Matilija Dam and Baldwin Road. However, because it is a canyon area, RM 16.5 to RM 16 of Matilija Creek would remain relatively stable. There would be approximately 2 feet of deposition expected in the reach immediately downstream of Robles Diversion. The river would be expected to remain relatively stable after Baldwin Road until the Casitas Springs area where an additional 2 feet of deposition would be expected over the next 50 years. The reach between Foster Park and Shell Road Bridge has experienced significant erosion in the past and this would be expected to continue for the next 50 years, with a maximum erosion depth of 3 feet in this reach.

Significant levee improvements would be required in several areas to prevent the existing flood risk from increasing. Immediately downstream of the dam, the Matilija Hot Springs Private Resort may need to be evacuated for a period of several years until the river stabilizes in that area. The aggradation there should be relatively minor, but some uncertainty exists as to the final equilibrium elevations.

Proceeding downstream, the bridge at Camino Cielo is a low water crossing that would cause aggradation and may increase the flood risk to those residences. This bridge would have to be modified or these residences may need to be evacuated. Immediately downstream of Robles Diversion, some of the Hawthorn Acres residences are built in the floodplain and a levee would need to be constructed to protect them. The Santa Ana Bridge is a severe constriction on the flow and it is in danger of being overtopped by the 100-yr flood if aggradation occurs at the bridge. Therefore, a bridge replacement would be suggested, where the new bridge would have a higher bridge deck and a wider opening to pass flows. The Casitas Levee is currently undersized and would need to be improved to meet the 100-yr flood protection criteria. An additional 2 feet of deposition would be expected at this site over the next 50 year and therefore, the levee would have to accommodate this as well.

Because of the re-supply of Matilija Creek sediment, the deposition at Robles Diversion may increase by approximately a factor of two if there are no changes to the current diversion facility. This would increase maintenance costs and perhaps increase the risk of missed diversions during high flow events. A sediment bypass structure would be recommended and its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". The Sediment By-pass would allow high flows to pass through the Robles area without being obstructed. Currently, the sluice gates have a capacity of 6,700 ft^3/s. The 10-yr flood in this area is 15,000 ft^3/s and would potentially cause a large amount of deposition behind the Robles Diversion Dam due to the severe backwater caused by the fixed elevation diversion dam. It is estimated that if a sediment bypass is installed, the diversion capability of CMWD should not be adversely affected. In addition, the sediment bypass would reduce the amount of excavation required and deposition amounts should be similar to those presently occurring.

Silt and clay concentrations in the Ventura River would not be significantly different from the No Action Alternative. However, the total sand transported to the ocean over a 50-year period would increase approximately 20% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

The Full Dam Removal/Natural Sediment Transport Slurry "Reservoir Area" Fines Offsite (Alternative 2a)

The Full Dam Removal/Natural Sediment Transport: Slurry "Reservoir Area" Fines Offsite Alternative (Alternative 2a) uses the natural flows to erode the delta and the upstream channel. The delta is composed of approximately 13% gravel, 54% sand, 28% silt and 5% clay and the upstream channel is composed of approximately 39% cobbles, 39% gravel, 16% sand and 6% silt. When flow starts to erode this material, a narrow deep channel would first be created through the material, followed by gradually widening of the channel through the delta deposits. The rate of widening will be dependent upon the flow rate: the larger the flood, the more material removed and the wider the channel through the delta.

Because the fraction of silt and clay is relatively small in the delta sediments, the turbidity impact will be of relatively short duration. After the first flood peak has past, the concentrations of fine material will quickly decrease, however, they will be 2 to 3 times larger than natural conditions. Currently, the fine concentrations fluctuate by a factor of two or more; so the increases, while real, would be within the range of the natural variability. After a flood with a return period greater than 10 years or after a period of 3 years, which ever comes first, the increase in fine sediment concentration would be expected to reduce to 10 % to 50 % greater than background concentrations. Within 10 years and as early as 5 years following dam removal, the fine sediment concentration will be similar to the No Action Alternative.

The rise in turbidity levels may affect the surface diversion potential at Foster Park on the Ventura River because they currently stop surface diversion when the turbidity level is higher than 10 NTU. The fraction of time that 10 NTU is exceeded at the surface intake would be increased significantly until the first flood passes. After the first flood, it is estimated that the concentrations would be increased by a factor of two to ten times and therefore the surface diversion would be shut down more often than presently. After the third flood passes, the concentrations should return to near natural levels. The upper and lower bounds on the volume of missed surface diversions are 7710 and 4680 ac-ft, respectively. It is recommended that the surface diversion at Foster Park be removed and be replaced by subsurface wells. The subsurface wells would not be adversely affected by the increase in turbidity.

Because the dam would be removed in one-notch in this alternative, approximately 3.9 million yd^3 of sediment would be available for transport in this alternative. Some of this material would deposit in the upper reaches of the Ventura River. There is considerable uncertainty regarding the deposition downstream of the dam and therefore the levee and floodwall design would be necessarily conservative.

Large amounts of sediment would deposit in the area impounded by Robles Diversion Dam with the current diversion design. Based on the simulations run using the 1991-2001 hydrology, Alternative 2a would deposit 70,000 yd^3 the first year following dam removal. Under equilibrium conditions, approximately 40,000 yd^3 would be deposited. Deposition in excess of 40,000 yd^3 could effectively shut down the diversion operations at Robles for that first year and therefore a sediment bypass structure would be recommended. The sediment bypass would reduce the deposition at the site and decrease the risk of missed diversions. Its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". The bypass delays the time at which the deposition becomes excessive and allows operators more time to respond to deposition problems.

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

The Full Dam Removal/Natural Sediment Transport (Alternative 2b)

The Full Dam Removal/Natural Sediment Transport: Natural Transport of "Reservoir Area" Fines Alternative (Alternative 2b) removes the dam all at once and allows natural flows to erode all the sediment stored behind Matilija Dam. The initial erosion would take place vertically and cut a deep channel through the reservoir sediments. The concentration of fine sediment downstream of the dam would be exceedingly large, greater than 100,000 mg/l, for a period of days to weeks. After this initial formation of a channel through the reservoir deposits, the flow would begin to cut a deep narrow channel through the delta deposits. When the flow rate increases during a flood, the channel through the delta would become much wider and a significant amount of sands, gravels, and cobbles material would be removed from the delta. The first two to three floods would carry extremely high sediment loads downstream. Concentrations may be more than 10 times greater than natural conditions for a period of several years. The concentration of fine material would decrease after each flood and would be expected to reduce to approximately twice-natural levels after three floods that are equal or greater than an average annual flood.

The deposition impacts in the upper reaches of the Ventura River would be large and the deposition elevations are uncertain. Therefore, large levees and floodwalls would be required to provide adequate flood protection. The deposition at Robles would be expected to be similar to Alternative 2a and similar mitigation measures as mentioned in Alternative 2a would be required.

Because the turbidity impacts would last much longer than in Alternative 2a, additional mitigation measures at Robles Diversion and Foster Park Diversion would be required. At Robles, a settling basin or alternate sources of water may be necessary to reduce the impact of fine material on Casitas Reservoir. The desilting basin would have to be large enough to accommodate the maximum volume of sediment that could enter Robles Canal. Because the fine sediment concentration would be much higher in Alternative 2b than Alternative 2a, it is recommended that the surface diversion at Foster Park be removed and be replaced by subsurface wells. The subsurface wells would not be adversely affected by the increase in turbidity.

Large amounts of sediment would deposit in the area impounded by Robles Diversion Dam with the current diversion design. Based on the simulations run using the 1991-2001 hydrology, Alternative 2b would deposit 80,000 yd^3 the first year following dam removal. Under equilibrium conditions, approximately 40,000 yd^3 would be deposited. Deposition in excess of 40,000 yd^3 could effectively shut down the diversion operations at Robles for that first year and therefore a sediment bypass structure is recommended. The sediment bypass would reduce the deposition at the site and decrease the risk of missed diversions. Its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". The bypass delays the time at which the deposition becomes excessive and allows operators more time to respond to deposition problems.

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

Incremental Dam Removal/Natural Sediment Transport Slurry "Reservoir Area" Fines Offsite (Alternative 3a)

The Incremental Dam Removal/Natural Sediment Transport: Slurry "Reservoir Area" Fines Offsite Alternative (Alternative 3a) removes the dam in two stages. A portion of the dam would be removed then a flood would be allowed to erode the sediment stored behind the dam and then the remainder of the dam would be removed. For this analysis, the elevation of the dam crest after the first notch would be 1030. This alternative has similar impacts to Alternative 2a, but there would be a greater measure of control of the deposition impacts. If, for example, more deposition than expected occurred at a particular location after the first stage of removal, it would be possible to mechanically remove that sediment from the stream channel or raise levees in that area before the second notch is started. Therefore, the flood risk associated with Alternative 3a would be much less than that of Alternative 2a. The levees constructed for this alternative would not have to be as high as for Alternatives 2a or 2b because the sediments would be released more slowly and would cause less downstream aggradation. However, if the region is experiencing severe drought conditions, there may be up to 7 years between floods and therefore up to 7 years may pass before sufficient sediment is eroded to perform the second notch.

The incremental removal alternatives 3a would be expected to deposit approximately 27,000 yd^3 at Robles Diversion the first year. However, the second notch would take place in the second year and cause approximately 70,000 yd^3 of deposition the following year with the current diversion design. Therefore, a sediment bypass structure is recommended at Robles Diversion. The sediment bypass would reduce the deposition at the site and decrease the risk of missed diversions. Its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". The bypass delays the time at which the deposition becomes excessive and allows operators more time to respond to deposition problems.

The turbidity impacts would be similar to Alternative 2a; however, the maximum concentrations would be less, but would occur twice because two notchings would be necessary. Similar to Alternative 2a, it is recommended that the surface diversion at Foster Park be removed and be replaced by subsurface wells. The subsurface wells would not be adversely affected by the increase in turbidity.

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

Incremental Dam Removal//Natural Sediment Transport (Alternative 3b)

The Incremental Dam Removal/Natural Sediment Transport: Natural Transport of "Reservoir Area" Fines Alternative (Alternative 3b) again has similar impacts to Alternative 2b, but the risks of reduced water supply and increased flooding would be less. The levees may not have to be constructed as high because the sediment would be eroded from the reservoir more slowly. The turbidity impacts would be extended over a longer period because new fines would be exposed after each stage of removal. If the region is experiencing severe drought conditions, up to 7 years may pass between the first notch and the second.

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

Full Dam Removal/Permanent Sediment Stabilization on Site (Alternative 4a)

In terms of downstream sediment impacts, Alternative 4a is considered similar to Alternative 1.

Full Dam Removal/Temporary Sediment Stabilization on Site (Alternative 4b)

In the Full Dam Removal/Temporary Sediment Temporary Sediment Stabilization Alternative (Alternative 4b), approximately 2.1 million yd^3 of reservoir fines would be removed and deposited in disposal sites. A channel would be then constructed through the remaining 3.9 million yd^3, which is composed of approximately 1 million yd^3 of silt and clay, 1.8 million yd^3 of sand, and 1 million yd^3 of gravel and cobble. The channel would be then stabilized up to a flood between a 2-yr to 10-yr flood. Only sections in the delta and reservoir area would be stabilized. The first flood would erode the residual sediment that is not stabilized. After this first flood passes through the reservoir area, a portion of the stabilization structure would be removed. The next flood that comes through would mobilize some of the sediment that would be exposed. The downstream impacts would be monitored. Then based on the monitoring, the next stabilization structure section would be removed. As before, a flood would be allowed to pass through the reservoir area and erode the exposed sediment. This process would be continued until all the stabilization structure is removed. Another option would be to use rock as bank protection. The rock could be designed to fail at a particular flow rate so that it is naturally eroded. However, no such design has yet been done.

The rate at which the sediment would erode would be a function of the slope stability of the sediment and the shear stress applied to the banks. The deposition impacts in the downstream river channel associated with this alternative would be initially slightly less severe than Alternative 2a because sediment would be not released as quickly. However, the long-term deposition would be similar to Alternative 2a for all but the reaches nearest the dam.

Large amounts of sediment would deposit in the area impounded by Robles Diversion Dam with the current diversion design. Based on the simulations run using the 1991-2001 hydrology, Alternative 4b would deposit up to 70,000 yd^3 the first year following dam removal. Under equilibrium conditions, approximately 40,000 yd^3 would be deposited. Deposition in excess of 40,000 yd^3 could effectively shut down the diversion operations at Robles for that first year and therefore a sediment bypass structure is recommended. The sediment bypass would reduce the deposition at the site and decrease the risk of missed diversions. Its design is given in "Exhibit I. Appraisal Level Design of High flow/Sediment By-pass". The bypass delays the time at which the deposition becomes excessive and allows operators more time to respond to deposition problems.

The turbidity impacts for Alternative 4b, however, would be slightly different from Alternative 2a. Because there would be multiple removals of stabilization structures, there would be multiple impacts of fine sediment. After each removal, there would be some fine sediment released into the river as the flood flow passes through the area. The fine sediment would be mobilized as the banks are eroded. As the flood recedes, the water elevation would recede from the banks and no longer erode the fine sediment. Therefore, the increases in turbidity would be mostly confined to the flood events and the lows flows would not experience large increases in turbidity. The magnitude of the sediment concentration increases would most likely be about 2 to 4 times greater than natural conditions before the removal of the first revetment. After the first revetment would be removed, the concentrations may temporarily increase between by a factor of 2 to 10 times the current condition. After the final removal of revetment, the turbidity levels should stabilize at equilibrium levels after one or two floods of average size pass through the reservoir area. Foster Park diversion would be affected by the increase in sediment concentration. The upper and lower bounds on the volume of missed surface diversions are 8820 and 4950 ac-ft, respectively. It is recommended that the surface diversion at Foster Park be removed and be replaced by subsurface wells. The subsurface wells would not be adversely affected by the increase in turbidity.

The sediment transport modeling to date shows that the gradual release of this material would not substantially change the composition of the Ventura River Bed. Plots of the d_{16}, d_{50} and d_{50} are given in Exhibit G, Section 19.4.5. The d_{16} is the diameter of which 16% of the sediment in the bed is finer. The release of sediment from behind the dam does cause the bed to become slightly finer, but the bed remains coarse and composed primarily of cobbles and gravel. In addition, the bed would eventually return to current conditions. The d_{16} would be greater than 6 mm for all times after dam removal in all reaches upstream of River Mile 2. In most reaches, the d_{16} would be above 10 mm for all times above River Mile 2. The d_{35} would be above 35 mm for all reaches above River Mile 2 for all times after dam removal. The d_{50} remains above 60 mm for all reaches above River Mile 2 for all times after dam removal. The silts and clays would not deposit onto the riverbed. Therefore, silt and clay would not enter into the groundwater aquifer or affect percolation of water into the aquifer

The total sand transported to the ocean over a 50-year period would increase approximately 32% in comparison to the No Action Alternative, which is similar to the increase under Alternative 2a. The sediment supply may be delayed relative to Alternative 2a, however, because the sediment would be temporarily stabilized in the reservoir area. It would be expected that by year

20, the sand supply of Alternative 4b would be very similar to Alternative 2a. The increased sand supply would provide some benefit to beach widths, but the benefit is difficult to quantify.

Alternative 4b has been identified as the Locally Preferred Plan (LPP). Initial incremental analysis has also identified Alternative 4b as the National Economic Development Plan (NER). The flood protection measures for this alternative were revised based upon a risk and uncertainty analysis. The results are shown in Table 11.1. The row labeled "Current Level of Protection" contains the approximate level of protection under current conditions and the row labeled "Level of Protection – No Mitigation" contains the level of assuming no mitigation measures were constructed. The row titled "Mitigation to Current Level" shows the height requirements of the new levees and the additional height requirements for existing levees to maintain their respective level of protection. The row titled "Levee Height to Mitigate Impacts and Provide 100-yr FEMA Level" shows the height requirements for new levees and the height additions to existing (upgrade) levees to have FEMA certification. This is based upon 95% chance of non-exceedance). The 95% non-exceedance value is used instead of the typical 90% non-exceedance value due to the large uncertainty associated with dam removal.

There are five locations identified: Hot Springs is located at approximate RM 16. Camino Cielo is at RM 15.5, near the Camino Ceilo bridge. Meiner Oaks is at approximately RM 14, just downstream of Robles Diversion. Live Oak is the town just upstream of Santa Ana Blvd. There is a current levee approximately 1 mile long that protects the town of Live Oak. There is another current levee at Casitas Springs from RM 7.8 to RM 6.8.

Note that Live Oak currently has over 100-year protection, so the mitigation levee would be greater than the 100-year FEMA requirement levee height. The difference would be two feet of levee height. The Camino Cielo site has bank overflow at the 10-year event, however damages to structures/crops do not occur until after the 50-year event for the without project condition.

At Hot Springs and Camino Cielo, preliminary planning and economic screening evaluation indicated that property purchase rather than levee construction would be the most appropriate alternative. Therefore, the alternative of levee construction was dropped from further consideration, and estimate of levee height was prepared for these two locations.

Table 11.1. Levee Recommendations Based on Risk and Uncertainty Analysis for Alternative 4b.

Location Description	Hot springs	Camino Cielo	Meiners Oaks	Live Oak	Casitas Springs
HEC-RAS Stationing	16.1932	15.5303	13.7311	9.5644	7.3844
Current Level of Protection	~100-yr	50-yr	100-yr	> 100-yr	50-yr
Level of Protection - No Mitigation	10-yr	10-yr	50-yr	20 yr	< 10-yr
Extent of Levee Construction	Purchase Property	Purchase Property	New	Upgrade	Upgrade
Levee Height to Mitigate Impacts to Current Level of Protection (ft)	-	-	5	6	3
Levee Height to Mitigate Impacts and Provide 100-yr FEMA Level	-	-	5	4	5

Summary Tables

Below are summary tables of the impacts associated with each alternative. Table 11.2 contains average deposition expected in each project reach. Table 11.3 contains impacts at the flow diversion along the Ventura River and sediment delivery to the ocean for each alternative. Table 11.4 contains the impacts associated with each alternative when a sediment bypass structure is built at Robles Diversion and subsurface wells are constructed at Foster Park to replace the surface diversion there.

Table 11.2. Summary Table of Deposition for All Alternatives. Results are at Year 50 of a 50-yr simulation.

Location	Alternative						
	No Action	1, 4a	2a	2b	3a	3b	4b
Reach 2 (ft)	1.5	2.2	3.4	3.6	3.5	3.6	3.6
Reach 3 (ft)	1.9	2.6	4.2	4.5	4.2	4.5	4.2
Reach 4 (ft)	-0.2	0.7	2.0	1.6	2.2	1.6	2.3
Reach 5 (ft)	-1.6	0.6	2.1	1.9	2.0	1.3	2.2
Reach 6a (ft)	-1.9	4.7	5.0	5.7	5.8	4.1	6.4
Reach 6b (ft)	-2.0	0.5	0.6	3.0	1.1	1.1	0.9

Table 11.3. Summary Table of Impacts at Diversions and At Ocean **without** Mitigation Measures.

Impact	*No Action*	*Alternatives 1, 4a*	*Alternatives 2a,3a*
Deposition at Robles Diversion	No Change for 40 years	Twice-current levels.	For the first 2 to 3 floods, the deposition may affect diversions. Stabilize at twice-current levels.
Turbidity Impact at Robles	Stabilize at 30 % increase within 10 years	Increase by average of 30%, but within natural variability.	For the first 2 to 3 floods the concentrations would increase by factor of 2 to 10, then stabilize at 30 % increase.
Turbidity Impact at Foster Park	No significant change	Increase by average of 30%, but within natural variability.	May increase period of missed surface diversion
Ocean Delivery	No Change for approximately 50 years	Increase sand delivery by approximately 20 % over 50 yr period	Increase sand delivery by approximately 32 % over 50 yr period
Impact		*Alternatives 2b,3b*	*Alternative 4b*
Deposition at Robles Diversion		For the first 2 to 3 floods, the deposition may affect diversions. Stabilize at twice-current levels.	Each flood following a removal of revetment may affect diversions. Stabilize at twice-current levels.
Turbidity Impact at Robles		For the first 2 to 3 floods, the concentrations would be at least 10 to 100 times higher than current, and then stabilize at 30 % increase. De-silting Basin would be required to mitigate concentrations.	Each flood following a removal of revetment would increase the turbidity by factor of 2 to 10, and then stabilize at 30 % increase. When revetments are not removed, similar to Alt 1, 4a.
Turbidity Impact at Foster Park		May increase period of missed surface diversion	May increase period of missed surface diversion
Ocean Delivery		Increase sand delivery by approximately 37 % over 50 yr period	Increase sand delivery by approximately 32 % over 50 yr period

Table 11.4. Summary Table of Impacts at Diversions and At Ocean **with** Mitigation Measures*.

Impact	***No Action***	***Alternatives 1, 4a***	***Alternatives 2a,3a***
Deposition at Robles Diversion	No Change for 40 years	Similar to current levels.	For the first 2 to 3 floods, the deposition would be larger than normal, but would not affect diversions. Stabilize at current levels.
Turbidity Impact at Robles	Stabilize at 30 % increase within 10 years	Increase by average of 30%, but within natural variability.	For the first 2 to 3 floods the concentrations would increase by factor of 2 to 10, then stabilize at 30 % increase.
Turbidity Impact at Foster Park	No significant change	Would not affect diversions	Would not affect diversions
Ocean Delivery	No Change for approximately 50 years	Increase sand delivery by approximately 20 % over 50 yr period	Increase sand delivery by approximately 32 % over 50 yr period
Impact		***Alternatives 2b,3b***	***Alternative 4b***
Deposition at Robles Diversion		For the first 2 to 3 floods, the deposition would be larger than normal, but would not affect diversions. Stabilize at current levels.	Each flood following a removal of revetment may increase deposition but would not affect diversions. Stabilize at current levels.
Turbidity Impact at Robles		For the first 2 to 3 floods, the concentrations would be at least 10 to 100 times higher than current, and then stabilize at 30 % increase. De-silting Basin would be required to mitigate concentrations.	Each flood following a removal of revetment would increase the turbidity by factor of 2 to 10, and then stabilize at 30 % increase. When revetments are not removed, similar to Alt 1, 4a.
Turbidity Impact at Foster Park		Would not affect diversions	Would not affect diversions
Ocean Delivery		Increase sand delivery by approximately 37 % over 50 yr period	Increase sand delivery by approximately 32 % over 50 yr period

*Mitigation measures include a sediment bypass structure and subsurface wells to replace surface diversion at Foster Park.

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13. Exhibit A. Hydrology Reports

DATE	PEER REVIEWER(S)	CODE
5/17/02	Robert E. Swain Robert E. Swain	D-8530

D-8530
PRJ-13.00

MAY 17 2002

MEMORANDUM

TO: Team Leader, Matilija Dam Ecosystem Restoration Feasibility Study, Ventura County, CA
Attention: D-8540 (Greimann)

FROM: Kenneth L. Bullard, Hydraulic Engineer
Flood Hydrology Group
Technical Service Center

SUBJECT: Ventura River 2- and 5-year Flood Peaks and Flow Duration Curves for Use with Matilija Dam Ecosystem Restoration Feasibility Study, Ventura, California

The attached study provides the supplemental data requested. This study produced the 2- and 5-year flood peaks using partial duration series analysis for five gauges in the Ventura River basin. The partial duration series analysis for the low return periods is necessary to overcome problems of overstating the return periods for flows of a specified magnitude that becomes a problem if only the annual series is used. The analysis and procedures to accomplish this partial duration series analysis are further explained in the report.

In addition, flow duration curves for ten stream gauge sites in the Ventura River basin with daily flow records are produced in this study.

If you have any questions please call me at 303-445-2539.

Kenneth L. Bullard

Attachment

cc: D-8470, D-8530 (Bullard, Schreiner/File-2) (w/att to each)

WBR: KBULLARD/2539 : 5-16-02 : JH/2536
[Ventura River 2-&5-Yr.mkb.wpd]

Ventura River
California

TECHNICAL SERVICE CENTER
Denver, Colorado

Ventura River 2- and 5-year Flood Peaks and Flow Duration Curves
For Use with
Matilija Dam Ecosystem Restoration Feasibility Study
Ventura County, California

Prepared by
Kenneth L. Bullard
Hydraulic Engineer
Flood Hydrology Group

U.S. Department of the Interior
Bureau of Reclamation

MAY 2002

UNITED STATES DEPARTMENT OF THE INTERIOR

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to tribes.

BUREAU OF RECLAMATION

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Ventura River 2- and 5-year Flood Peaks and Flow Duration Curves
For Use with
Matilija Dam Ecosystem Restoration Feasibility Study
Ventura County, California

Authorization: The Bureau of Reclamation (Reclamation) was contracted by the Los Angles District of the Corps of Engineers (Corps) to perform necessary hydrologic and hydraulic computations related to the sediment disposition problems that may be created by potentially removing Matilija Dam on the Ventura River. Part of the required studies is to update existing flood plain maps using new peak flow values based on an additional 30 years of data since the last study in 1970. This report supplies 2-year and 5-year peak flow values to supplement the February 2002 report USBR Flood Frequency Study for the Ventura River. In addition, flow duration curves for ten gauge records in the Ventura River basin are also presented.

Partial Duration Series for 2- and 5-year Flood Peaks: In most streams several flood of events less than 5-years in annual series return period magnitude occur in most years. These are the event magnitudes that are in the range of flows required for this study. For estimates of peak flow frequency for return periods less than 5-years the common recommended procedure is to start with a partial duration series of data. A partial duration series for peak flow data will list all peak flows for the stream gauge that are above some selected base flow. Typically two or three values per year, sometimes many more values per year are recorded. In practice the partial duration series (PDS) of peak flows will always include at least one value per year even if that value falls below the predetermined base flow for selection of other peaks in other years. An annual series of peak flows will only contain one value, the largest peak flow in any year, for all of the years of record. The annual series is most typically used in peak flow analysis for larger return periods, 10-years and above. Most stream gauge sites in the United States report peak flow data for both annual and partial duration series.

The standard procedure for analyzing annual series of peak flows is provided in Bulletin 17B (B17B) (Geological Survey, 1981). This procedure can produce numerical values for peak flows with calculated return periods less than 5-years. The probability model used is usually adequate as an approximation for the risks due to large-magnitude low-frequency floods that are unlikely to occur at all in any given year and are extremely unlikely to occur more than once in any given year. The annual-flood analysis does not furnish the necessary information about the likelihood of multiple flood occurrences per year. Generally speaking, for the low return periods, less than 5-years, the annual-flood model understates the total risk or total effect for the year because of the undercounting of the number of minor floods in the year. Conversely, it overstates the recurrence interval of minor floods of a given magnitude, again because of the failure to recognize the occurrence of multiple events per year (Kirby, 2000).

The partial duration series frequency analysis is briefly described in The Handbook of Hydrology (Maidment, 1992). The analysis procedure is essentially the same as for the annual series data except that an additional step is required to convert the partial duration series return periods back to an equivalent annual series return period. Mathematical relations exist that help with this transformation and are described later in this report.

Basic Data Availability: Five stream gauges in the Ventura River basin have partial duration series values. Only one of the stations is on the main stem of the Ventura River. Those stations and the length of record are given in the preceding table. Other gauges on the main stem of the Ventura River at Meiners Oaks and below Matilija Dam were considered but none of them had partial duration series data.

Table 1
Ventura River Basin Stream Gauges for Use with Partial Duration Series Analysis

Location	USGS Gauge Station No	Drainage Area (sq. mi.)	Gauge Datum (ft)	Partial Duration Series Record Years	Number of PDS Peaks
Coyote Creek nr Oak View	11117600	13.2	577.37	1952-1997	165
Coyote Creek nr Ventura	11118000	41.2	224.95	1933-2000	154
NF Matilija Crk abv Matilija HotSprgs	11116000	15.6	1142.02	1938+1956-2000	61
San Antonio Crk nr Casitas Hot Sprgs	11117500	51.2	307.55	1934-1983	130
Ventura Rvr at Ventura prior to 1960	11118500	188.0	200.0	1933-1959*	50

* Only values through 1959 are used in this study due to the completion of Casitas Dam in 1959.

Other gauges on the main stem of the Ventura River at Meiners Oaks and below Matilija Dam were considered but none of them had partial duration series data.

Statistical Analysis: The initial analysis of the partial duration series data for the five gauge sites was performed using a standard log-Pearson III analysis and bulletin 17B procedures as if the series were an annual series. Low outliers were detected and treated but no regional skew coefficients were applied. Regional skew coefficients for partial duration series have not been defined for this area of California. Tables 4 – 8 at the end of this report display the results of the log-Pearson III analysis.

These results were then transformed to the desired annual series 2- and 5-year return periods. A good theoretical and practical explanation for the transformation between partial duration series (PDS) and annual series is given in Maidment (1992) chapter 18, section 6.1. Because the arrival rate of events being modeled in the PDS is larger than with the annual series the calculated flow for return periods for the PDS must in fact be smaller than the corresponding flow for the same return period in an annual series. The simple transformation used in this study is given as equation 18.6.3b in Maidment (1992).

$$Tp = - 1/(Ln(1 - 1/Ta)$$

Where:

- Tp = calculated PDS return period
- Ta = annual series return period corresponding to Tp
- Ln = natural logarithm function

This equation is based on theoretical considerations that include the idea that the PDS will include all flow values that are hydrologically independent and are above a specified threshold flow. The theoretical basis also includes the idea that in some low flow years there may be no value to include with the PDS. These ideas may not be strictly enforced in the current study.

Similar, but slightly different, results of converting partial duration return periods to annual series return periods can be obtained from tables in other hydrology textbooks (Linsley, Kohler, Paulhus, 1975).

Tables 4 through 8 at the end of this report display the log-Pearson III analysis results for the PDS data of the five gauge records.

By application of the above equation for this analysis, the required annual series equivalent 2-year and 5-year flood peaks corresponds to the 2.542-year and the 5.517-year flood from the PDS log-Pearson III analysis. These flood peaks at the odd interval return periods were determined from a plot of peak discharge versus partial duration series return periods for the log-Pearson III results at each gauge site. Smooth curves were fit through the partial duration return periods versus peak flow data. The required peak flows for the odd intervals were determined from the smooth curves. Figures 1 through 5 at the end of this report graphically display the curve fitting part of the analysis. The annual series equivalent peak flows are given in the following table:

Table 2
Summary of 2- and 5-year Annual Series Equivalent Peak Flows from Partial Duration Series Analysis

Location	USGS Gauge Station Number	Drainage Area (sq. mi.)	Annual Series Equivalent	
			2-Year (ft^3/s)	5-Year (ft^3/s)
Coyote Creek near Oak View	11117600	13.2	600	1560
Coyote Creek near Ventura	11118000	41.2	6894	14835
NF Matilija Creek above Matilija Hot Springs	11116000	15.6	2984	7594
San Antonio Creek near Casitas Hot Springs	11117500	51.2	746	1541
Ventura River at Ventura Prior to 1960	11118500	188.0	4522	11057

Distribution of 2- and 5-Year flows along the Ventura River: Since there is only one gauge used in the partial duration series analysis that actually exists on the main stem of the Ventura River, it is not readily apparent how to extend the results to the remaining reaches of the river. For this study a plot of the 2- and 5-year annual equivalent peak flows determined for the various gauges versus the drainage area of the gauge site was created. Ordinary least squares regression lines were plotted for each set of return period and peak flow data. Figure 6 of this report displays this plot. From the plot it can be noted that the regression lines come very close to the data points for the Ventura River at Ventura with a drainage area of 188.0 square miles. The data for this gauge site largely controls the location of the regression line. For the smaller drainage areas much scatter in the data is noted. Overall the regressions are poor. Although the correlation coefficients are very low, the regression lines seem reasonable and they are based on gauge data from five stream gauges in the basin. Table 9 at the end of this report displays the final annual series equivalent 2- and 5-year peaks for various locations and drainage areas on the main stem of the Ventura River.

The only available alternative to using the drainage area regression would be to spread the 2- and 5-year peaks along the river based on ratios of flows for the various locations for the 10- through 500-year floods presented in the previous Corps and Federal Emergency Management Assistance (FEMA) reports for the Ventura River. Those reports provide no explanation for the distribution of the flood peaks along the river. Using this alternative method the results of this study would be based only on one stream gauge record, not five and then distributed according to an undocumented method.

Figure 7 displays the results of applying the regression of peak flow versus drainage area for the 2- and 5-year peak flows to various other locations on the main stem of the Ventura River. Higher return period flows determined in the earlier Reclamation study (USBR 2002) for various locations on the Ventura River are also displayed in both table 9 and figure 7. The distribution of the higher returns period flows follows from previous FEMA and Corps studies. Those previous studies did not provide any explanation for the distribution of peak flows along the main stem of the Ventura River. No significance can be attached to the fact that for the current study the 2- and 5-year peak flow distribution appears different. Different methodology, different data sets and different flow distribution techniques were used between the two studies.

<u>Flow Duration Curves</u>: In addition to peak flow estimates the study requires flow duration curves for sediment calculations at several points in the basin. A simple flow duration curve is formed by listing all available daily flow data in ascending order and then calculating the percent of time that a given daily flow value is not exceeded. For this study this was accomplished using available daily stream flow records from the USGS NWIS WEB page for California, and the EXCEL spreadsheet software. The following table lists the ten stream gauge records with daily flow records that could be used for flow duration curve construction.

Table 10 at the end of this report displays the numerical results associated with the flow duration curve analysis. Figures 8 through 17 at the end of this report display the flow duration curves for these ten gauge sites in graphical form.

Table 3
Stream Gauge Stations in the Ventura River Basin with Daily Flows for Flow Duration Curve Construction

Stream Gauge Location	USGS Number	Years of Record	Drainage Area (sq.mi.)
Matilija Creek above Reservoir at Matilija Hot Springs, CA	11114500	21	15.6
Matilija Creek at Matilija Hot Springs, CA	11115500	56	54.7
North Fork Matilija Creek at Matilija Hot Springs, CA	11116000	51	15.6
Ventura River near Ojai, CA	11116500	3	70.7
Ventura River near Meiners Oaks, CA	11116550	30	76.4
San Antonio Creek near Casitas Springs, CA	11117500	34	51.2
Coyote Creek near Oak View, CA	11117600	30	13.2
Santa Ana Creek near Oak View, CA	11117800	30	9.11
Coyote Creek near Ventura, CA*	11118000	56	41.20(2.00)
Ventura River near Ventura , CA	11118500	71	188

* Since the construction of Casitas Dam in 1959, the contributing area is considered to be 2.00 sq.mi.

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References:

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Ventura River 2- and 5-year Flood Peaks and Flow Duration Curves
For Use with
Matilija Dam Ecosystem Restoration Feasibility Study
Ventura County, California

SIGNATURE PAGE

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Table 4
Ventrua River at Ventura Partial Duration Series Analysis
USGS Gauge 1118500, Drainage Area =188.0 square miles, Datum = 200.0 feet

Mean of Logs	Std.Dev	Data Skew	Reg.Skew	Final Skew
3.4508	0.6397	-0.4163	0.0000	-0.4163

RANK	PlotPos	YEAR	Q	EXCEED.	FREQ.Q	LOW	HIGH
1	0.01961	0	39200.0	0.99000	59	25	111
2	0.03922	0	35000.0	0.98000	100	46	177
3	0.05882	0	29500.0	0.97500	119	57	207
4	0.07843	0	24600.0	0.96000	176	90	292
5	0.09804	0	23000.0	0.95000	213	113	347
6	0.11765	0	20000.0	0.90000	405	237	620
7	0.13725	0	18700.0	0.80000	850	549	1230
8	0.15686	0	17000.0	0.70000	1413	960	2000
9	0.17647	0	15200.0	0.60000	2145	1499	3023
10	0.19608	0	13900.0	0.57040	2405	1690	3393
11	0.21569	0	13000.0	0.50000	3126	2214	4440
12	0.23529	0	8000.0	0.42960	4038	2866	5804
13	0.25490	0	6010.0	0.40000	4497	3190	6508
14	0.27451	0	5000.0	0.30000	6540	4599	9754
15	0.29412	0	4930.0	0.20000	9952	6853	15523
16	0.31373	0	4840.0	0.10000	17249	11404	28914
17	0.33333	0	4740.0	0.05000	26424	16798	47195
18	0.35294	0	4670.0	0.04000	29782	18708	54198
19	0.37255	0	4500.0	0.02500	37505	23004	70822
20	0.39216	0	4330.0	0.02000	41479	25169	79630
21	0.41176	0	4050.0	0.01000	55098	32402	110915
22	0.43137	0	3350.0	0.00500	70635	40375	148369
23	0.45098	0	3330.0	0.00200	94088	51994	207776
24	0.47059	0	3220.0				
25	0.49020	0	3030.0				
26	0.50980	1	2840.0				
27	0.52941	1	2400.0				
28	0.54902	1	2120.0				
29	0.56863	1	2000.0				
30	0.58824	1	1910.0				
31	0.60784	1	1890.0				
32	0.62745	1	1500.0				
33	0.64706	1	1460.0				
34	0.66667	1	1440.0				
35	0.68627	1	1420.0				
36	0.70588	1	1200.0				
37	0.72549	1	1190.0				
38	0.74510	1	1070.0				
39	0.76471	1	1040.0				
40	0.78431	1	936.0				
41	0.80392	1	870.0				
42	0.82353	1	800.0				
43	0.84314	1	790.0				
44	0.86275	1	712.0				
45	0.88235	1	640.0				
46	0.90196	1	539.0				
47	0.92157	1	203.0				
48	0.94118	1	35.0				
49	0.96078	1	2.4				

50	0.98039	1	0.3

Table 5

Coyote Creek near Oak View Partial Duration Series Peaks

USGS Gauge 1117600, Drainage Area =13.2 square miles, Datum = 577.77 feet

Mean of Logs	Std.Dev	Data Skew	Reg.Skew	Final Skew
2.6704	0.5898	0.5332	0.0000	0.5332

RANK	PlotPos	YEAR	Q	EXCEED.	FREQ.Q	LOW	HIGH
1	0.00602	1983	11600.0	0.99000	34	25	45
2	0.01205	1978	10700.0	0.98000	43	32	56
3	0.01807	1969	9700.0	0.97500	47	35	60
4	0.02410	1969	8800.0	0.96000	57	43	72
5	0.03012	1995	8420.0	0.95000	63	48	80
6	0.03614	1992	8400.0	0.90000	90	71	112
7	0.04217	1978	7360.0	0.80000	146	118	178
8	0.04819	1980	6780.0	0.70000	213	175	255
9	0.05422	1969	6650.0	0.60000	298	248	355
10	0.06024	1995	6110.0	0.57040	329	274	391
11	0.06627	1993	5030.0	0.50000	415	348	494
12	0.07229	1995	4770.0	0.42960	528	444	630
13	0.07831	1986	4220.0	0.40000	587	494	702
14	0.08434	1966	3840.0	0.30000	866	725	1046
15	0.09036	1991	3820.0	0.20000	1398	1153	1725
16	0.09639	1962	3800.0	0.10000	2830	2260	3653
17	0.10241	1993	3750.0	0.05000	5264	4051	7122
18	0.10843	1993	3550.0	0.04000	6348	4826	8721
19	0.11446	1952	3440.0	0.02500	9275	6874	13154
20	0.12048	1965	3320.0	0.02000	11035	8080	15885
21	0.12651	1970	3150.0	0.01000	18534	13076	27930
22	0.13253	1958	3010.0	0.00500	30325	20628	47784
23	0.13855	1973	2960.0	0.00200	56354	36572	94041
24	0.14458	1965	2940.0				
25	0.15060	1988	2900.0				
26	0.15663	1958	2820.0				
27	0.16265	1980	2590.0				
28	0.16867	1983	2200.0				
29	0.17470	1958	2050.0				
30	0.18072	1959	1880.0				
31	0.18675	1965	1840.0				
32	0.19277	1957	1720.0				
33	0.19880	1978	1600.0				
34	0.20482	1978	1560.0				
35	0.21084	1975	1340.0				
36	0.21687	1957	1260.0				
37	0.22289	1992	1240.0				
38	0.22892	1995	1230.0				
39	0.23494	1986	1200.0				
40	0.24096	1973	1150.0				
41	0.24699	1978	1150.0				
42	0.25301	1992	1050.0				
43	0.25904	1983	1010.0				
44	0.26506	1993	1010.0				
45	0.27108	1976	1010.0				
46	0.27711	1963	978.0				
47	0.28313	1952	972.0				
48	0.28916	1958	954.0				
49	0.29518	1979	877.0				

50	0.30120	1978	870.0
51	0.30723	1991	850.0
52	0.31325	1995	844.0
53	0.31928	1969	770.0
54	0.32530	1970	765.0
55	0.33133	1966	741.0
56	0.33735	1958	739.0
57	0.34337	1966	736.0
58	0.34940	1979	692.0
59	0.35542	1960	674.0
60	0.36145	1958	674.0
61	0.36747	1954	616.0
62	0.37349	1974	585.0
63	0.37952	1977	560.0
64	0.38554	1976	558.0
65	0.39157	1986	545.0
66	0.39759	1983	533.0
67	0.40361	1959	527.0
68	0.40964	1996	503.0
69	0.41566	1961	500.0
70	0.42169	1952	496.0
71	0.42771	1983	487.0
72	0.43373	1980	481.0
73	0.43976	1983	474.0
74	0.44578	1956	468.0
75	0.45181	1967	467.0
76	0.45783	1973	462.0
77	0.46386	1982	436.0
78	0.46988	1980	431.0
79	0.47590	1992	422.0
80	0.48193	1973	420.0
81	0.48795	1978	386.0
82	0.49398	1992	386.0
83	0.50000	1971	381.0
84	0.50602	1964	376.0
85	0.51205	1982	371.0
86	0.51807	1978	354.0
87	0.52410	1992	353.0
88	0.53012	1954	353.0
89	0.53614	1967	350.0
90	0.54217	1995	338.0
91	0.54819	1957	336.0
92	0.55422	1958	318.0
93	0.56024	1986	304.0
94	0.56627	1996	296.0
95	0.57229	1967	293.0
96	0.57831	1971	280.0
97	0.58434	1982	279.0
98	0.59036	1983	267.0
99	0.59639	1995	261.0
100	0.60241	1983	258.0
101	0.60843	1975	257.0
102	0.61446	1994	252.0
103	0.62048	1967	237.0
104	0.62651	1981	228.0
105	0.63253	1997	224.0
106	0.63855	1986	220.0

107	0.64458	1987	216.0
108	0.65060	1952	215.0
109	0.65663	1952	210.0
110	0.66265	1978	200.0
111	0.66867	1986	200.0
112	0.67470	1992	197.0
113	0.68072	1977	194.0
114	0.68675	1963	180.0
115	0.69277	1967	179.0
116	0.69880	1980	179.0
117	0.70482	1958	177.0
118	0.71084	1967	173.0
119	0.71687	1974	172.0
120	0.72289	1981	172.0
121	0.72892	1996	169.0
122	0.73494	1959	169.0
123	0.74096	1979	166.0
124	0.74699	1969	164.0
125	0.75301	1994	164.0
126	0.75904	1966	156.0
127	0.76506	1961	154.0
128	0.77108	1971	154.0
129	0.77711	1952	151.0
130	0.78313	1977	145.0
131	0.78916	1976	144.0
132	0.79518	1953	142.0
133	0.80120	1996	138.0
134	0.80723	1969	130.0
135	0.81325	1982	130.0
136	0.81928	1960	128.0
137	0.82530	1988	128.0
138	0.83133	1959	126.0
139	0.83735	1995	125.0
140	0.84337	1978	125.0
141	0.84940	1987	125.0
142	0.85542	1981	124.0
143	0.86145	1957	121.0
144	0.86747	1978	120.0
145	0.87349	1955	117.0
146	0.87952	1952	115.0
147	0.88554	1952	109.0
148	0.89157	1962	105.0
149	0.89759	1960	105.0
150	0.90361	1963	100.0
151	0.90964	1970	88.0
152	0.91566	1974	83.0
153	0.92169	1970	75.0
154	0.92771	1956	75.0
155	0.93373	1961	73.0
156	0.93976	1964	69.0
157	0.94578	1973	66.0
158	0.95181	1952	64.0
159	0.95783	1978	63.0
160	0.96386	1973	61.0
161	0.96988	1975	60.0
162	0.97590	1974	58.0
163	0.98193	1969	53.0

164	0.98795	1968	52.0
165	0.99398	1952	51.0

Table 6

Coyote Creek near Ventura Partial Duration Series Peaks

USGS Gauge 11118000, Drainage Area = 41.2 square miles, Datum = 224.95 feet

Mean of Logs	Std.Dev	Data Skew	Reg.Skew	Final Skew
3.7702	0.4510	0.6453	0.0000	0.6453

RANK	PlotPos	YEAR	Q	EXCEED.	FREQ.Q	LOW	HIGH
1	0.00645	1978	73000.0	0.99000	867	678	1071
2	0.01290	1995	65000.0	0.98000	1014	804	1240
3	0.01935	1998	62500.0	0.97500	1074	855	1308
4	0.02581	1969	60000.0	0.96000	1228	988	1483
5	0.03226	1983	56000.0	0.95000	1318	1066	1585
6	0.03871	1938	56000.0	0.90000	1707	1408	2025
7	0.04516	1978	49800.0	0.80000	2419	2041	2822
8	0.05161	1969	45000.0	0.70000	3188	2729	3684
9	0.05806	1943	44000.0	0.60000	4104	3548	4717
10	0.06452	1992	44000.0	0.57040	4418	3828	5074
11	0.07097	1980	40700.0	0.50000	5273	4586	6051
12	0.07742	1973	38300.0	0.42960	6338	5522	7283
13	0.08387	1969	38000.0	0.40000	6872	5989	7909
14	0.09032	1998	35500.0	0.30000	9273	8055	10762
15	0.09677	1933	34000.0	0.20000	13456	11563	15890
16	0.10323	1958	28400.0	0.10000	23461	19631	28738
17	0.10968	1962	25600.0	0.05000	38474	31228	49016
18	0.11613	1995	25100.0	0.04000	44712	35927	57695
19	0.12258	1952	23200.0	0.02500	60681	47728	80427
20	0.12903	1970	22800.0	0.02000	69831	54371	93732
21	0.13548	1973	22000.0	0.01000	106384	80277	148452
22	0.14194	1965	21600.0	0.00500	159037	116356	230541
23	0.14839	1966	21600.0	0.00200	264588	185967	402893
24	0.15484	1996	19800.0				
25	0.16129	1965	19600.0				
26	0.16774	1997	18300.0				
27	0.17419	1941	17300.0				
28	0.18065	1978	17200.0				
29	0.18710	1980	16400.0				
30	0.19355	1991	16300.0				
31	0.20000	1982	14710.0				
32	0.20645	1969	14400.0				
33	0.21290	1944	13000.0				
34	0.21935	1937	12800.0				
35	0.22581	1935	12500.0				
36	0.23226	1958	12100.0				
37	0.23871	1933	12000.0				
38	0.24516	1965	11600.0				
39	0.25161	1958	11500.0				
40	0.25806	1945	11500.0				
41	0.26452	1952	11400.0				
42	0.27097	1946	11300.0				
43	0.27742	1983	11200.0				
44	0.28387	1995	9830.0				
45	0.29032	1982	9660.0				
46	0.29677	1945	9600.0				
47	0.30323	1967	9110.0				
48	0.30968	1970	8800.0				
49	0.31613	1959	8280.0				

50	0.32258	1978	8260.0
51	0.32903	1973	8240.0
52	0.33548	1973	8240.0
53	0.34194	1977	8170.0
54	0.34839	1958	7690.0
55	0.35484	1957	7650.0
56	0.36129	1958	7420.0
57	0.36774	1975	7210.0
58	0.37419	1936	7200.0
59	0.38065	1958	7150.0
60	0.38710	1952	7070.0
61	0.39355	1974	6860.0
62	0.40000	1952	6530.0
63	0.40645	1978	6390.0
64	0.41290	1983	6330.0
65	0.41935	1979	6300.0
66	0.42581	1957	6080.0
67	0.43226	1967	6050.0
68	0.43871	1980	5720.0
69	0.44516	1940	5500.0
70	0.45161	1995	5390.0
71	0.45806	1974	5140.0
72	0.46452	1939	5000.0
73	0.47097	1991	5000.0
74	0.47742	1992	4960.0
75	0.48387	2000	4900.0
76	0.49032	1996	4870.0
77	0.49677	1946	4850.0
78	0.50323	1979	4820.0
79	0.50968	1971	4810.0
80	0.51613	1978	4730.0
81	0.52258	1966	4440.0
82	0.52903	1963	4400.0
83	0.53548	1954	4400.0
84	0.54194	1954	4350.0
85	0.54839	1982	4190.0
86	0.55484	1944	4100.0
87	0.56129	1956	3900.0
88	0.56774	1983	3740.0
89	0.57419	1979	3700.0
90	0.58065	1973	3680.0
91	0.58710	1971	3650.0
92	0.59355	1976	3650.0
93	0.60000	1970	3600.0
94	0.60645	1952	3370.0
95	0.61290	1980	3330.0
96	0.61935	1970	3300.0
97	0.62581	1996	3160.0
98	0.63226	1946	3150.0
99	0.63871	1941	3150.0
100	0.64516	1995	3110.0
101	0.65161	1967	3100.0
102	0.65806	1952	3090.0
103	0.66452	1995	3070.0
104	0.67097	1978	3060.0
105	0.67742	1946	3040.0
106	0.68387	1950	3000.0

107	0.69032	1957	2970.0
108	0.69677	1996	2960.0
109	0.70323	1970	2910.0
110	0.70968	1959	2900.0
111	0.71613	1998	2820.0
112	0.72258	1997	2790.0
113	0.72903	1980	2770.0
114	0.73548	1978	2750.0
115	0.74194	1961	2730.0
116	0.74839	1962	2720.0
117	0.75484	1980	2690.0
118	0.76129	1983	2660.0
119	0.76774	1978	2620.0
120	0.77419	1958	2610.0
121	0.78065	1994	2590.0
122	0.78710	1964	2590.0
123	0.79355	1991	2580.0
124	0.80000	1954	2570.0
125	0.80645	1962	2510.0
126	0.81290	1965	2440.0
127	0.81935	1959	2440.0
128	0.82581	1963	2410.0
129	0.83226	1981	2160.0
130	0.83871	1952	2150.0
131	0.84516	1998	2030.0
132	0.85161	1992	2020.0
133	0.85806	1971	1950.0
134	0.86452	1967	1940.0
135	0.87097	1992	1910.0
136	0.87742	1995	1790.0
137	0.88387	1982	1790.0
138	0.89032	1966	1720.0
139	0.89677	1946	1720.0
140	0.90323	1964	1710.0
141	0.90968	1992	1680.0
142	0.91613	1994	1590.0
143	0.92258	1992	1510.0
144	0.92903	1979	1490.0
145	0.93548	1973	1480.0
146	0.94194	1982	1480.0
147	0.94839	1984	1450.0
148	0.95484	1983	1410.0
149	0.96129	1981	1370.0
150	0.96774	1991	1360.0
151	0.97419	1946	1350.0
152	0.98065	1964	1340.0
153	0.98710	1960	1330.0
154	0.99355	1981	1310.0

Table 7

NF Matilija Creek above Matilija Hot Springs Partial Duration Series

USGS Gauge 11116000, Drainage Area = 15.6 square miles, Datum = 1142.02 feet

Mean of Logs	Std.Dev	Data Skew	Reg.Skew	Final Skew
3.3277	0.6069	0.1418	0.0000	0.1418

RANK	PlotPos	YEAR	Q	EXCEED.	FREQ.Q	LOW	HIGH
1	0.01613	1998	38000.0	0.99000	95	51	156
2	0.03226	1938	35000.0	0.98000	134	75	212
3	0.04839	1969	31200.0	0.97500	151	86	235
4	0.06452	1983	20850.0	0.96000	197	117	299
5	0.08065	1969	20800.0	0.95000	226	136	339
6	0.09677	1992	14900.0	0.90000	363	233	521
7	0.11290	1962	12200.0	0.80000	650	446	897
8	0.12903	1978	11000.0	0.70000	999	713	1351
9	0.14516	1970	9860.0	0.60000	1448	1059	1944
10	0.16129	1969	9040.0	0.57040	1608	1183	2158
11	0.17742	1965	9000.0	0.50000	2058	1527	2769
12	0.19355	1958	8600.0	0.42960	2638	1965	3577
13	0.20968	1965	8400.0	0.40000	2937	2188	4001
14	0.22581	1973	7530.0	0.30000	4317	3196	6025
15	0.24194	1980	6900.0	0.20000	6820	4950	9915
16	0.25806	1993	5940.0	0.10000	13002	9020	20374
17	0.27419	1988	4960.0	0.05000	22372	14787	37729
18	0.29032	1966	4640.0	0.04000	26248	17082	45291
19	0.30645	1958	4530.0	0.02500	36083	22738	65251
20	0.32258	1986	4410.0	0.02000	41654	25857	76975
21	0.33871	1959	3820.0	0.01000	63445	37632	125106
22	0.35484	1957	3600.0	0.00500	93666	53184	196443
23	0.37097	1965	3550.0	0.00200	151051	81169	342203
24	0.38710	1958	3130.0				
25	0.40323	1974	2650.0				
26	0.41935	1991	2430.0				
27	0.43548	1958	2400.0				
28	0.45161	1971	2200.0				
29	0.46774	1970	2150.0				
30	0.48387	1958	2100.0				
31	0.50000	1979	2060.0				
32	0.51613	1958	1900.0				
33	0.53226	1983	1850.0				
34	0.54839	1974	1720.0				
35	0.56452	1958	1610.0				
36	0.58065	1967	1410.0				
37	0.59677	1970	1410.0				
38	0.61290	1961	1390.0				
39	0.62903	1956	1230.0				
40	0.64516	1967	1210.0				
41	0.66129	1966	1200.0				
42	0.67742	2000	1170.0				
43	0.69355	1957	1120.0				
44	0.70968	1967	912.0				
45	0.72581	1996	800.0				
46	0.74194	1976	706.0				
47	0.75806	1996	619.0				
48	0.77419	1984	583.0				
49	0.79032	1994	570.0				

50	0.80645	1960	528.0
51	0.82258	1982	520.0
52	0.83871	1963	512.0
53	0.85484	1965	484.0
54	0.87097	1981	431.0
55	0.88710	1963	318.0
56	0.90323	1977	301.0
57	0.91935	1960	292.0
58	0.93548	1990	226.0
59	0.95161	1988	194.0
60	0.96774	1987	172.0
61	0.98387	1998	165.0

Table 8

San Antonio Creek near Casitas Hot Springs Partial Duration Series

USGS Gauge 11117500 Drainage Area = 51.2 square miles, Datum = 307.55 feet

Mean of Logs	Std.Dev	Data Skew	Reg.Skew	Final Skew
2.8161	0.4201	0.7302	0.0000	0.7302

RANK	PlotPos	YEAR	Q	EXCEED.	FREQ.Q	LOW	HIGH
1	0.00763	1969	8400.0	0.99000	117	91	144
2	0.01527	1980	8120.0	0.98000	133	106	163
3	0.02290	1938	8000.0	0.97500	140	111	170
4	0.03053	1978	5460.0	0.96000	157	126	190
5	0.03817	1933	5300.0	0.95000	167	135	201
6	0.04580	1978	4830.0	0.90000	209	172	249
7	0.05344	1969	4600.0	0.80000	286	240	334
8	0.06107	1966	4450.0	0.70000	367	313	425
9	0.06870	1983	4410.0	0.60000	462	399	533
10	0.07634	1943	4200.0	0.57040	495	428	569
11	0.08397	1958	3690.0	0.50000	583	505	670
12	0.09160	1965	3000.0	0.42960	691	601	796
13	0.09924	1965	2690.0	0.40000	745	648	859
14	0.10687	1983	2520.0	0.30000	987	856	1147
15	0.11450	1982	2300.0	0.20000	1402	1203	1660
16	0.12214	1952	2200.0	0.10000	2380	1988	2928
17	0.12977	1969	1960.0	0.05000	3826	3096	4908
18	0.13740	1983	1960.0	0.04000	4422	3541	5751
19	0.14504	1978	1900.0	0.02500	5942	4653	7951
20	0.15267	1962	1840.0	0.02000	6808	5275	9235
21	0.16031	1969	1770.0	0.01000	10253	7685	14500
22	0.16794	1978	1670.0	0.00500	15184	11015	22374
23	0.17557	1973	1670.0	0.00200	25013	17387	38872
24	0.18321	1967	1650.0				
25	0.19084	1970	1620.0				
26	0.19847	1973	1420.0				
27	0.20611	1944	1350.0				
28	0.21374	1941	1340.0				
29	0.22137	1958	1270.0				
30	0.22901	1939	1250.0				
31	0.23664	1978	1230.0				
32	0.24427	1980	1200.0				
33	0.25191	1958	1180.0				
34	0.25954	1957	1160.0				
35	0.26718	1967	1090.0				
36	0.27481	1980	1040.0				
37	0.28244	1979	1030.0				
38	0.29008	1980	1020.0				
39	0.29771	1957	1020.0				
40	0.30534	1945	1020.0				
41	0.31298	1950	1000.0				
42	0.32061	1956	992.0				
43	0.32824	1983	925.0				
44	0.33588	1965	852.0				
45	0.34351	1982	836.0				
46	0.35115	1958	835.0				
47	0.35878	1983	826.0				
48	0.36641	1936	810.0				
49	0.37405	1973	810.0				

50	0.38168	1970	800.0
51	0.38931	1974	799.0
52	0.39695	1935	750.0
53	0.40458	1945	710.0
54	0.41221	1971	691.0
55	0.41985	1966	684.0
56	0.42748	1973	663.0
57	0.43511	1979	662.0
58	0.44275	1980	618.0
59	0.45038	1969	600.0
60	0.45802	1967	595.0
61	0.46565	1946	578.0
62	0.47328	1970	570.0
63	0.48092	1958	562.0
64	0.48855	1975	554.0
65	0.49618	1974	547.0
66	0.50382	1958	540.0
67	0.51145	1961	535.0
68	0.51908	1982	527.0
69	0.52672	1973	504.0
70	0.53435	1965	504.0
71	0.54198	1973	504.0
72	0.54962	1962	499.0
73	0.55725	1959	496.0
74	0.56489	1983	491.0
75	0.57252	1971	486.0
76	0.58015	1966	474.0
77	0.58779	1963	470.0
78	0.59542	1957	458.0
79	0.60305	1967	454.0
80	0.61069	1967	450.0
81	0.61832	1958	430.0
82	0.62595	1959	418.0
83	0.63359	1972	414.0
84	0.64122	1975	405.0
85	0.64885	1962	390.0
86	0.65649	1977	390.0
87	0.66412	1983	387.0
88	0.67176	1958	374.0
89	0.67939	1967	358.0
90	0.68702	1971	348.0
91	0.69466	1980	333.0
92	0.70229	1980	325.0
93	0.70992	1981	311.0
94	0.71756	1963	307.0
95	0.72519	1964	304.0
96	0.73282	1982	301.0
97	0.74046	1961	300.0
98	0.74809	1977	292.0
99	0.75573	1980	290.0
100	0.76336	1967	289.0
101	0.77099	1971	282.0
102	0.77863	1973	282.0
103	0.78626	1958	268.0
104	0.79389	1959	268.0
105	0.80153	1976	266.0
106	0.80916	1970	261.0

107	0.81679	1974	255.0
108	0.82443	1955	255.0
109	0.83206	1978	254.0
110	0.83969	1960	249.0
111	0.84733	1975	243.0
112	0.85496	1972	240.0
113	0.86260	1978	238.0
114	0.87023	1978	238.0
115	0.87786	1964	235.0
116	0.88550	1979	231.0
117	0.89313	1978	224.0
118	0.90076	1973	220.0
119	0.90840	1968	214.0
120	0.91603	1979	211.0
121	0.92366	1974	204.0
122	0.93130	1979	201.0
123	0.93893	1967	198.0
124	0.94656	1970	194.0
125	0.95420	1960	184.0
126	0.96183	1966	170.0
127	0.96947	1971	155.0
128	0.97710	1958	154.0
129	0.98473	1978	152.0
130	0.99237	1964	151.0

Table 9

Peak flows distributed along Ventura River by River Mile

	Drainage Area (sq. mi.)	River Station (miles) (USBR 2000)	Gauge Location in1997 FIS for ratio n this Stud	Peak Flows in cfs (rounded to three significant digits) Return Period (years)						
				2*	5*	10	20	50	100	500
Upstream	54.3	15.6	Matilija	3058	7085	12500	15200	18800	21600	27900
Downstrea	70.4	14.0	Matilija	3252	7581	15000	18800	24000	27100	35200
At Baldwin	81	11.1	Matilija	3380	7907	16000	19800	24800	28300	36700
At Casitas	143	7.8	Ventura	4129	9816	35200	44400	56600	66600	89000
at Casitas	188	5.9	Ventura	4522	11057	36400	46400	59700	69700	93100
At Shell Cl	222	0.0	Ventura	5083	12248	41300	52700	67900	78900	105500

* 2- and 5-Year values from partial duration series analysis of five gauges then distributed along the river based on drainage area vs. discharge relationship

Table 10
Summary of Flow Duration Data for Stream Gauges in Ventura River Basin

	Location -->	Matilija Creek	Matilija Creek	North Fork Ma	Ventura River	Ventura River	San Antonio C	Coyote Creek	Santa Ana Cre	Coyote Creek	Ventura River near Ventura
Gauge	Number -->	11114500	11115500	11116000	11116500	11116550	11117500	11117600	11117800	11118000	11118500
Begin	Year -->	1949	1933	1933	1922	1959	1950	1959	1959	1927	1930
End	Year -->	1969	1988	1983	1924	1988	1983	1988	1988	1982	2000
Number	of Years -->	21	56	51	3	30	34	30	30	56	71
Drainage	Area -->	15.6	54.7	15.6	70.7	76.4	51.2	13.2	9.11	41.20(2.00)	188.0
Gauge	Datum -->	1160.2	900.0	1142.02	NA	NA	307.55	577.37	612.43	224.95	200.0
										*	*
		Flow Values									
Percent	0	0.3	0.1	0.10	1.0	0.0	0.0	0.0	0.0	0.00	0.00
Of Time	10	1.0	1.3	0.50	3.5	0.0	0.0	0.11	0.0	0.00	0.0
Flow is	20	1.6	2.3	0.85	4.5	0.0	0.0	0.24	0.0	0.00	0.0
Below	30	2.2	3.2	1.2	5.5	0.0	0.0	0.4	0.0	0.03	0.3
This	40	3.3	4.2	1.5	9	0.2	0.4	0.56	0.1	0.06	1.2
Value	50	4.5	5.5	2.2	12	1.5	0.9	0.8	0.2	0.09	3.0
	60	7.2	7.5	3.0	14	3.7	2.0	1	0.5	0.14	6.2
	70	9.5	11	4.1	20	6.9	3.6	1.5	0.9	0.23	11
	80	14	19	6.5	34	10	5.7	2.6	2.0	0.37	22
	90	35	53	15	58	15	15	6.9	6.5	0.68	63
	91	39	60	17	63	16	17	7.8	7.4	0.78	73
	92	46	70	19	69	17	20	9.2	8.7	1.0	88
	93	53	83	23	75	19	24	11	10	1.2	109
	94	63	103	27	87	21	28	14	12	1.8	140
	95	78	128	34	100	25	36	18	15	2.5	189
	96	96	163	43	124	30	49	23	20	5.3	275
	97	130	210	57	145	48	70	32	27	12	410
	98	212	276	84	181	158	102	50	43	30	609
	99	386	470	156	252	298	218	127	96	68	1180
	99.5	738	775	275	373	585	421	240	193	167	2100
	99.7	1070	1070	378	452	919	746	417	333	232	3300
	99.9	2890	2120	830	755	5120	1880	950	753	318	7130
	99.95	4050	3480	1390	764	7140	2920	1825	1010	430	10400
	99.99	6210	6840	2810	837	10600	4300	2500	1730	575	20000
	100	8610	8340	4980	910	13300	10400	2980	1900	612	22000

* Flow Duration Curve for Coyote Creek Near Ventura, CA, USGS No. 11118000 and
for Ventura River near Ventura, CA USGS No. 11118500 are both from 1959 to the present
after the construction of Casitas Dam.

in Ventura River Basin

San Antonio C	Coyote Creek	Santa Ana Cre	Coyote Creek	Ventura River near Ventura
11117500	11117600	11117800	11118000	11118500
1950	1959	1959	1927	1930
1983	1988	1988	1982	2000
34	30	30	56	71
51.2	13.2	9.11	41.20(2.00)	188.0
307.55	577.37	612.43	224.95	200.0
			*	*
lues --				
0.0	0.0	0.0	0.00	0.00
0.0	0.11	0.0	0.00	0.0
0.0	0.24	0.0	0.00	0.0
0.0	0.4	0.0	0.03	0.3
0.4	0.56	0.1	0.06	1.2
0.9	0.8	0.2	0.09	3.0
2.0	1	0.5	0.14	6.2
3.6	1.5	0.9	0.23	11
5.7	2.6	2.0	0.37	22
15	6.9	6.5	0.68	63
17	7.8	7.4	0.78	73
20	9.2	8.7	1.0	88
24	11	10	1.2	109
28	14	12	1.8	140
36	18	15	2.5	189
49	23	20	5.3	275
70	32	27	12	410
102	50	43	30	609
218	127	96	68	1180
421	240	193	167	2100
746	417	333	232	3300
1880	950	753	318	7130
2920	1825	1010	430	10400
4300	2500	1730	575	20000
10400	2980	1900	612	22000

18000 and
959 to the present

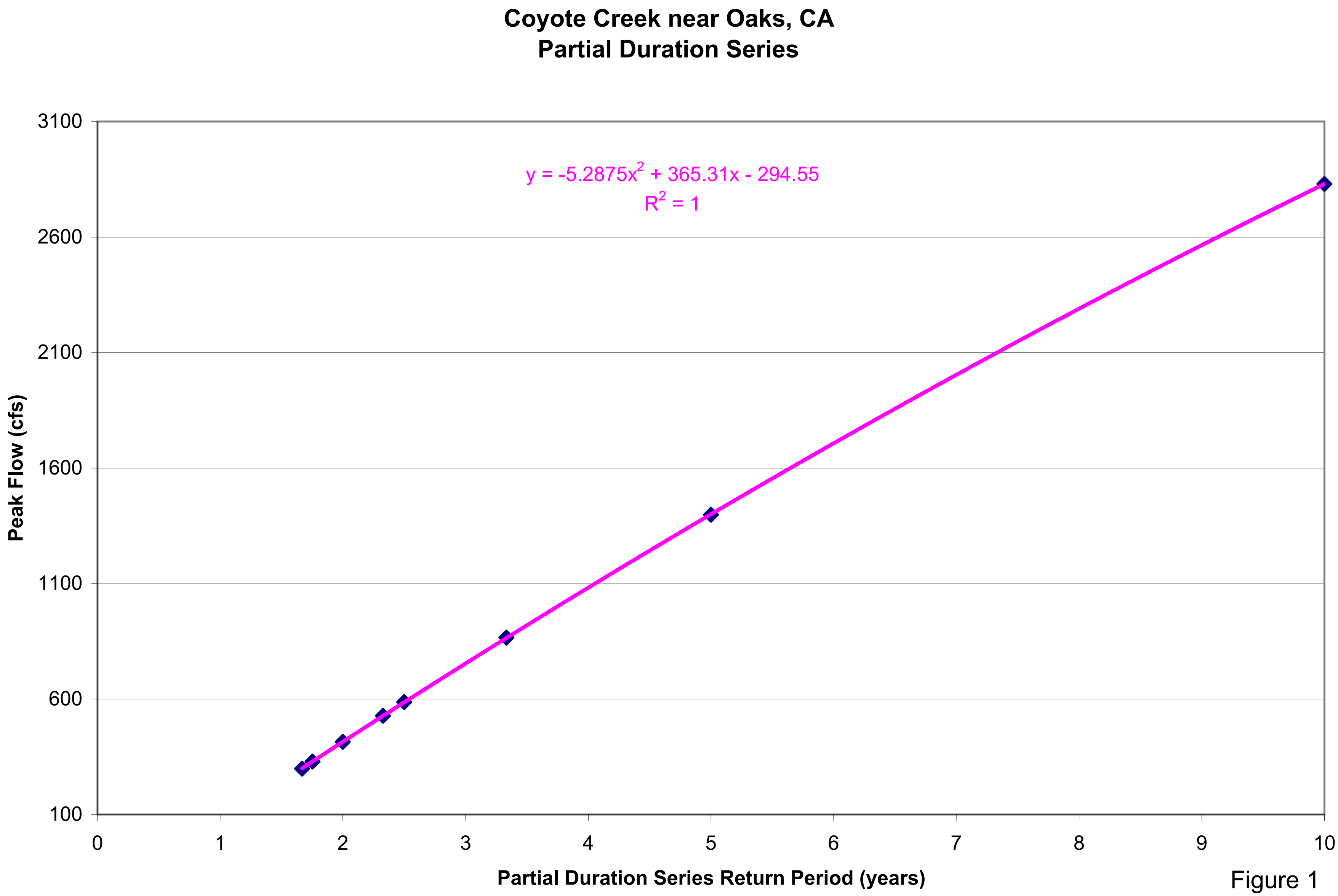
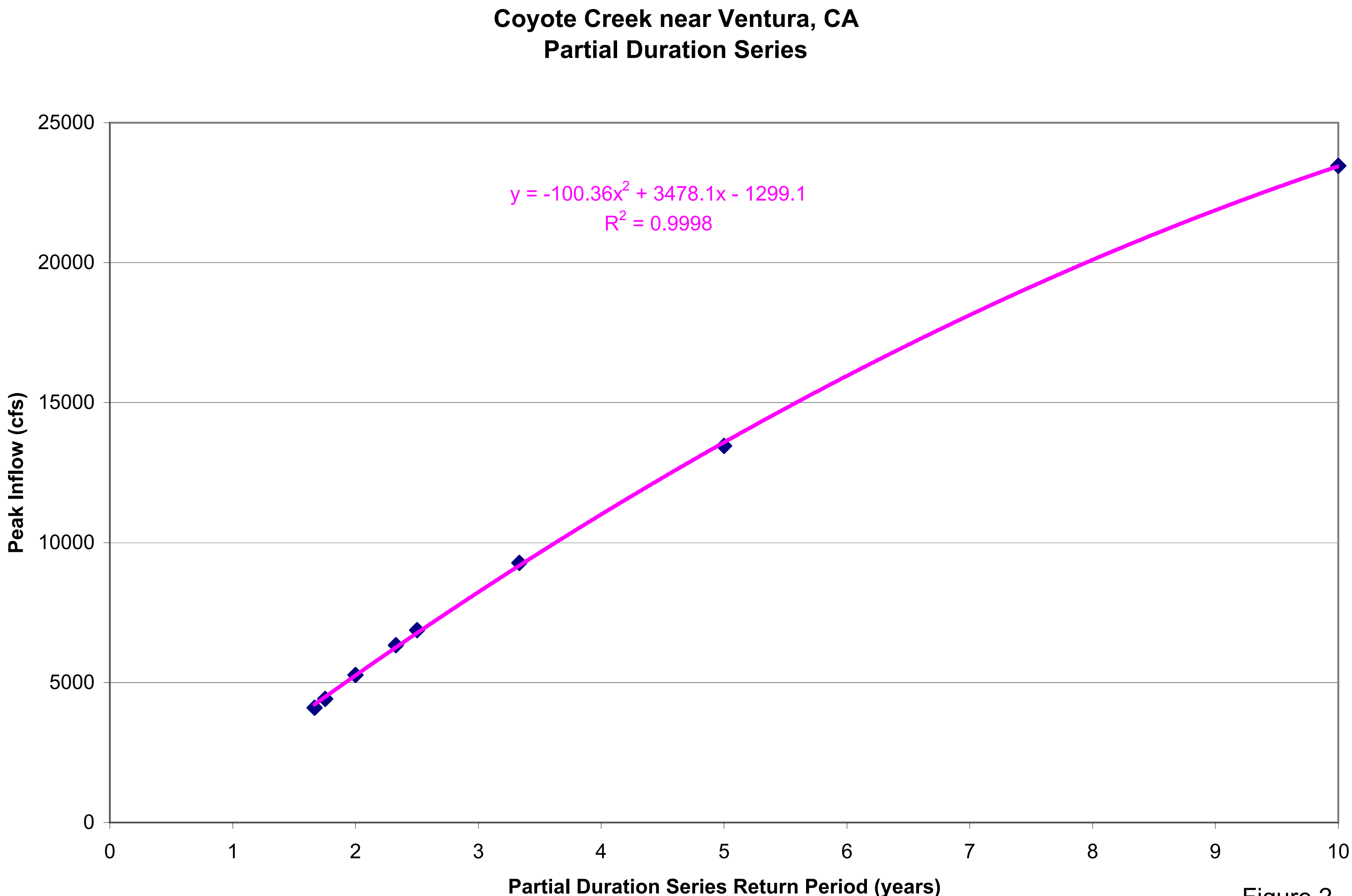

Coyote Creek near Oaks, CA
Partial Duration Series
y = -5.2875x2 + 365.31x - 294.55
R2 = 1
Peak Flow (cfs)
3100
2600
2100
1600
1100
600
100
0
1
2
3
4
5
6
7
8
9
10
Partial Duration Series Return Period (years)

Figure 1



Coyote Creek near Ventura, CA
Partial Duration Series
y = -100.36x2 + 3478.1x - 1299.1
R2 = 0.9998
Peak Inflow (cfs)
0
5000
10000
15000
20000
25000
0
1
2
3
4
5
6
7
8
9
10
Partial Duration Series Return Period (years)

Figure 2

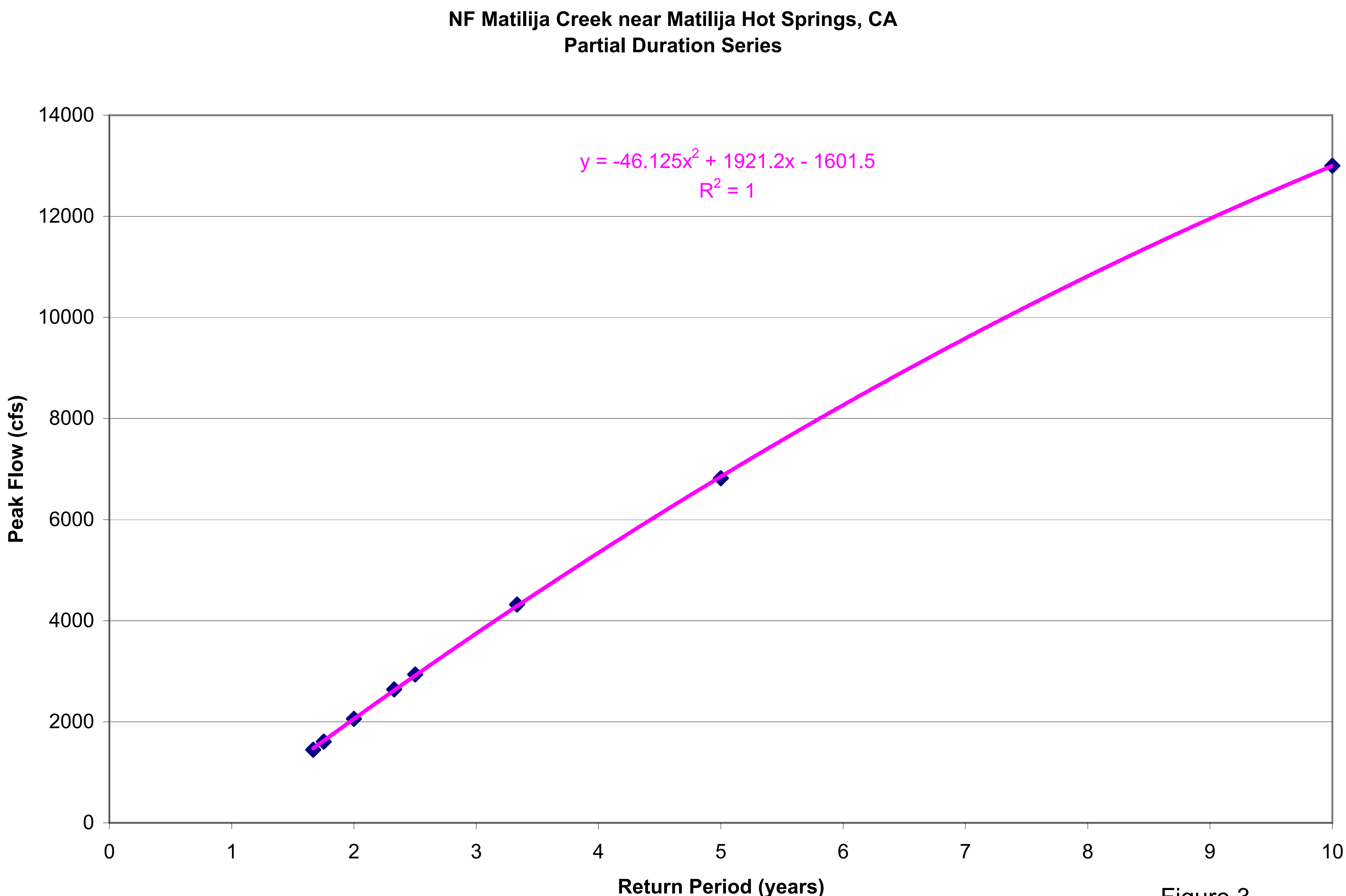
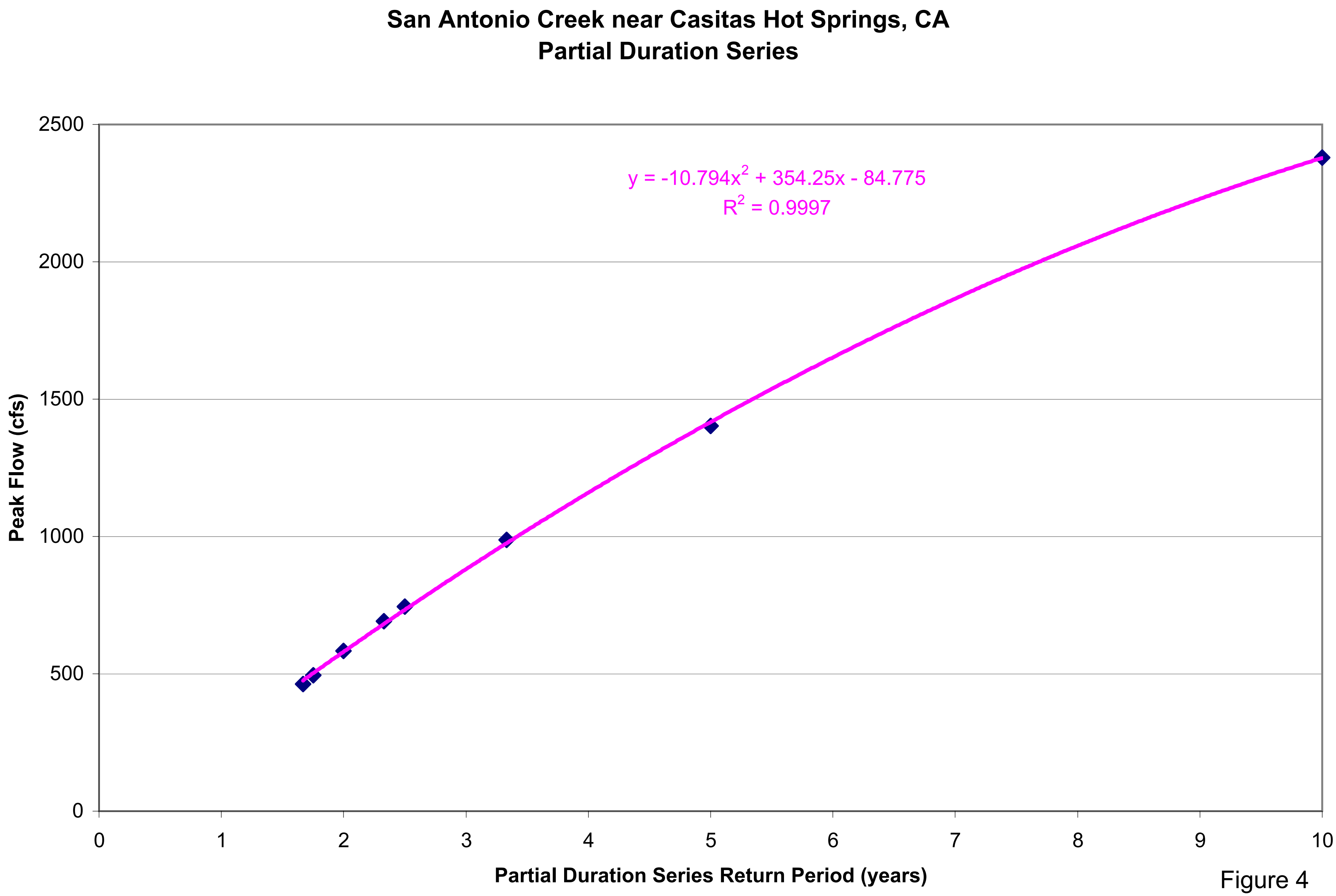

NF Matilija Creek near Matilija Hot Springs, CA
Partial Duration Series
y = -46.125x^2 + 1921.2x - 1601.5
R^2 = 1
Peak Flow (cfs)
Return Period (years)
0
2000
4000
6000
8000
10000
12000
14000
0
1
2
3
4
5
6
7
8
9
10

Figure 3



San Antonio Creek near Casitas Hot Springs, CA
Partial Duration Series
y = -10.794x2 + 354.25x - 84.775
R2 = 0.9997
Peak Flow (cfs)
0
500
1000
1500
2000
2500
0
1
2
3
4
5
6
7
8
9
10
Partial Duration Series Return Period (years)

Figure 4

Ventura River at Ventrua, CA
Partial Duration Series

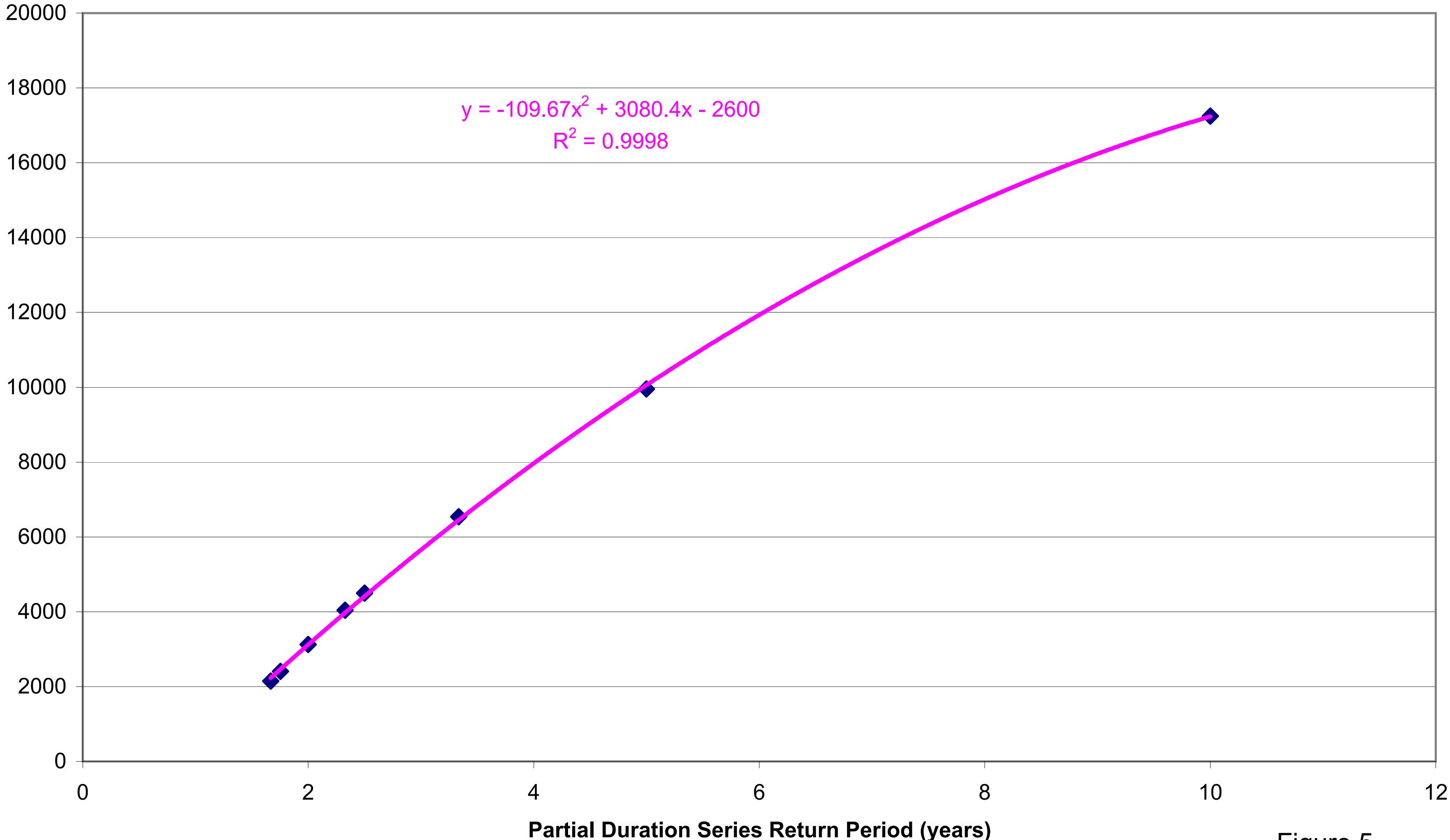


Figure 5

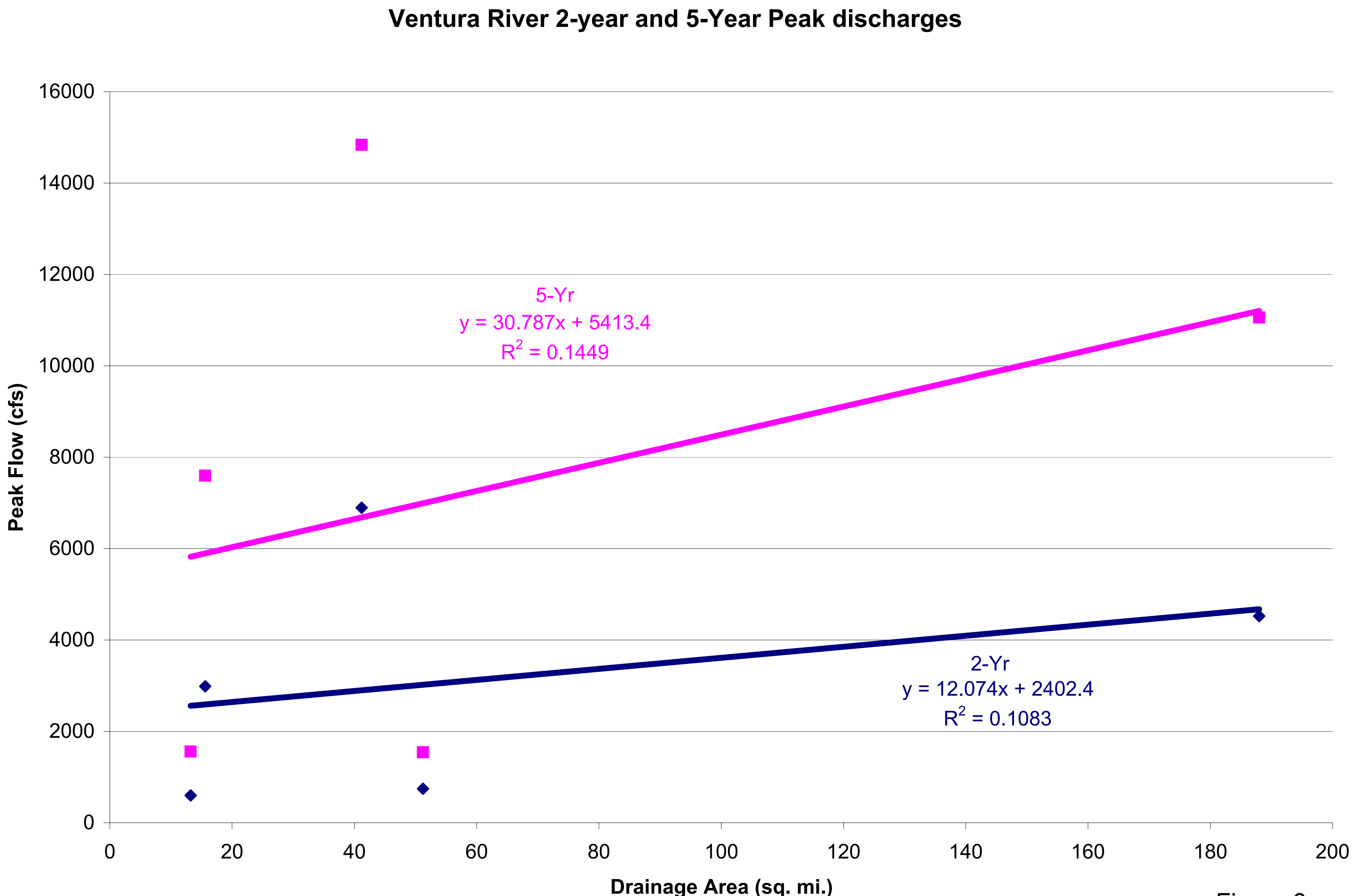

Ventura River 2-year and 5-Year Peak discharges
Peak Flow (cfs)
16000
14000
12000
10000
8000
6000
4000
2000
0
5-Yr
y = 30.787x + 5413.4
R^2 = 0.1449
2-Yr
y = 12.074x + 2402.4
R^2 = 0.1083
0
20
40
60
80
100
120
140
160
180
200
Drainage Area (sq. mi.)

Figure 6

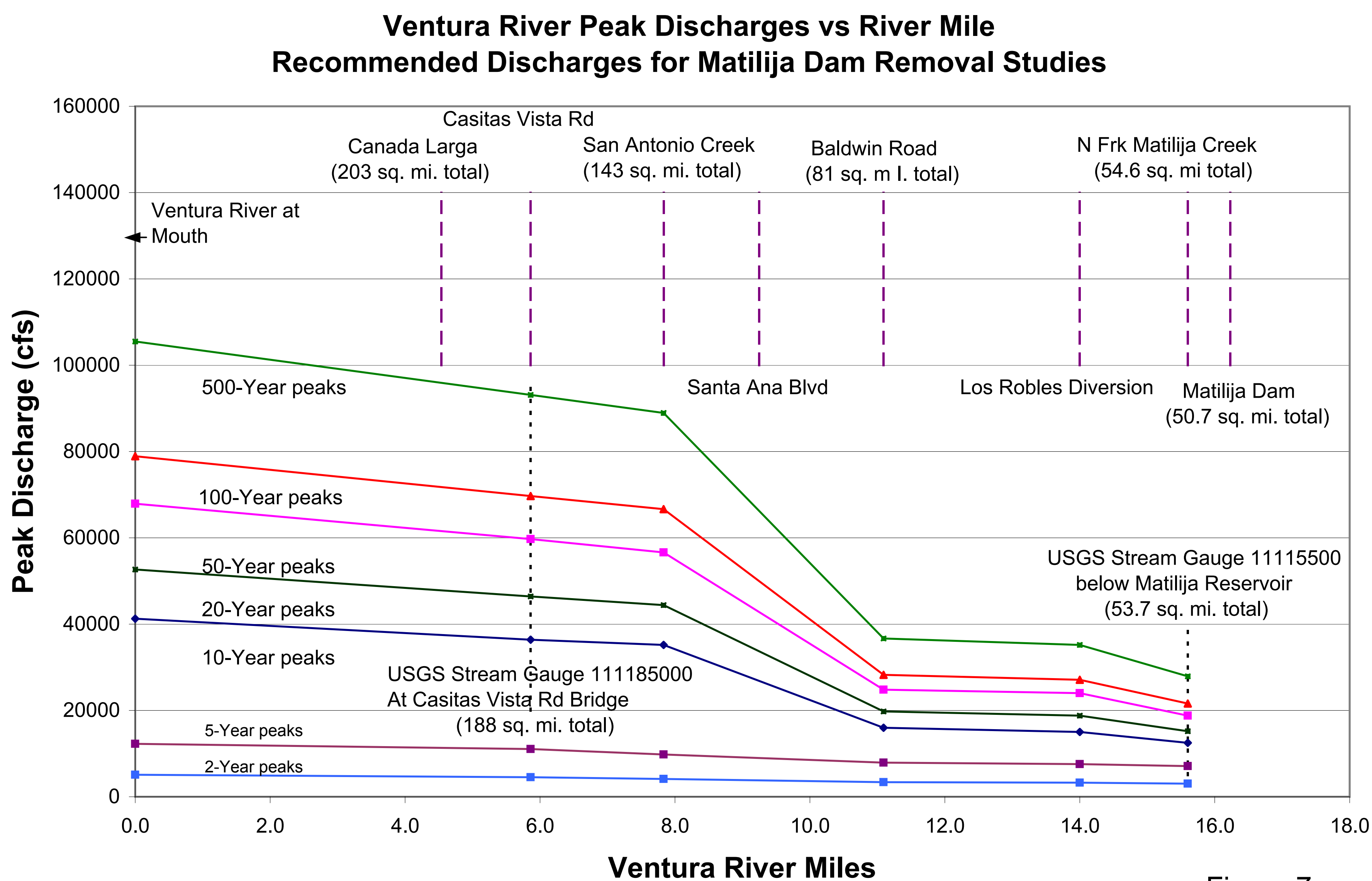

Ventura River Peak Discharges vs River Mile
Recommended Discharges for Matilija Dam Removal Studies
Peak Discharge (cfs)
Ventura River Miles
Casitas Vista Rd
Canada Larga
(203 sq. mi. total)
San Antonio Creek
(143 sq. mi. total)
Baldwin Road
(81 sq. m I. total)
N Frk Matilija Creek
(54.6 sq. mi total)
Ventura River at
Mouth
500-Year peaks
Santa Ana Blvd
Los Robles Diversion
Matilija Dam
(50.7 sq. mi. total)
100-Year peaks
50-Year peaks
20-Year peaks
10-Year peaks
USGS Stream Gauge 11115500
below Matilija Reservoir
(53.7 sq. mi. total)
USGS Stream Gauge 111185000
At Casitas Vista Rd Bridge
(188 sq. mi. total)
5-Year peaks
2-Year peaks
160000
140000
120000
100000
80000
60000
40000
20000
0
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0

Figure 7

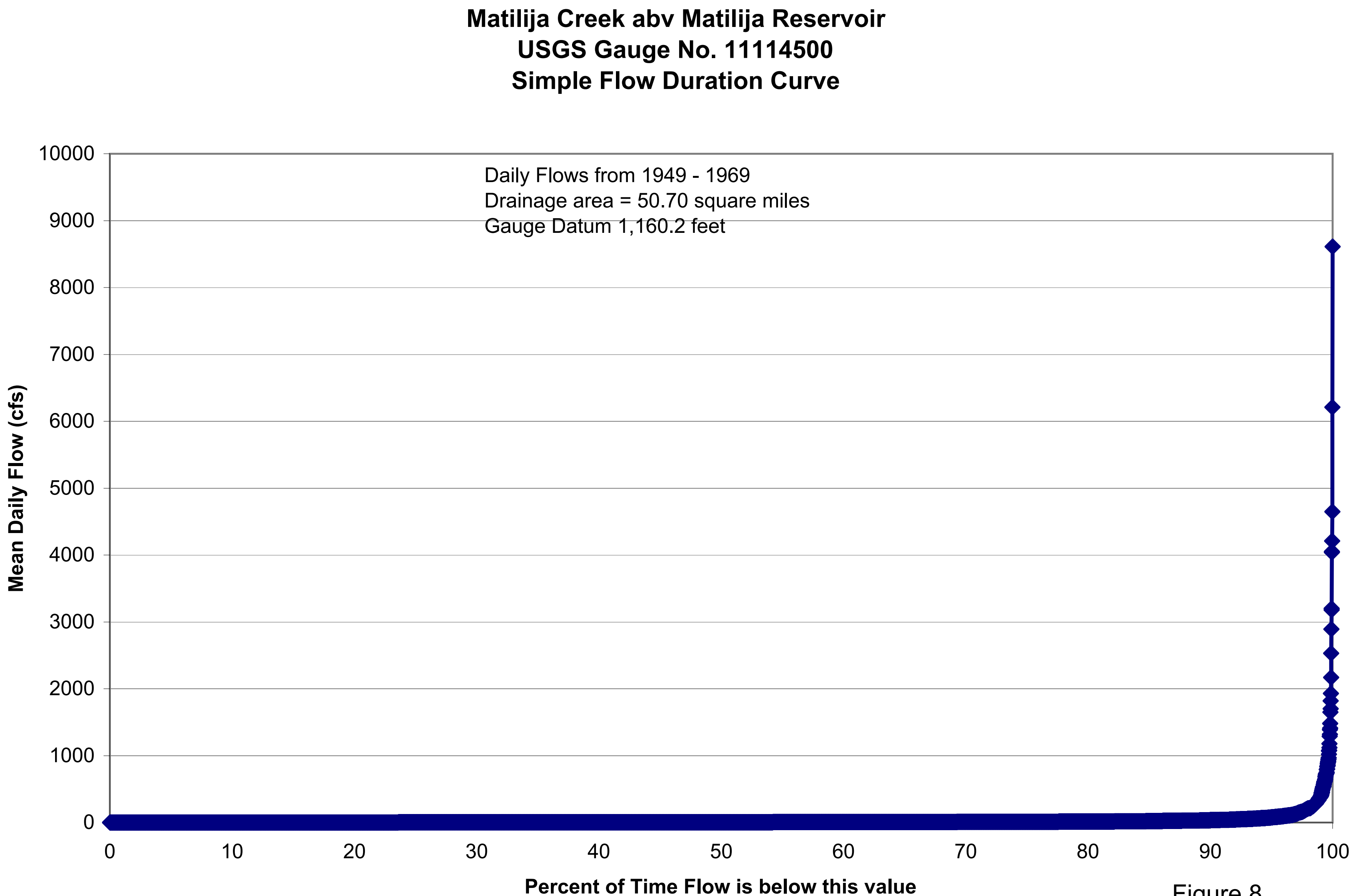

Matilija Creek abv Matilija Reservoir
USGS Gauge No. 11114500
Simple Flow Duration Curve
Daily Flows from 1949 - 1969
Drainage area = 50.70 square miles
Gauge Datum 1,160.2 feet
Mean Daily Flow (cfs)
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Flow is below this value

Figure 8

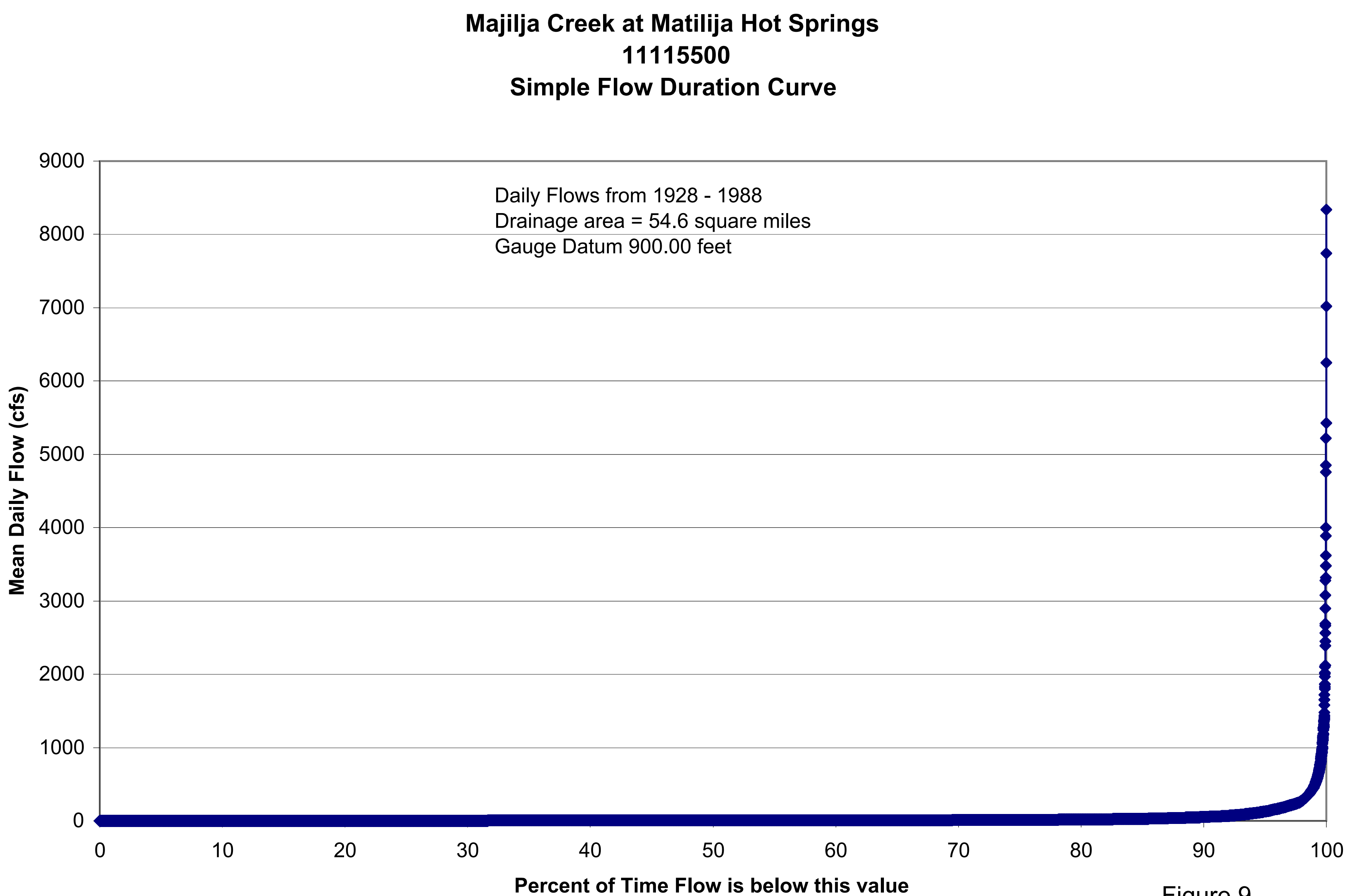

Majilja Creek at Matilija Hot Springs
11115500
Simple Flow Duration Curve
Daily Flows from 1928 - 1988
Drainage area = 54.6 square miles
Gauge Datum 900.00 feet
Mean Daily Flow (cfs)
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Flow is below this value

Figure 9

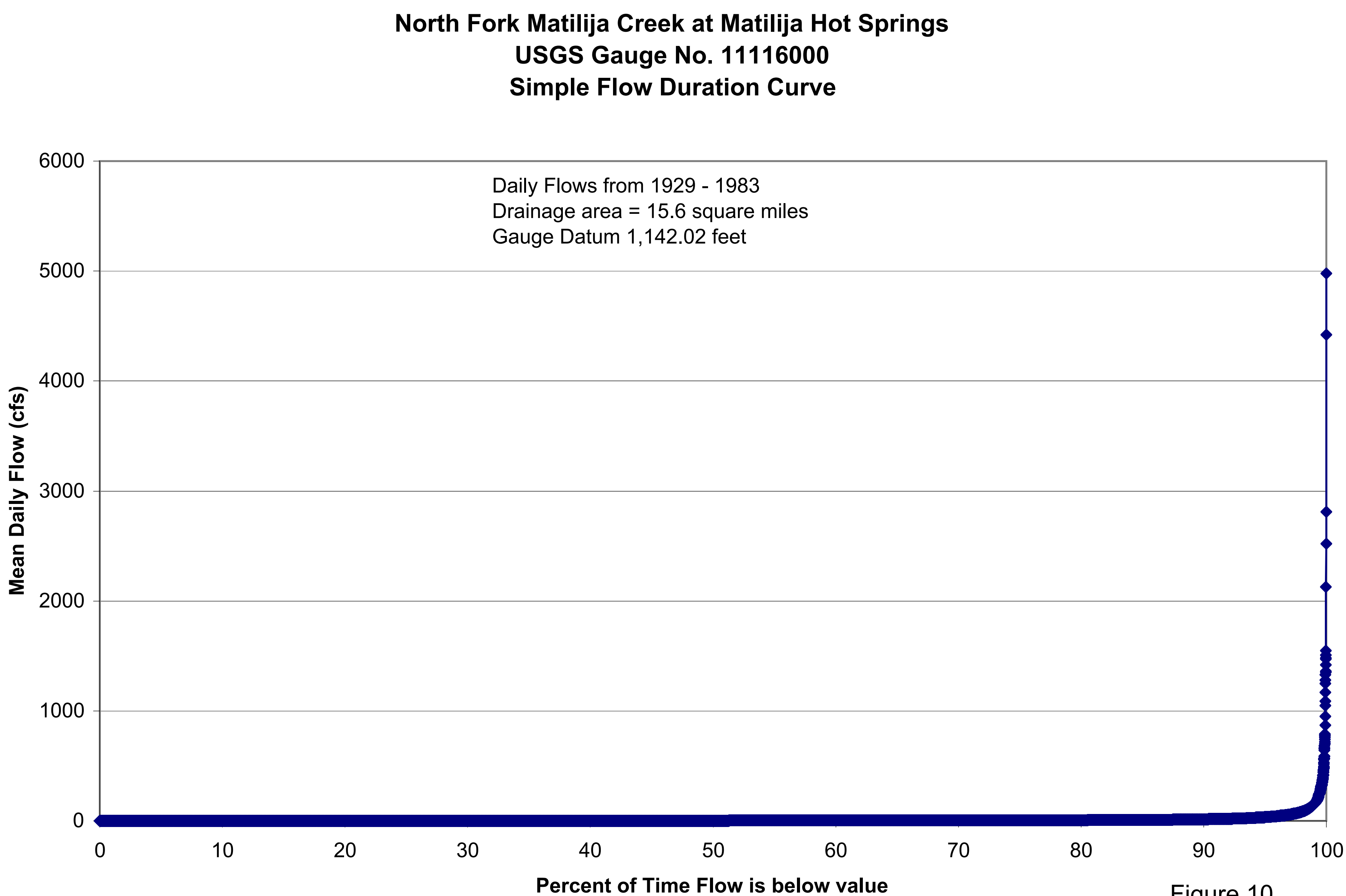
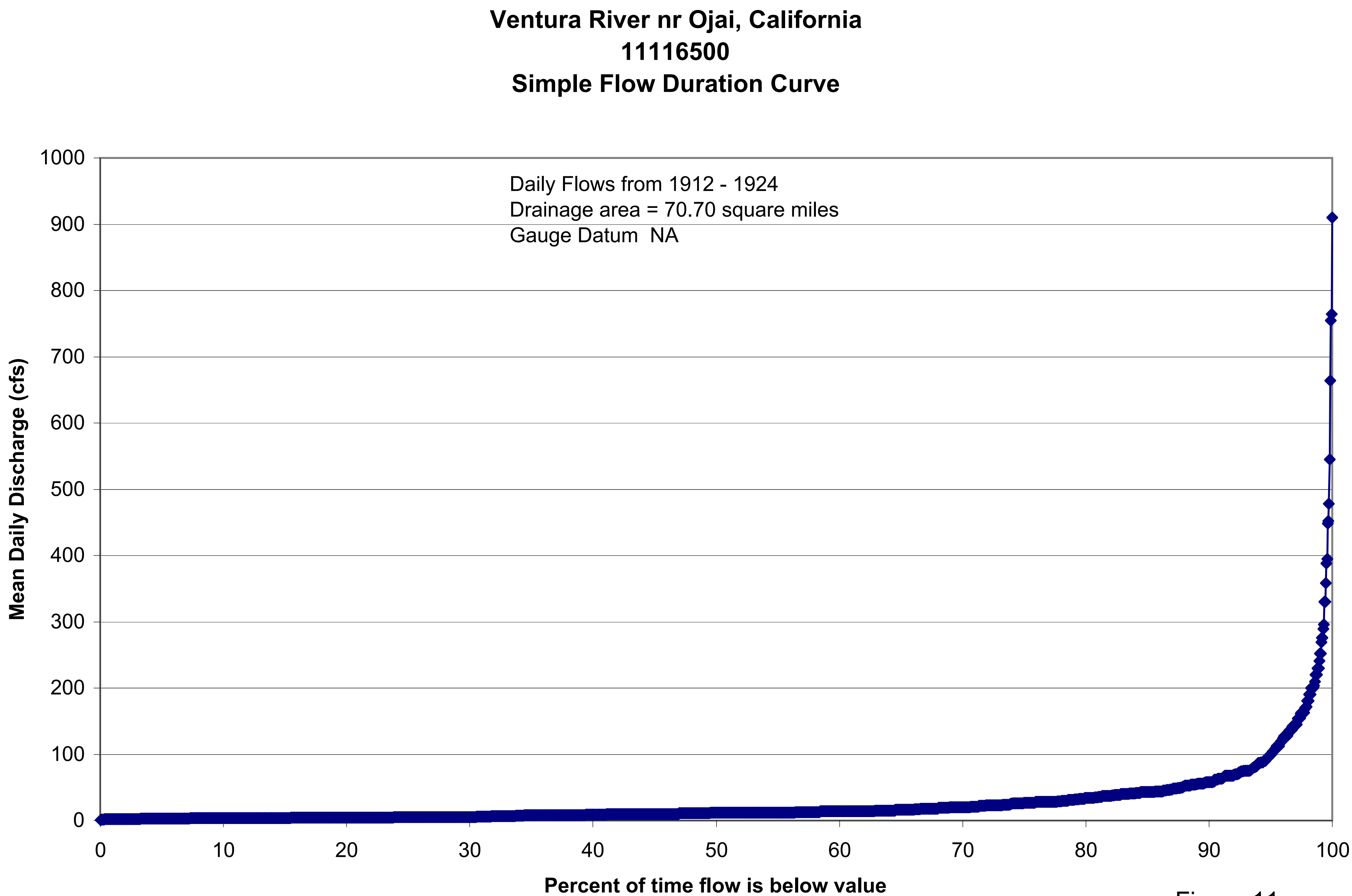

North Fork Matilija Creek at Matilija Hot Springs
USGS Gauge No. 11116000
Simple Flow Duration Curve
Daily Flows from 1929 - 1983
Drainage area = 15.6 square miles
Gauge Datum 1,142.02 feet
Mean Daily Flow (cfs)
6000
5000
4000
3000
2000
1000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Flow is below value

Figure 10



Ventura River nr Ojai, California
11116500
Simple Flow Duration Curve
Daily Flows from 1912 - 1924
Drainage area = 70.70 square miles
Gauge Datum NA
Mean Daily Discharge (cfs)
1000
900
800
700
600
500
400
300
200
100
0
0
10
20
30
40
50
60
70
80
90
100
Percent of time flow is below value

Figure 11

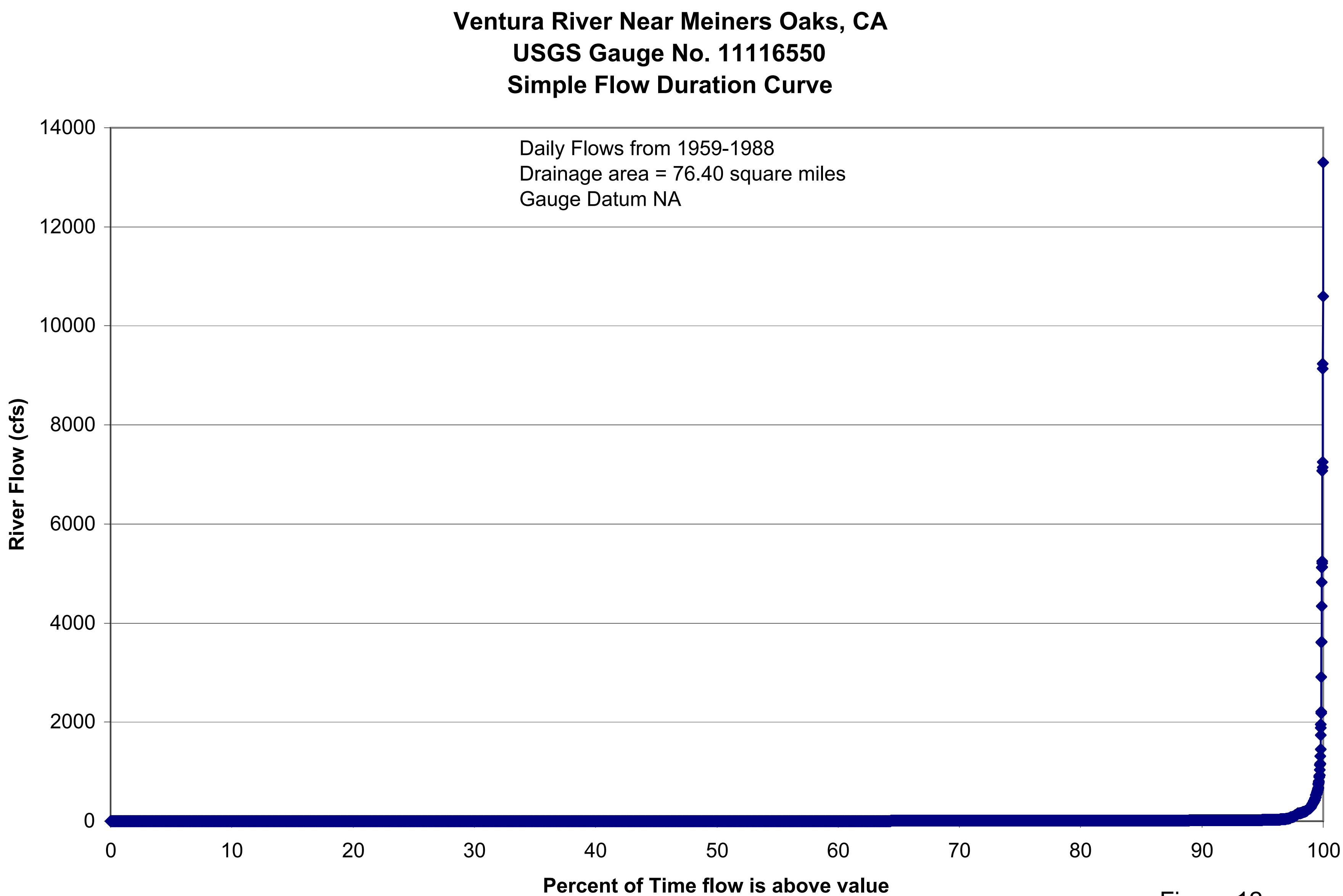
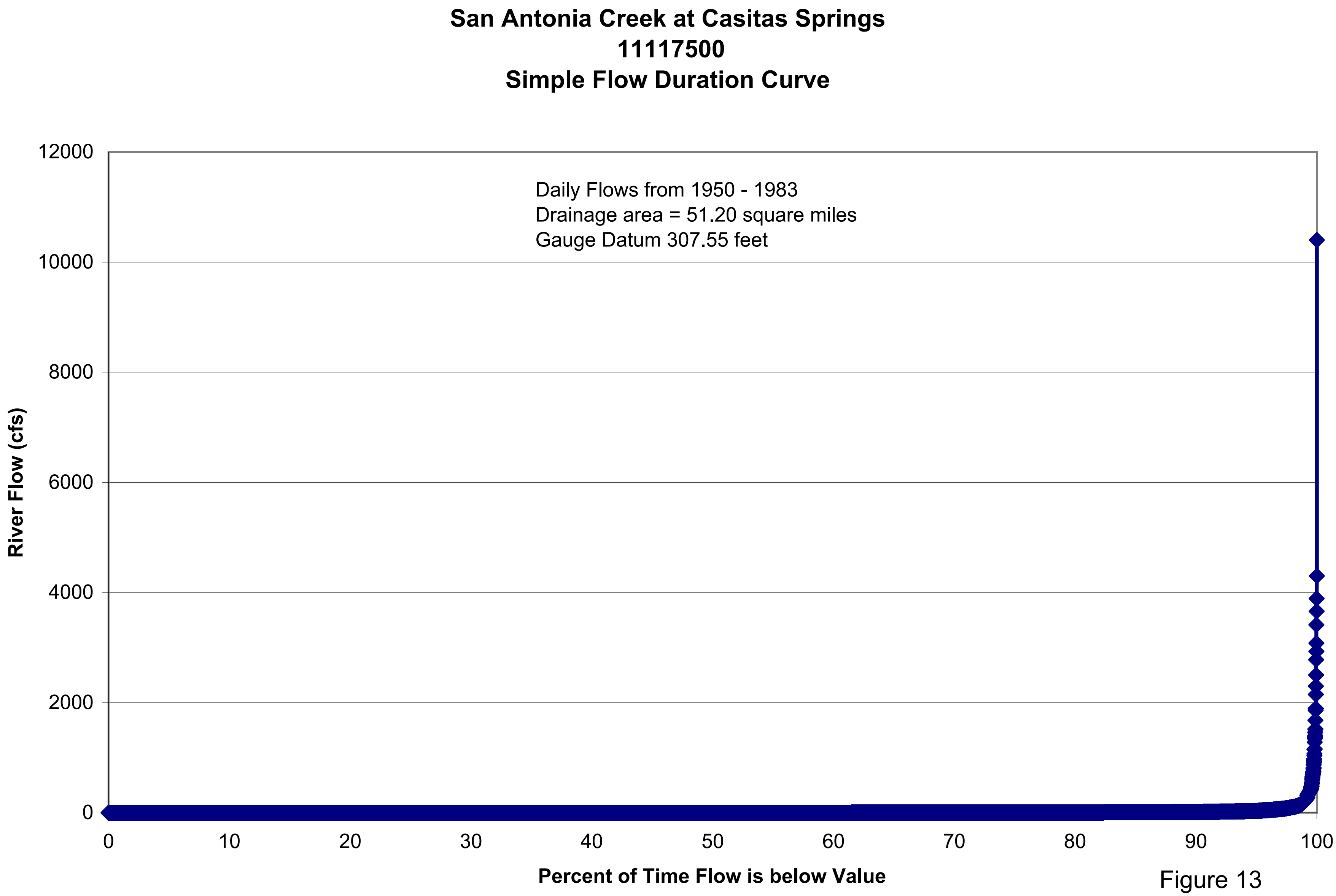

Ventura River Near Meiners Oaks, CA
USGS Gauge No. 11116550
Simple Flow Duration Curve
Daily Flows from 1959-1988
Drainage area = 76.40 square miles
Gauge Datum NA
River Flow (cfs)
0
2000
4000
6000
8000
10000
12000
14000
0
10
20
30
40
50
60
70
80
90
100
Percent of Time flow is above value

Figure 12



San Antonia Creek at Casitas Springs
11117500
Simple Flow Duration Curve
Daily Flows from 1950 - 1983
Drainage area = 51.20 square miles
Gauge Datum 307.55 feet
River Flow (cfs)
12000
10000
8000
6000
4000
2000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Flow is below Value

Figure 13

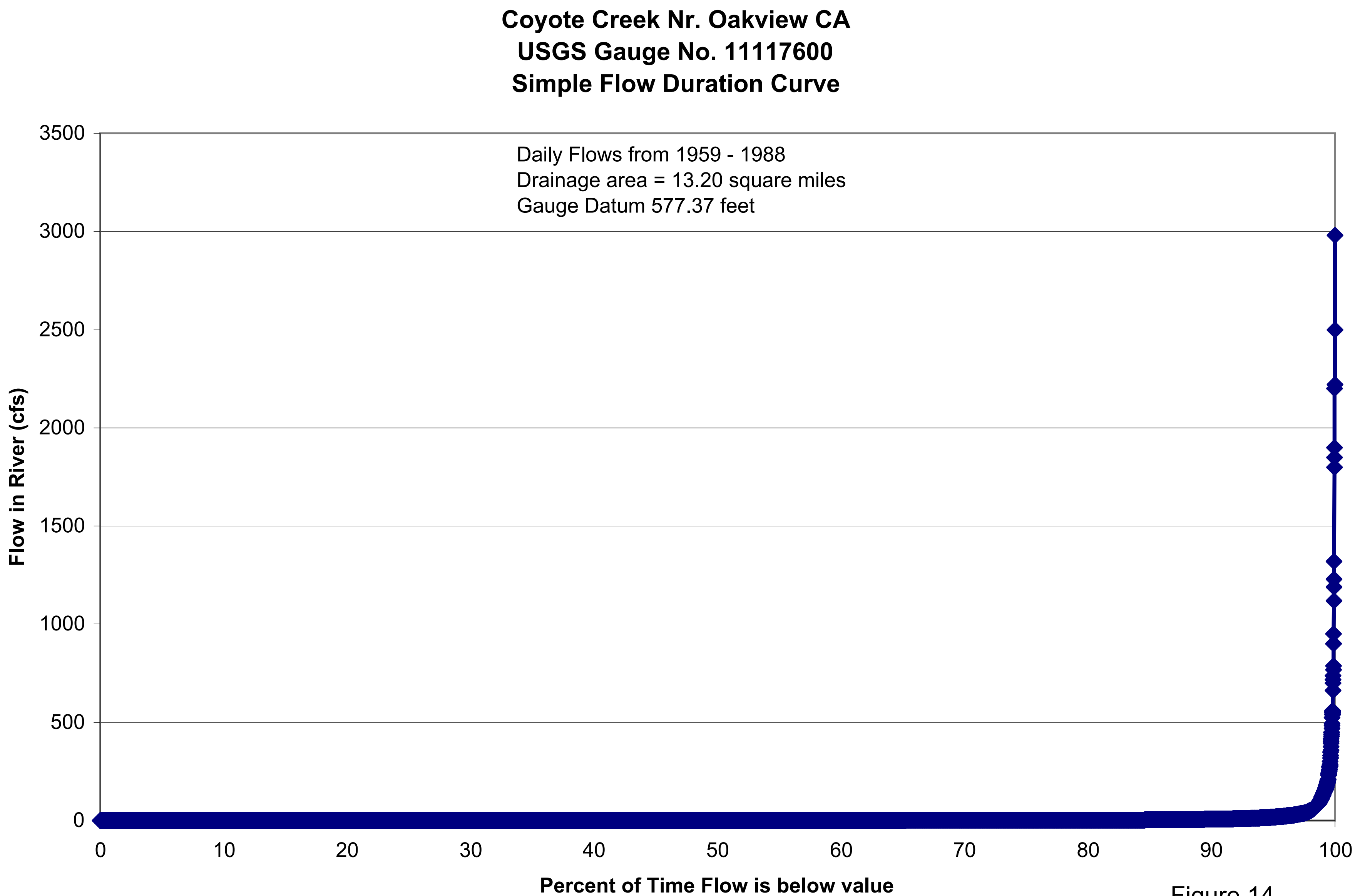

Coyote Creek Nr. Oakview CA
USGS Gauge No. 11117600
Simple Flow Duration Curve
Daily Flows from 1959 - 1988
Drainage area = 13.20 square miles
Gauge Datum 577.37 feet
Flow in River (cfs)
0
500
1000
1500
2000
2500
3000
3500
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Flow is below value

Figure 14

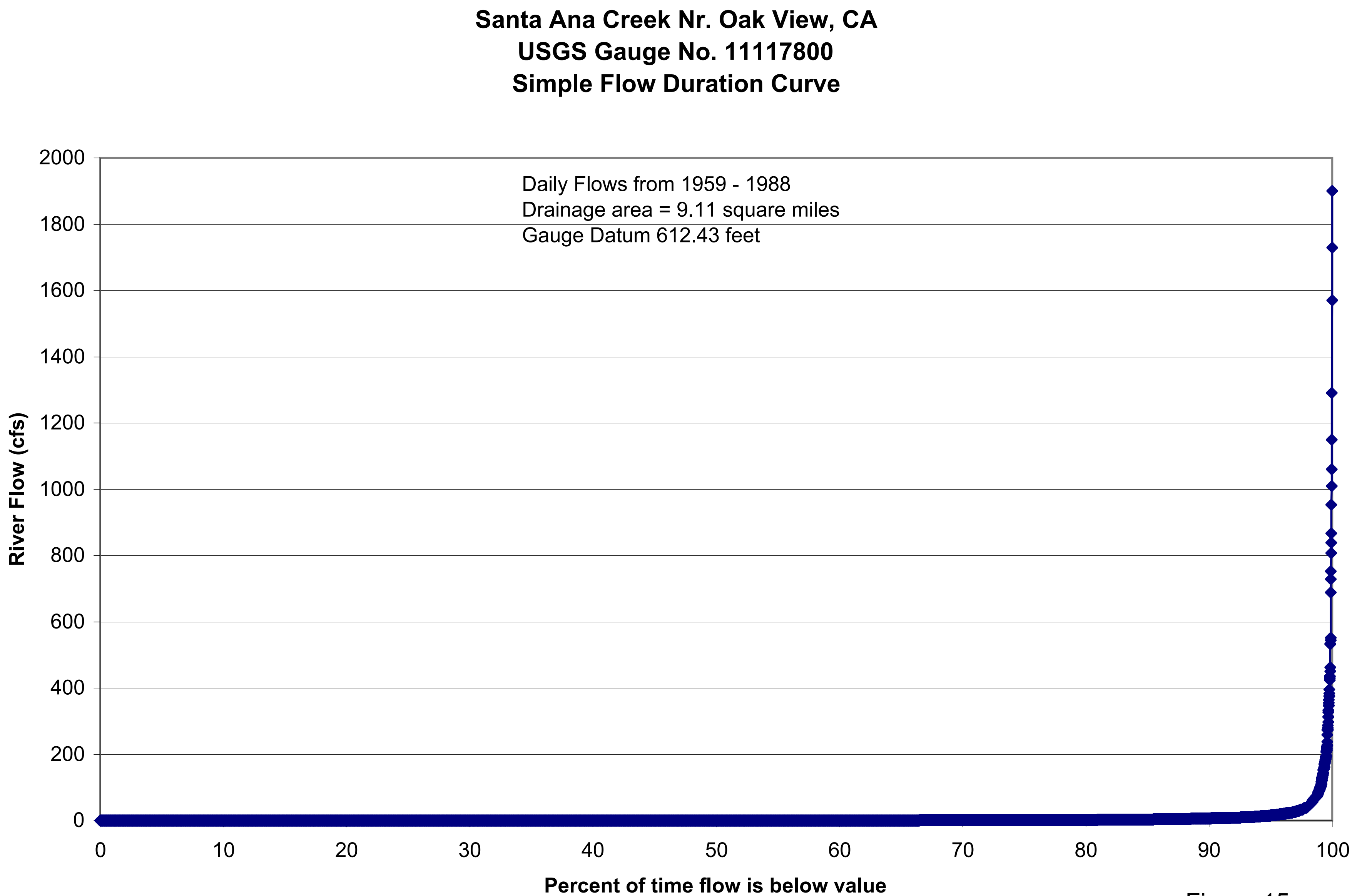

Santa Ana Creek Nr. Oak View, CA
USGS Gauge No. 11117800
Simple Flow Duration Curve
Daily Flows from 1959 - 1988
Drainage area = 9.11 square miles
Gauge Datum 612.43 feet
River Flow (cfs)
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0
10
20
30
40
50
60
70
80
90
100
Percent of time flow is below value

Figure 15

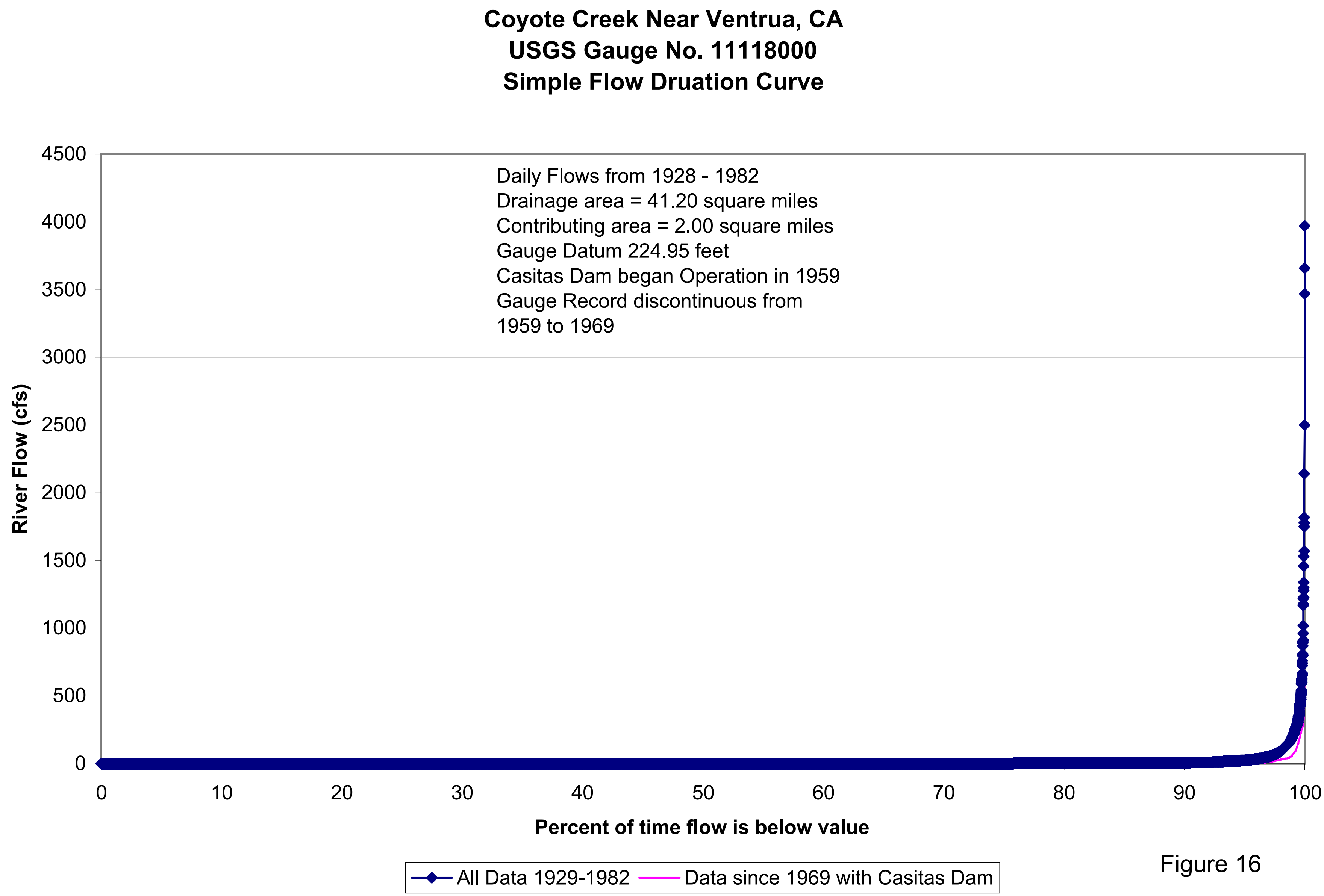
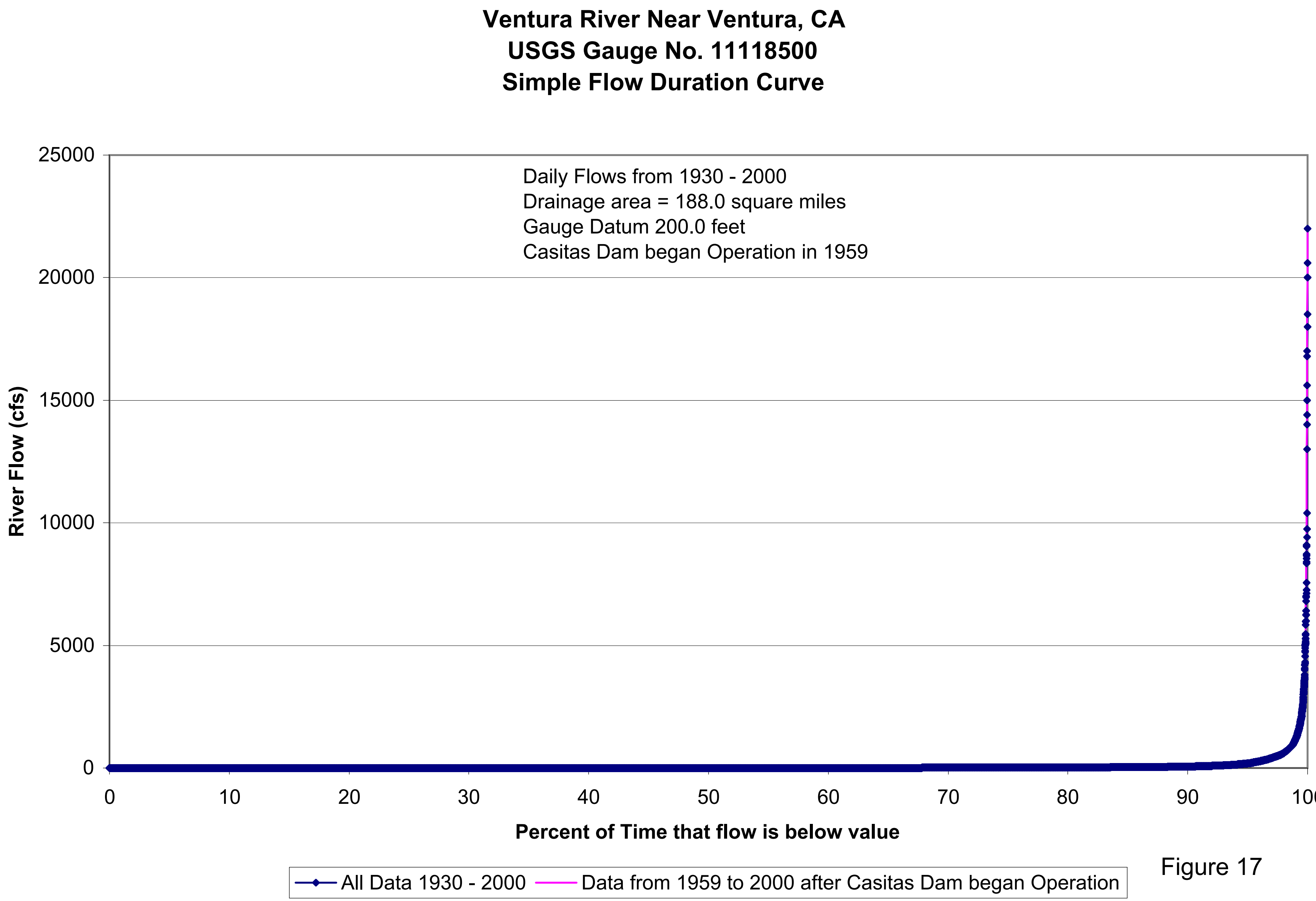

Coyote Creek Near Ventrua, CA
USGS Gauge No. 11118000
Simple Flow Druation Curve
Daily Flows from 1928 - 1982
Drainage area = 41.20 square miles
Contributing area = 2.00 square miles
Gauge Datum 224.95 feet
Casitas Dam began Operation in 1959
Gauge Record discontinuous from
1959 to 1969
River Flow (cfs)
4500
4000
3500
3000
2500
2000
1500
1000
500
0
0
10
20
30
40
50
60
70
80
90
100
Percent of time flow is below value
All Data 1929-1982
Data since 1969 with Casitas Dam

Figure 16



Ventura River Near Ventura, CA
USGS Gauge No. 11118500
Simple Flow Duration Curve
Daily Flows from 1930 - 2000
Drainage area = 188.0 square miles
Gauge Datum 200.0 feet
Casitas Dam began Operation in 1959
River Flow (cfs)
25000
20000
15000
10000
5000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time that flow is below value
All Data 1930 - 2000
Data from 1959 to 2000 after Casitas Dam began Operation

Figure 17

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DATE	PEER REVIEWER(S)	COD
2/21/02	Robert E. Swain	D853
PEER REVIEW NOT REQUIRED		Author's Initi

D-8530
PRJ-13.00

February 20, 2002

MEMORANDUM

TO: Team Leader, Matilija Dam Ecosystem Restoration Feasibility Study, Ventura County, California
Attention: D-8540 (Greimann)

FROM: Kenneth L. Bullard, Hydraulic Engineer
Flood Hydrology Group
Technical Service Center

SUBJECT: Ventura River Peak Flow Flood Frequency Study for use with Matilija Dam Ecosystem Restoration Feasibility Study, Ventura County, California

Attached is the study you requested relative to flood peak flows along the Ventura River below Matilija Dam. This a revised version of the study from the earlier version dated February 11, 2002. The revisions in this version include a more appropriate title, inclusion of 20-year flood estimates, and more discussion about old stream gauge sites on the Ventura River that were not included in the current analysis. These additional discussions and revisions were included in the report after reviews and comments by Mitch Declau and Blair Greimann, D-8540.

The results of this report are similar at the 100-year level to the previous USBR "Appraisal Report." No technique that uses the complete peak flow data set could be found that produced satisfying results for the entire 10- to 500-year return period range. Most standard techniques significantly over estimate the peak flows in this range of return periods when compared to the experienced flood peaks over the last 68 years. The selected technique does provide peak flow estimates consistent with the experienced data and is believed to best represent the flood potential of the Ventura River.

Additional studies will be undertaken to define peak flows for the 2- to 5-year range using partial duration series data and different analysis techniques.

If you have any questions about this report please call me at 303-445-2539.

Kenneth L. Bullard

Attachment

cc: D-8530 (Bullard/Schreiner/File-2) (w/att to ea)

TECHNICAL SERVICE CENTER
Denver, Colorado

Ventura River Peak Flow Flood Frequency Study For Use With Matilija Dam Ecosystem Restoration Feasibility Study Ventura County, California

Prepared by
Kenneth L. Bullard
Hydraulic Engineer
Flood Hydrology Group

U.S. Department of the Interior
Bureau of Reclamation

FEBRUARY 2002

UNITED STATES DEPARTMENT OF THE INTERIOR

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to tribes.

BUREAU OF RECLAMATION

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

U.S. DEPARTMENT OF THE INTERIOR
MARCH 3, 1849
WATER RESOURCES MANAGEMENT.....FOR THE FUTURE
U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Ventura River Peak Flow Flood Frequency Study
For Use With
Matilija Dam Ecosystem Restoration Feasibility Study
Ventura County, California

SIGNATURE PAGE

Prepared by:

Kenneth L. Bullard

Date Feb. 21, 2002

Kenneth L. Bullard, Hydraulic Engineer
Flood Hydrology Group

Peer Reviewed by:

Robert E. Swain

Date 2/21/02

Robert E. Swain, Hydraulic Engineer
Flood Hydrology Group

Ventura River Peak Flow Flood Frequency Study
For Use With
Matilija Dam Ecosystem Restoration Feasibility Study
Ventura County, California

Authorization: The Bureau of Reclamation was contracted by the Los Angeles District of the Corps of Engineers to perform necessary hydrologic and hydraulic computations related to the sediment disposition problems that may be created by potentially removing Matilija Dam on the Ventura River. Part of the required studies is to update existing flood plain maps using new peak flow values based on an additional 30 years of data the last study in 1970. Specifically the Flood Hydrology Group of the Technical Service Center of the Bureau of Reclamation in Denver, Colorado received authorization to conduct this study by means of a Service Agreement dated June 2001.

Basic Data Availability: Figure 1 of this report displays a basin map and general location map of the Ventura River. The areas of interest for this study are on the main stem of the Ventura River from the Pacific Ocean upstream to the location of Matilija Reservoir. Three mainstream gauges with sufficient data exist on the Ventura River that will impact the current study effort.

Stream Gauges Used for Ventura River Peak Flow Analysis				
USGS Station Number	Location	Drainage Area (sq.mi.)	Period of Record	Source
11114500	Matilija Creek Ab Reservoir Nr Matilija Hot Springs, CA	50.7	1948 - 1969 (destroyed)	USGS
11115500	Matilija Creek at Matilija Hot Springs, CA	54.7	1927 - Present	USGS
11118500	Ventura River Nr Ventura, CA (at bridge on Casitas Vistas Road)	188	1929 - Present	USGS

The largest recorded peak flow for the Ventura River Nr Ventura, CA was 60,000 ft^3/s in 1969. This is the largest peak flow in 68 years of record and should serve as a close approximation to any estimate of the 100-year peak flow value for this site.

Other gauges exist on nearby tributaries but are not considered useful to the current analysis. For the peak flows in the river near Matilija Reservoir the gauge record above the reservoir (USGS No. 11114500) is considered to be the best source of peak flow information for the 1948-1969 period. To extend this record the peak flow values from the gauge immediately downstream (USGS No. 11115500) can be used for the years 1927-1947, and for 1970 to the present.

Initial storage at Matilija Reservoir began in 1948. Following a large flood in 1969 Matilija Reservoir became essentially full of sediment and lost almost all of its flood storage capability. The upstream gauge site was destroyed during this flood event. During subsequent years the downstream gauge record is believed to closely approximate the reservoir inflow since little flood storage space is available behind Matilija Dam. Thus combining of the two gauge records for Matilija Creek is possible to create a complete 73-year peak flow record (1927-2000).

Just below Matilija Reservoir, Matilija Creek joins with the North Fork of Matilija Creek (drainage area 15.6 square miles). The combined flows become known as the Ventura River from this point downstream to the Pacific Ocean. The gauge record for the North Fork (USGS No. 1116000) has data from 1933 to 1983 with one missing year, for a total of 49 years of peak flow data. Peak flows from the North Fork of Matilija Creek do not coincide in time with peak flows from the main Matilija Creek. The larger of the two peak flows for any year usually comes from the larger drainage area on the main Matilija Creek. Except for one or two years the peak flows from the USGS record on the North Fork of Matilija Creek cannot be combined or substituted for the peak flow record from the other main stream sites. The gauge record for North Fork Matilija Creek was not used in this analysis.

Two other gauges with peak flow data on the main stem of the Ventura River were considered for use in this study.

The gauge for the Ventura River near Meiners Oaks, CA (USGS No. 11116550) has peak flow data from water years 1960 through 1982. The gauge is located immediately downstream from Robles diversion dam and is also downstream for Matilija dam. The USGS considers the records at this gauge site prior to 1978 to be “poor.” The gauge was relocated 500 feet downstream and with a datum 4.15 feet lower in 1978 in an attempt to provide better quality stage values. The gauge was destroyed by a flood on March 1, 1983 and has never been replaced. The official USGS comments for the gauge record include the fact that since 1959 flows up to 500 ft^3/s may be diverted at Robles diversion dam. The gauge record for this site reflects only the flows released below Robles diversion. The peak flow data for this gauge includes 23 years of record of which 14 years show peak flows less than 500 ft^3/s. These low peaks are assumed to be significantly affected by the diversion. The three largest recorded peak flows are 28,000 ft^3/s, 19,900 ft^3/s and 10,000 ft^3/s in 1969, 1978 and 1973, respectively. Each of these three peak flows is only an estimate since the gauge height for each of these events is unknown. A simple log-Pearson III peak flow analysis of the entire 23-year peak flow record provides a station skew value of positive 0.58. This is indicative of a strongly regulated peak flow record and is much different than the other non-regulated peak flow records in the area that show large negative skew values. This gauge record has not been included in any previous study of the peak flows for the main stem of the Ventura River. This stream gauge record was considered unsuitable for inclusion in this study.

The gauge for the Ventura River near Ojai, CA (USGS No. 11116500) was also considered. This stream gauge record has peak flows for only three years, 1922 to 1924. This is an insufficient number of peak flows to provide any meaningful statistical analysis. This gauge record was also considered unsuitable for inclusion in this study.

Tables 1 and 2 at the end of this report present the basic peak flow data available for Matilija Creek at Matilija Hot Springs and Ventura River near Ventura, respectively. Beginning in about 1989 data collection for these gauge sites was taken over by various local agencies. Not all years following 1989 have peak flow data available.

<u>Past Studies</u>: The current FEMA FIS (Flood Insurance Study)[1] was published and revised in 1997. This 1997 FIS report presents flood plain delineation maps for the main stem of the Ventura River representing 100-year peak flow conditions. This report references other previous FPI (Flood Plain Information) studies prepared by the Corps of Engineers[2,3] in 1970, where the basic peak flow discharge calculations were created. In 1970 the Corps of Engineers performed hydrologic frequency studies based on the available peak flow information from the same gauges listed above including the large flood of 1969.

The Corps of Engineers used techniques at that time that are different than the officially accepted B17B[4] (Bulletin 17B) procedures that have been available for such studies since 1982. The Corps of Engineers used a form of regional analysis and was able to statistically extend the record length of each gauge site. The exact details of the procedure are not given in the 1970 vintage Corps reports available at the time of this study. There is no record of what mean, standard deviation or skew values were used in the final determination of the recommended 100-year discharges.

The resulting 100-year peak discharges for the Ventura River at the bridge on Casitas Vistas Road and Matilija Creek below Matilija Dam recommended by the FEMA FPI report in 1997 are:

Ventura River at Ventura, CA (at bridge on Casitas Vistas Road) (drainage area = 188 sq. mi.)	Q_{100} = 68,000 ft^3/s
Ventura River below Matilija Dam (drainage area = 54.6 sq. mi.)	Q_{100} = 27,500 ft^3/s

There is a slight discrepancy between various reports as to the drainage area for the Ventura River at Ventura at the bridge on Casitas Road. This report uses 188 square miles for this location; some previous reports use 184 square miles for this location.

Discharges for other locations (not at gauge sites) and for return periods of 10-, 50-, 100-, and 500-year peak flows are also given in the 1997 FPI report. A list of locations, drainage areas and discharges for the Ventura River that were used in the 1997 FPI report is displayed in table 6 of this report.

There was no explanation given in the 1997 FIS or any Corps of Engineer FPI report of how the discharges were distributed between the two gauge locations to arrive at peak flows for the other locations on the Ventura River. A plot of the 1997 FIS recommended discharges and the associated drainage areas was made as part of this study and is displayed in Figure 2. This plot appears to show a consistent variation of the discharge with drainage area for the length of the main stem of the Ventura River.

A different approach was used in the 2000 USBR[5] report. In this report a station-log skew value of –1.0 was assigned to the Ventura River at Ventura annual peak flow data set. The remaining LPIII (Log-Pearson type III distribution) parameters (log-mean and log-standard deviation) were computed in the normal fashion. The result was a good eyeball fit to the plotted peak flows at the high end of the recorded data set. The 100-year peak flow from this approach was 69,500 ft^3/s. This value is only slightly higher than the largest recorded peak of 69,000 ft^3/s and logically supports the analysis. The same approach was used for the data below Matilija Reservoir and produced a 100-year peak flow at that site of 23,500 ft^3/s. This estimated peak is only slightly above the maximum recorded peak of 20,000 ft^3/s at this site in 1969 and again logically supports this analysis. Comparisons at the 10-, to 25-year discharge levels show that this approach produces slightly low estimates compared to the experienced data. This procedure is essentially a graphical approach to the problem and may be permitted in the absence of any other documented approach that can produce a good fit.

The resulting 100-year discharges presented in the 2000 USBR study are:

Ventura River at Ventura, CA (at bridge on Casitas Road) (drainage area = 188 sq. mi.)	Q_{100} = 69,500 ft^3/s
Ventura River below Matilija Dam (drainage area = 54.6 sq. mi.)	Q_{100} = 23,500 ft^3/s

Discharges for other return periods at these two locations are also given in the 2000 USBR report.

Current Study Frequency Analysis: As a first step to the current peak flow analysis the Ventura County Flood Control District was contacted and asked to provide updated peak flow information for the Ventura River sites listed above. Some additional peak flows for the years 1991 through 1998 were made available for Matilija Creek below the reservoir site. These additional data are included with the data in table 1.

The first attempt to create peak flows for various return periods was to run the updated peak flow data sets for the two sites with the B17B procedure, and with the prescribed regional skew value –0.3. This attempt produced nearly the same results as described in the 2000 USBR report. The fitted B17B curve significantly overestimated the largest of the recorded peak flows events in the last 68 years.

One problem that was apparent with the direct application of the B17B procedure was the presence of low outliers in the data sets for both locations. Low outliers are peak flows that are significantly below the majority of the other peaks. The threshold for determining the presence of the low outliers is briefly described in the B17B documentation. These values may have been recorded in extremely dry years without any significant rainfall generated floods. When the B17B procedure attempts to fit all of the data, including the low outliers, the resulting log-mean, log-standard deviation, and log-skew values are such that the fitted LPIII curve will become very high on the high end of the data set. B17B documentation also prescribes a method to handle such low outlier events.

The next attempts to fit the peak flow data for the two sites used in this study followed the B17B procedure for treatment of low outliers. The low outlier procedure essentially eliminates the low outlier points one at a time and fits the remaining peak flow data to an LPIII distribution with the knowledge that the record length is the same as if the low outlier point were still present. The result is changes in the parameter estimates of the fitted LPIII distribution (the log-mean becomes larger, the log standard deviation becomes smaller and log-skew becomes larger) with the ultimate result being that the high end of the fitted curve will come down once all of the low outlier points have been removed. With this procedure applied and including the prescribed regional skew of –0.3, the resulting estimates of the 100-year value were only slightly lower.

Another major problem leading to overestimation of peak flows by the LPIII distribution is the use of the regional B17B log-skew map value. The B17B documentation suggests using a regional map log-skew value to weight with the computed station log-skew value. The use of a regional map-log skew will eliminate some wide variances in estimated peak flows in nearby gauge sites with differing lengths of record. Calculated station skew values are very sensitive to high or low outliers and are also very sensitive to short record lengths (less than 100-years). As the length of the station record increases, the weight given to the regional map log-skew decreases. The result is an adopted log-skew value somewhere between the map skew and the calculated station skew for the final fitted LPIII distribution parameters. For example using the Ventura River at Ventura, CA (USGS No. 11118500) peak flow record with two outliers treated results in a station log-skew is –0.4205 and the final adopted log-skew considering the regional log-skew value is –0.3889.

These log-skew values are well below the map log-skew value of –0.3 for this region. The result of applying the map log-skew in B17B is to increase the log-skew value for the fitted LPIII distribution and at the same time adjusting the other LPIII parameters. The final estimated 100-year discharge for the analysis with the regional map skew applied is 128,000 ft^3/s. This is more than 2 times higher than the largest recorded peak flow in 68 years, 63,600 ft^3/s . Tables 3 and 4 at the end of this report display the results of the B17B computations including low-outlier treatment and the inclusion of the regional skew of –0.3. The plotting positions shown in these tables are calculated using the Weibull plotting position formula, as suggested in the B17B documentation. These plotting position values are used in later computations in this report.

For this study the LPIII distribution was also used to fit the available data sets without the use of the regional map log–skew values. If no low outliers are treated in the data sets the computed 100-year value becomes very low, near 47,000 ft^3/s for the Ventura River at Ventura, CA. This value is below two of the recorded peak flows for the last 68 years and does not seem appropriate. If two low outliers are treated and no-regional map log-skew is used, the estimated 100-year peak is 123,000 ft^3/s, which is again about twice as large as the maximum recorded peak in the last 68 years and is not appropriate.

The use of the EMA[6] (Expected Moments Algorithm) to estimate the parameters of the LPIII distribution was also tried. Many statisticians consider the EMA procedure better for handling high and low outliers in a peak flow distribution. In this case, for the Ventura River at Ventura, CA the estimated 100-year peak is about 40,000 ft^3/s and considered very low for this analysis. The result is probably highly influenced by the low outliers. One additional analysis with the EMA approach was made. In this attempt all peaks below 9,800 ft^3/s were censored. The frequency curve was analyzed using only 24 peak flows above 9,800 ft^3/s. In this analysis the parameters of the LPIII distribution are fit and the resulting flows at various return periods are based the fitted distribution. The results were again disappointing with a 100-year value of 97,000 ft^3/s, which is considered to high based on the physical record. Similar results occurred if the EMA analysis was used with censoring out of all but the top 50 percent of the events.

Figures 3 and 4 of this report display the LPIII curve fits to the data using some of the attempts to fit the peak flow records at these two sites using the complete data record and the various techniques described above. None of the approaches tried using the complete set of peak flows, with or without low outliers, can produce satisfying results over the entire 10-year to 100-year return period range.

<u>The Selected Approach</u>: Reviewing the plot of peak flows for the Ventura River at Ventura CA against their plotting positions, figure 3, it is appears that the largest seven to ten peaks do not follow the same pattern or distribution as the vast majority of the remaining flood peaks (ignoring the two very low peaks which were previously identified as low outliers). The same is true for the peak flow frequency plot for the Matilija Creek data, figure 4. This is an indication that the very largest flood peaks on this river follow a different distribution than the main body of the peak flow data. An approach often suggested for this situation is a top end fitting. In this type of analysis the peak flows and plotting positions, or the equivalent return period, are fit with a curve by a least squares procedure. The resulting regression equation is then used to determine the peak flow for the desired return periods within a reasonable degree of extrapolation. This method does fit a curve to the highest recorded peaks and it does eliminate all of the problems associated with trying to fit the complete data set with low outliers. The assumption that more than one distribution for peak flows is in effect at any one site has not been proven.

Regression equations were fit to the peak flow and return period data using the top 20, the top 10 and the top 7 peaks for each gauge record. Figures 5 and 6 of this report display the results of these calculations for the two peak flow data sets. The estimated peak flows for the 100-year and 500-year return periods for each regression are also shown on the plots. The decision as to what level of "censoring" to use does not appear to be very sensitive for these data if 10 or fewer events were selected. The difference between fitting the top 10 or the top 7 points will produce only minor differences in the estimated 100-year flood peaks. It was decided to use the top fitting technique with the top 7 peak events or about the top 10 percent of the data. It is noted that the top 7 events extend from 10- to about 70-years based on the Weibull plotting positions. This is the range of return periods of primary interest. Table 5 of this report displays the application of the regression equations using the top 7 peaks at each gauge to calculate the peak flows for the four return periods of interest. The resulting 10-, 50-, 100-, and 500-year peak flow estimates for each site are summarized in following table:

Recommended Peak Flows for the Ventural River at Existing Stream Gauge Sites (by top end fitting of peak flow data)		
Location	Ventura River Below Matilija Reservoir (location of USGS Gauge No. 11115500)	Ventura River at Ventura, CA (at Casitas Road Bridge - location of USGS Gauge No. 11118500)
Return Period (Years)	Peak Flows (ft^3/s)	
10	12,500	36,400
20	15,200	46,400
50	18,800	59,700
100	21,600	69,700
500	27,900	93,100

Comparison of Selected Peaks with 1997 FEMA FIS Study Values: Figures 7 and 8 display the highest recorded peak flow data for the two gauges of interest in this study. Also displayed on these figures are the calculated peaks for this study based on the top 7 fitting procedure described above and the peak flows for these same sites given in the 1997 FEMA FIS Study. For the Ventura River at Ventura the selected peaks of this study compare favorably being only slightly higher for the 50-, 100- and 500-year events. At the 10-year event the current study produces a higher peak flow that is in good agreement with the experienced peak flows. The FIS study estimate for a 10-year peak falls significantly below the plotting positions of the experienced floods at this level. For the gauge site below Matilija Reservoir the comparison shows that all of the peaks selected for this study are considerably lower than the 1997 FEMA FIS study. This apparent conflict was also noted with the peaks determined in the 2000 USBR study. It is also seen that the 1997 FEMA FIS peaks fall well above the actual recorded events based on the plotting positions calculated. Reasons for the high 1997 FEMA FIS peaks may be the same as described above when different methods of fitting the LPIII distribution were tried. Most of the methods produced very high peak flows for these return periods.

The estimated 10-, 20-, 50-, 100-, and 500-year peaks based on the top 7 fitting approach of this study are believed to best represent the flood possibilities near the upper end of the Ventura River based on the floods experienced to date.

Distribution of Peak Flows along the Ventura River: The Corps of Engineers in their 1971 FPI and the 1997 FIS report do provide some additional discharges at other locations along the Ventura River. There is no explanation given as to how the peak flows are distributed along the rest of the river.

For this study the peak flows are distributed to other locations along the Ventura River based on ratios of flows given in the 1997 FEMA FIS. In the 1997 FIS peak flows are given at the two gauge sites, below Matilija Reservoir (USGS No. 11115500) and for the Ventura River at Ventura at Casitas Vista Road Bridge (USGS No. 11118500). Flows for four other sites on the main stem of the Ventura River are also given. Ratios of the peak flows for the ungauged sites to the gauge sites were calculated and are displayed in table 6. The ratios for the nearest gauge site were used in this study to distribute the new peak flows to the ungauged sites listed in the 1997 FIS. Table 7 displays the results of the calculations used to distribute the peak flows along the Ventura River. For the 20-year event the 1997 FIS study does not provide peak flow values. For this study the ratios needed to distribute the 20-year peak flows in the Ventura River were estimated by taking the average of the ratios for the various locations at the 10-year and the 50-year discharge levels. These assumed ratios were then applied to the calculated 20-year peak flows at the gauge sites.

Figure 9 displays the resulting peak flows plotted as a function of the associated drainage area along with the comparable flows presented in the 1997 FIS. The USBR 2000 report also provides river mile data for various locations including the six locations from the 1997 FIS report. Figure 10 displays the peak flow information calculated for the six sites above plotted as a function of river miles. The additional locations given in the USBR 2000 report are also indicated on this plot.

The large jump in the peak flows between Baldwin Road and San Antonio Creek can be mostly explained by the large increase in drainage area below the confluence with San Antonio Creek. The Ventura River drainage area just above this confluence is 91.8 square miles. San Antonio Creek adds another 51.2 square miles at this point or approximately an additional 56 percent of the total above the confluence. It might be proper to extend the gauge record from the Matilija Creek gauge site to just above the confluence with San Antonio Creek at about river mile 8 and then jump to the higher flows based on the downstream Ventura River gauge.

Recommended Future Study: The flood peaks on the Ventura River recommended and displayed in table 7 of this study are recommended for the 10- to 500-year return period range. The primary intended use is for flood plane determinations for flows between the 10-year and 500-year return periods. If future studies require small return period flood peaks, in the 2- to 5-year range then the approach taken in this study is not appropriate. A partial duration series analysis should be undertaken and the fitting technique should be changed based on the data available for that analysis.

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Table 1

Peak flows for combined gauges at Matilija Reservoir

(Gauge 1114500 Matilija River abv. Reservoir used between 1949 and 1969)

(Gauge 1115500 Matilija River at Matilija Hot Springs used for all other years)

Gauge Number	Date	Peak (cfs)
11115500	1/19/1933	4460
11115500	12/31/1933	7000
11115500	1/15/1935	2050
11115500	2/2/1936	1430
11115500	2/14/1937	2180
11115500	3/2/1938	15900
11115500	3/9/1939	1040
11115500	2/25/1940	1320
11115500	3/4/1941	4290
11115500	12/28/1941	780
11115500	1/22/1943	15000
11115500	2/22/1944	4900
11115500	2/2/1945	2800
11115500	3/30/1946	4500
11115500	11/20/1946	3500
11115500	4/14/1948	12
11114500	3/11/1949	60
11114500	2/6/1950	155
11114500	4/28/1951	6
11114500	1/15/1952	8800
11114500	12/20/1952	235
11114500	2/13/1954	582
11114500	1/18/1955	66
11114500	1/26/1956	1040
11114500	1/13/1957	1820
11114500	4/3/1958	5440
11114500	2/16/1959	2500
11114500	1/10/1960	73
11114500	1/26/1961	42
11114500	2/9/1962	6570
11114500	2/9/1963	863
11114500	4/1/1964	344
11114500	4/9/1965	328
11114500	12/29/1965	5540
11114500	12/6/1966	5190
11114500	3/8/1968	149
11114500	1/25/1969	19600
11115500	3/2/1970	496
11115500	12/1/1970	520
11115500	12/29/1971	380
11115500	2/11/1973	6810
11115500	1/9/1974	465
11115500	3/8/1975	1820
11115500	2/10/1976	529
11115500	1/9/1977	80
11115500	3/4/1978	16500
11115500	3/28/1979	966
11115500	2/16/1980	10600
11115500	4/22/1981	323
11115500	4/1/1982	271
11115500	3/1/1983	12200
11115500	12/25/1983	1250
11115500	1/29/1985	240
11115500	2/14/1986	9730
11115500	3/4/1987	165
11115500	2/29/1988	2050
11115500	3/18/1991	5400
11115500	2/12/1992	11450
11115500	1/13/1993	5180
11115500	3/10/1995	10360
11115500	2/20/1996	570
11115500	2/23/1998	14000

Table 2
Peak flows for Ventura River Nr. Ventura, CA

Gauge Number	Date	Peak (cfs)
11118500	1/19/1933	13000
11118500	12/31/1933	23000
11118500	1/5/1935	6010
11118500	2/12/1936	3330
11118500	2/14/1937	13900
11118500	3/2/1938	39200
11118500	3/9/1939	2840
11118500	2/25/1940	4330
11118500	3/1/1941	15200
11118500	12/28/1941	1190
11118500	1/22/1943	35000
11118500	2/22/1944	20000
11118500	2/2/1945	17000
11118500	3/30/1946	8000
11118500	11/20/1946	2400
11118500	3/24/1948	2.4
11118500	3/11/1949	35
11118500	2/6/1950	2000
11118500	3/1/1951	0.3
11118500	1/15/1952	29500
11118500	12/20/1952	1040
11118500	2/13/1954	3030
11118500	1/18/1955	203
11118500	1/26/1956	4050
11118500	1/13/1957	936
11118500	4/3/1958	18700
11118500	2/16/1959	3220
11118500	2/1/1960	966
11118500	11/6/1960	308
11118500	2/10/1962	12400
11118500	2/9/1963	1060
11118500	11/20/1963	132
11118500	4/9/1965	744
11118500	11/24/1965	11200
11118500	12/6/1966	9900
11118500	3/8/1968	665
11118500	1/25/1969	58000
11118500	3/4/1970	1930
11118500	12/21/1970	3120
11118500	12/27/1971	2090
11118500	2/11/1973	15700
11118500	1/7/1974	2540
11118500	3/8/1975	5150
11118500	9/29/1976	1990
11118500	1/2/1977	856
11118500	2/10/1978	63600
11118500	3/28/1979	4280
11118500	2/16/1980	37900
11118500	3/1/1981	1210
11118500	4/1/1982	834
11118500	3/1/1983	27000
11118500	12/25/1983	1500
11118500	12/19/1984	412
11118500	2/14/1986	22100
11118500	3/6/1987	174
11118500	2/29/1988	4000
11118500	12/21/1988	236
11118500	2/17/1990	516
11118500	3/19/1991	11300
11118500	2/12/1992	45800
11118500	1/18/1993	12500
11118500	2/20/1994	1820
11118500	1/10/1995	43700
11118500	2/20/1996	3660
11118500	1/26/1997	4960
11118500	2/23/1998	38800
11118500	1/31/1999	106
11118500	2/23/2000	3280

Table 3

(Results of this LPIII analysis are not the final results recommended in the study)

Ventura River at Ventura, CA 2 low outliers, Reg SK -0.3

Mean of Logs	Std.Dev	Data Skew	Reg.Skew	Final Skew
3.5295	0.7751	-0.4205	-0.3000	-0.3899

RANK	PlotPos	YEAR	Q	EXCEED.	FREQ.Q	LOW	HIGH
1	0.01449	1978	63600.0	0.99000	32	14	63
2	0.02899	1969	58000.0	0.98000	60	28	110
3	0.04348	1992	45800.0	0.97500	75	36	133
4	0.05797	1995	43700.0	0.96000	119	61	203
5	0.07246	1938	39200.0	0.95000	150	79	250
6	0.08696	1998	38800.0	0.90000	323	188	505
7	0.10145	1980	37900.0	0.80000	787	503	1160
8	0.11594	1943	35000.0	0.70000	1453	977	2088
9	0.13043	1952	29500.0	0.60000	2407	1663	3437
10	0.14493	1983	27000.0	0.57040	2765	1919	3950
11	0.15942	1933	23000.0	0.50000	3799	2657	5459
12	0.17391	1986	22100.0	0.42960	5183	3630	7531
13	0.18841	1944	20000.0	0.40000	5907	4133	8639
14	0.20290	1958	18700.0	0.30000	9320	6453	14034
15	0.21739	1945	17000.0	0.20000	15560	10526	24494
16	0.23188	1973	15700.0	0.10000	30532	19754	51669
17	0.24638	1941	15200.0	0.05000	51616	31997	93161
18	0.26087	1937	13900.0	0.04000	59831	36611	110068
19	0.27536	1933	13000.0	0.02500	79574	47440	152030
20	0.28986	1993	12500.0	0.02000	90154	53118	175181
21	0.30435	1962	12400.0	0.01000	128274	73031	261724
22	0.31884	1991	11300.0	0.00500	174822	96476	372724
23	0.33333	1965	11200.0	0.00200	250253	133062	562006
24	0.34783	1966	9900.0				
25	0.36232	1946	8000.0				
26	0.37681	1935	6010.0				
27	0.39130	1975	5150.0				
28	0.40580	1997	4960.0				
29	0.42029	1940	4330.0				
30	0.43478	1979	4280.0				
31	0.44928	1956	4050.0				
32	0.46377	1988	4000.0				
33	0.47826	1996	3660.0				
34	0.49275	1936	3330.0				
35	0.50725	2000	3280.0				
36	0.52174	1959	3220.0				
37	0.53623	1970	3120.0				
38	0.55072	1954	3030.0				
39	0.56522	1939	2840.0				
40	0.57971	1974	2540.0				
41	0.59420	1946	2400.0				
42	0.60870	1971	2090.0				
43	0.62319	1950	2000.0				
44	0.63768	1976	1990.0				
45	0.65217	1970	1930.0				
46	0.66667	1994	1820.0				
47	0.68116	1983	1500.0				
48	0.69565	1981	1210.0				

49	0.71014	1941	1190.0
50	0.72464	1963	1060.0
51	0.73913	1952	1040.0
52	0.75362	1960	966.0
53	0.76812	1957	936.0
54	0.78261	1977	856.0
55	0.79710	1982	834.0
56	0.81159	1965	744.0
57	0.82609	1968	665.0
58	0.84058	1990	516.0
59	0.85507	1984	412.0
60	0.86957	1960	308.0
61	0.88406	1988	236.0
62	0.89855	1955	203.0
63	0.91304	1987	174.0
64	0.92754	1963	132.0
65	0.94203	1999	106.0
66	0.95652	1949	35.0
67	0.97101	1948	2.4
68	0.98551	1951	0.3

Table 4

(Results of this LPIII analysis are not the final results recommended in the study)

Matilija Dam Peak Inflows with Regional Skew = -0.3

Two low outliers detected and treated

Mean of Logs	Std.Dev	Data Skew	Reg.Skew	Final Skew
3.0981	0.8185	-0.6584	-0.3000	-0.5506

RANK	PlotPos	YEAR	Q	EXCEED.	FREQ.Q	LOW	HIGH
1	0.01587	1969	19600.0	0.99000	7.4	2.8	15.9
2	0.03175	1978	16500.0	0.98000	15	6	30
3	0.04762	1938	15900.0	0.97500	20	8	38
4	0.06349	1943	15000.0	0.96000	33	16	61
5	0.07937	1998	14000.0	0.95000	43	21	77
6	0.09524	1983	12200.0	0.90000	103	56	169
7	0.11111	1992	11450.0	0.80000	276	168	422
8	0.12698	1980	10600.0	0.70000	536	346	799
9	0.14286	1995	10360.0	0.60000	920	612	1364
10	0.15873	1986	9730.0	0.57040	1065	712	1582
11	0.17460	1952	8800.0	0.50000	1489	1003	2227
12	0.19048	1933	7000.0	0.42960	2058	1388	3121
13	0.20635	1973	6810.0	0.40000	2357	1587	3601
14	0.22222	1962	6570.0	0.30000	3761	2503	5938
15	0.23810	1965	5540.0	0.20000	6300	4093	10436
16	0.25397	1958	5440.0	0.10000	12214	7586	21807
17	0.26984	1991	5400.0	0.05000	20163	12011	38392
18	0.28571	1966	5190.0	0.04000	23158	13626	44928
19	0.30159	1993	5180.0	0.02500	30160	17315	60687
20	0.31746	1944	4900.0	0.02000	33809	19199	69137
21	0.33333	1946	4500.0	0.01000	46459	25567	99456
22	0.34921	1933	4460.0	0.00500	61027	32661	136010
23	0.36508	1941	4290.0	0.00200	83013	43013	193780
24	0.38095	1946	3500.0				
25	0.39683	1945	2800.0				
26	0.41270	1959	2500.0				
27	0.42857	1937	2180.0				
28	0.44444	1988	2050.0				
29	0.46032	1935	2050.0				
30	0.47619	1957	1820.0				
31	0.49206	1975	1820.0				
32	0.50794	1936	1430.0				
33	0.52381	1940	1320.0				
34	0.53968	1983	1250.0				
35	0.55556	1956	1040.0				
36	0.57143	1939	1040.0				
37	0.58730	1979	966.0				
38	0.60317	1963	863.0				
39	0.61905	1941	780.0				
40	0.63492	1954	582.0				
41	0.65079	1996	570.0				
42	0.66667	1976	529.0				
43	0.68254	1970	520.0				
44	0.69841	1970	496.0				
45	0.71429	1974	465.0				
46	0.73016	1971	380.0				
47	0.74603	1964	344.0				
48	0.76190	1965	328.0				

49	0.77778	1981	323.0
50	0.79365	1982	271.0
51	0.80952	1985	240.0
52	0.82540	1952	235.0
53	0.84127	1987	165.0
54	0.85714	1950	155.0
55	0.87302	1968	149.0
56	0.88889	1977	80.0
57	0.90476	1960	73.0
58	0.92063	1955	66.0
59	0.93651	1949	60.0
60	0.95238	1961	42.0
61	0.96825	1948	12.0
62	0.98413	1951	6.0

Table 5

Regression Equation Estimates for 10-, 50-, 100-, and 500-Year Peaks
from fitting of top 7 peaks with plotting position return periods (Figures 5 and 6)

Based on peak flow data available through the year 2000 and Weibull plotting positions
for Ventura River at Ventura - at Casitas Vista Road Bridge (USGS No. 111185000)

Return Period (years)	Log of Return Period	Coefficient of Log of Return Period	Coeff times Log of Return Period	Regression Constant	Final Peak Discharge by Regression (cfs)	Final Discharge to three significant Digits (cfs)
10	2.303	14494	33373.7	2997.4	36371	36400
20	2.996	14494	43420.1	2997.4	46418	46400
50	3.912	14494	56700.9	2997.4	59698	59700
100	4.605	14494	66747.3	2997.4	69745	69700
500	6.215	14494	90074.5	2997.4	93072	93100

Based on peak flow data available through the year 2000 and Weibull plotting positions
for Matilija Creek below Matilija Reservoir above N. Fork Matilija Creek (USGS No. 11115500)

Return Period (years)	Log of Return Period	Coefficient of Log of Return Period	Coeff times Log of Return Period	Regression Constant	Final Peak Discharge by Regression (cfs)	Final Discharge to three significant Digits (cfs)
10	2.303	3937.3	9066.0	3432.5	12498	12500
20	2.996	3937.3	11795.1	3432.5	15228	15200
50	3.912	3937.3	15402.8	3432.5	18835	18800
100	4.605	3937.3	18131.9	3432.5	21564	21600
500	6.215	3937.3	24468.8	3432.5	27901	27900

Table 6

Ventura River FEMA Study Peak Discharge Values
From FEMA Report 1997 which references Corps of Engineers FPI Report dated 1970

Location From 1997 FEMA FIS Study	Drainage Area (sq. mi.)	River Station (miles) (USBR 2000)	Return Period (years) Recommended Peak Flows for 1997 FIS 10 (cfs)	20 (cfs)	50 (cfs)	100 (cfs)	500 (cfs)
Upstream of Matilija Creek Conf with N. Frk Matilija Crk	54.3	15.6	12500	NA	23500	27500	36500
Downstream of Confluence with N. Fork Matilija Creek	70.4	14.0	15000	NA	30000	34500	46000
At Baldwin Road	81	11.1	16000	NA	31000	36000	48000
At Casitas Springs	143	7.8	29000	NA	55000	65000	86000
at Casitas Vista Road Bridge (Gauge Location)	188	5.9	30000	NA	58000	68000	90000
At Shell Chemical Plant	222	0.0	34000	NA	66000	77000	102000

Ratios of peak flows to Ventura River At Ventura Gauge (at Casitas Vista Road Bridge)

Location From 1997 FEMA FIS Study	Drainage Area (sq. mi.)	River Station (miles) (USBR 2000)	Return Period (years) Ratios of Peak Flows for 1997 FIS 10	20 Avg 10 & 50	50	100	500
Upstream of Matilija Creek Conf with N. Frk Matilija Crk	54.3	15.6	0.4167	0.4109	0.4052	0.4044	0.4056
Downstream of Confluence with N. Fork Matilija Creek	70.4	14.0	0.5000	0.5086	0.5172	0.5074	0.5111
At Baldwin Road	81	11.1	0.5333	0.5339	0.5345	0.5294	0.5333
At Casitas Springs	143	7.8	0.9667	0.9575	0.9483	0.9559	0.9556
at Casitas Vista Road Bridge (Gauge Location)	188	5.9	1.0000	1.0000	1.0000	1.0000	1.0000
At Shell Chemical Plant	222	0.0	1.1333	1.1356	1.1379	1.1324	1.1333

Ratio of Peak flows to Matilija Creek Gauge at Conf with N. Fork Matilija Creek

Location From 1997 FEMA FIS Study	Drainage Area (sq. mi.)	River Station (miles) (USBR 2000)	Return Period (years) Ratios of Peak Flows for 1997 FIS 10	20 Avg 10 & 50	50	100	500
Upstream of Matilija Creek Conf with N. Frk Matilija Crk	54.3	15.6	1.0000	1.0000	1.0000	1.0000	1.0000
Downstream of Confluence with N. Fork Matilija Creek	70.4	14.0	1.2000	1.2383	1.2766	1.2545	1.2603
At Baldwin Road	81	11.1	1.2800	1.2996	1.3191	1.3091	1.3151
At Casitas Springs	143	7.8	2.3200	2.3302	2.3404	2.3636	2.3562
at Casitas Vista Road Bridge (Gauge Location)	188	5.9	2.4000	2.4340	2.4681	2.4727	2.4658
At Shell Chemical Plant	222	0.0	2.7200	2.7643	2.8085	2.8000	2.7945

Table 7

Peak flows distributed along Ventura River by River Mile

	Drainage Area (sq. mi.)	River Station (miles) (USBR 2000)	Gauge Location in1997 FIS for ratio in this Study	Peak Flows in cfs (rounded to three significant digits) Return Period (years)				
				10	20	50	100	500
Upstream of Matilija Creek Conf with N. Frk Matilija Crk	54.3	15.6	Matilija	12500	15200	18800	21600	27900
Downstream of Confluence with N. Fork Matilija Creek	70.4	14.0	Matilija	15000	18800	24000	27100	35200
At Baldwin Road	81	11.1	Matilija	16000	19800	24800	28300	36700
At Casitas Springs	143	7.8	Ventura	35200	44400	56600	66600	89000
at Casitas Vista Road Bridge (Gauge Location)	188	5.9	Ventura	36400	46400	59700	69700	93100
At Shell Chemical Plant	222	0.0	Ventura	41300	52700	67900	78900	105500

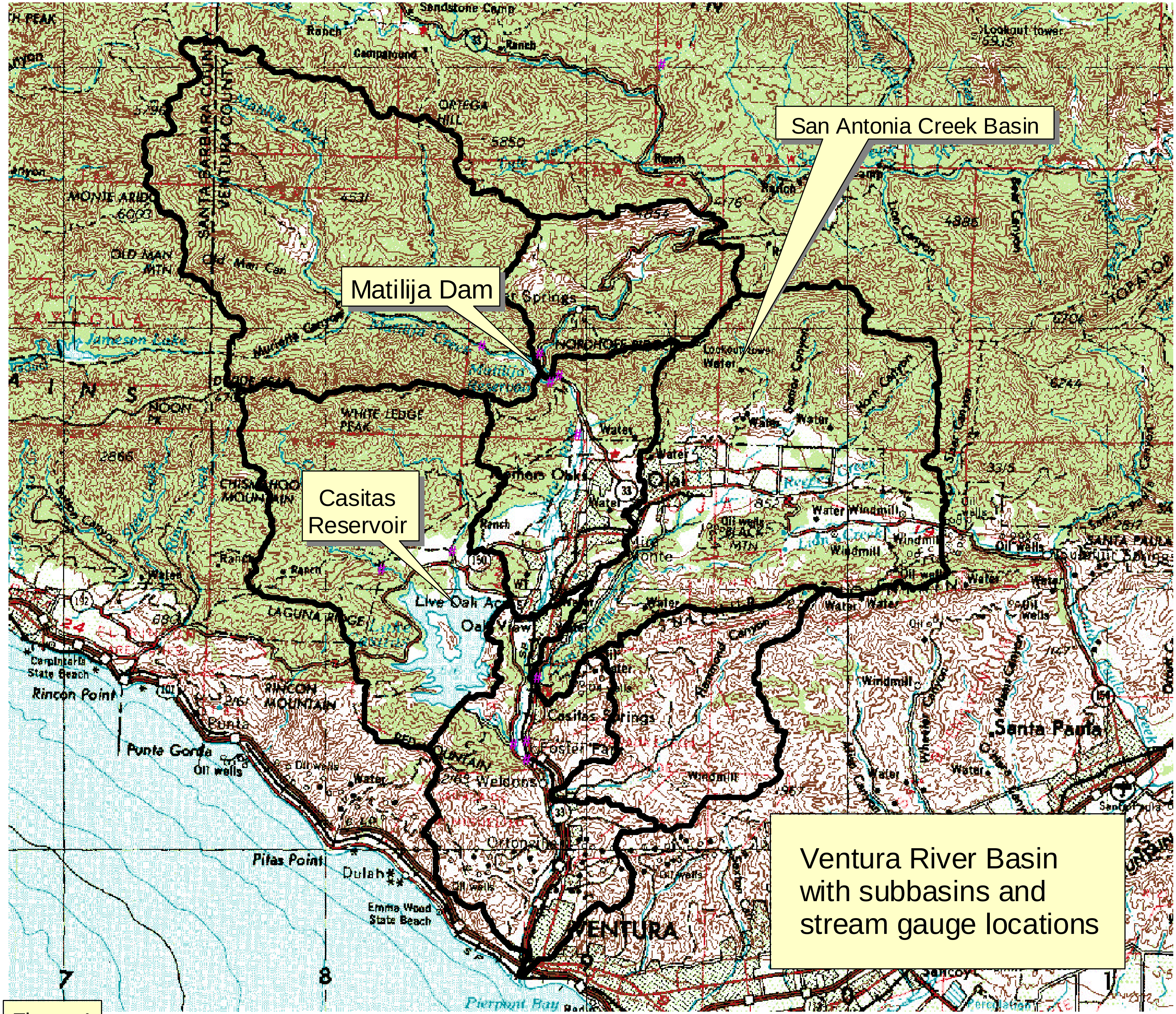

San Antonia Creek Basin
Matilija Dam
Casitas Reservoir
Ventura River Basin with subbasins and stream gauge locations

Figure 1

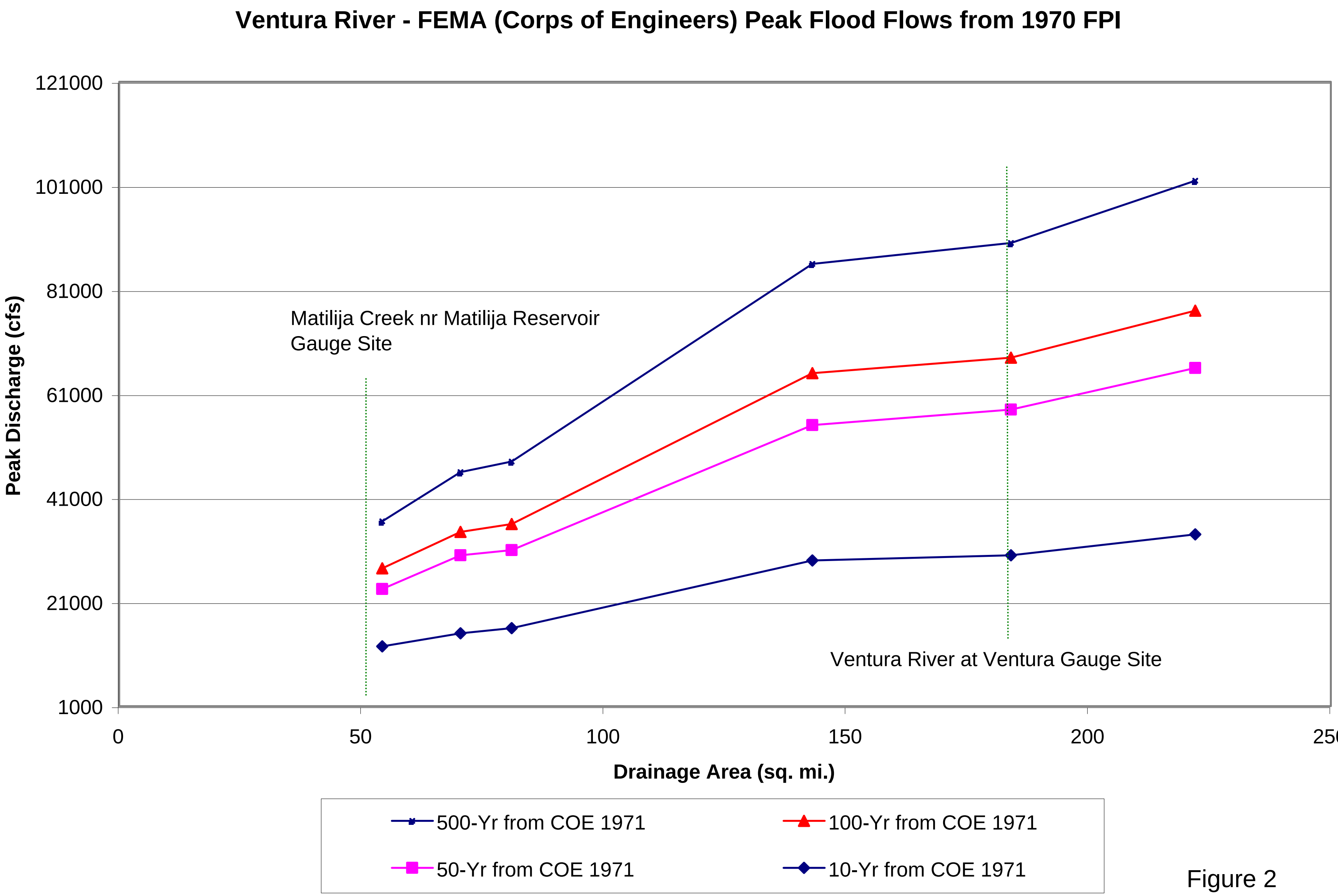
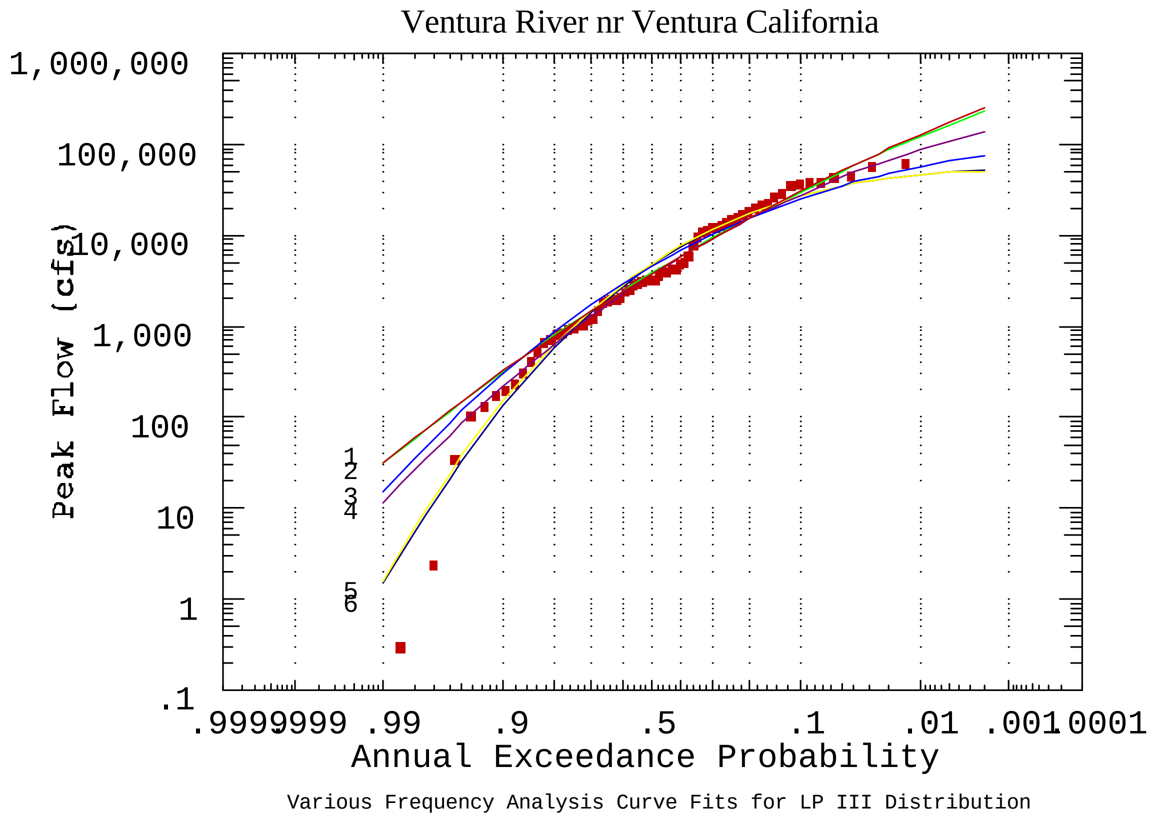
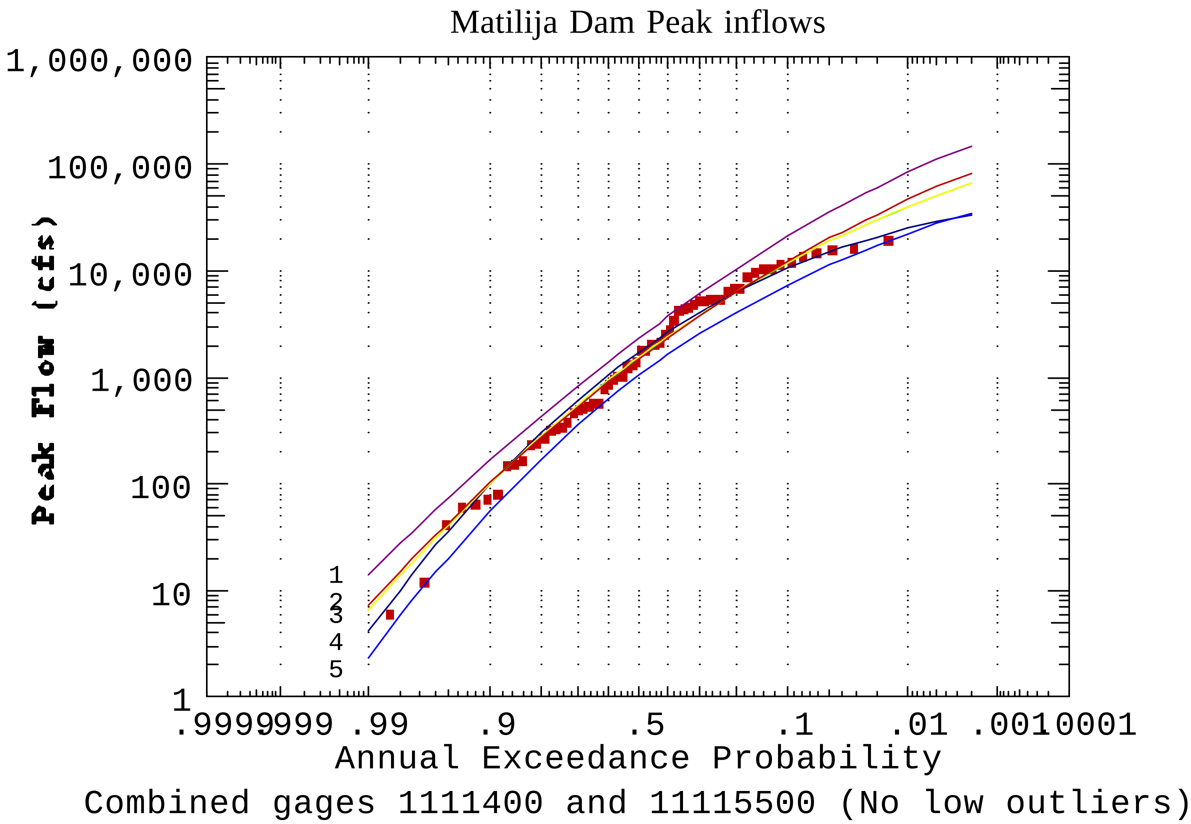

Ventura River - FEMA (Corps of Engineers) Peak Flood Flows from 1970 FPI
Peak Discharge (cfs)
121000
101000
81000
61000
41000
21000
1000
0
50
100
150
200
250
Drainage Area (sq. mi.)
Matilija Creek nr Matilija Reservoir Gauge Site
Ventura River at Ventura Gauge Site
500-Yr from COE 1971
100-Yr from COE 1971
50-Yr from COE 1971
10-Yr from COE 1971

Figure 2



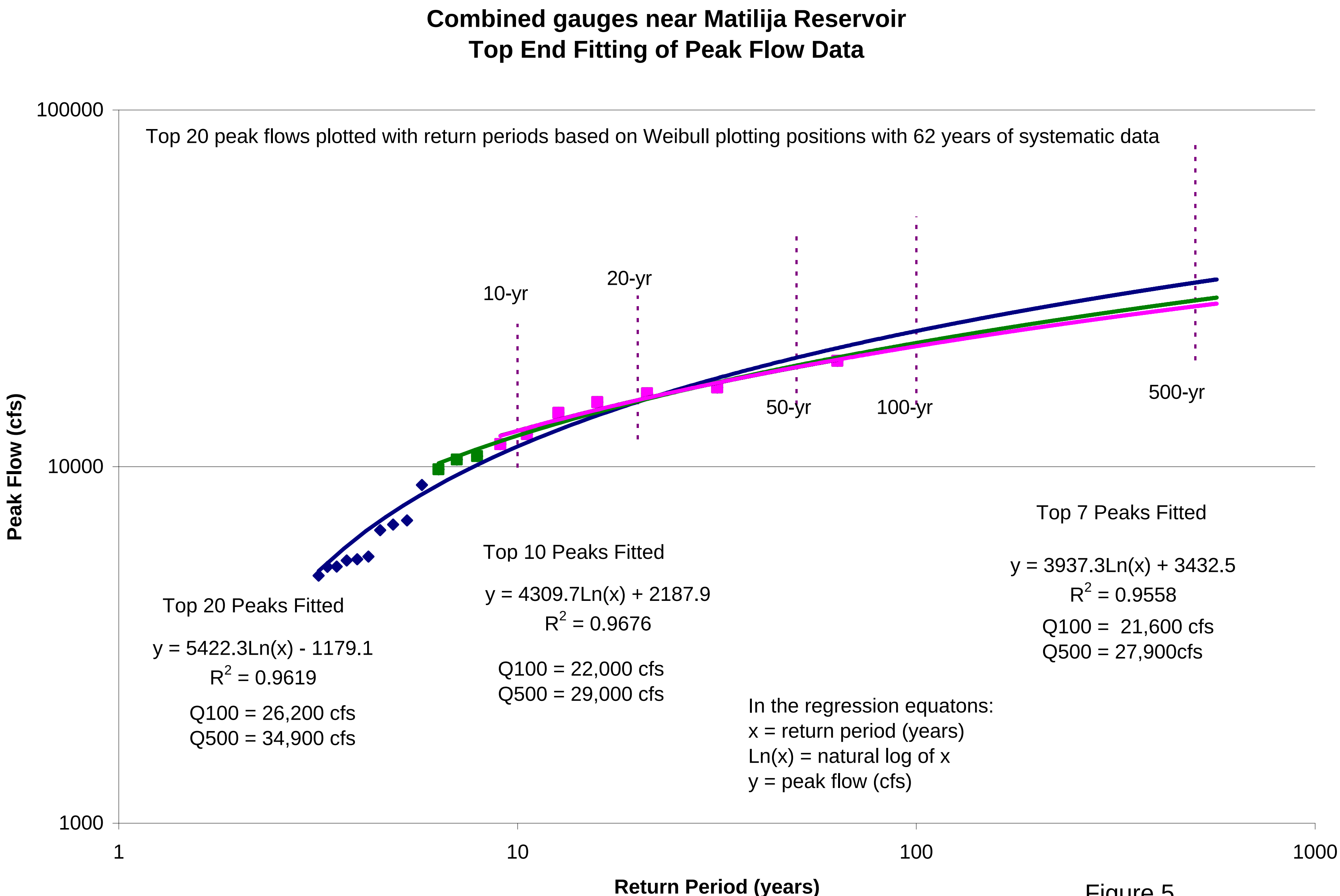
Curve 1 Log-Pearson III (with 2 outliers treated), station log-skew = -0.4205
Curve 2 EMA Log-Pearson III Fit (no outliers treated) station log-skew = -0.7556
Curve 3 Log-Pearson III (Bulletin 17B Procedure with regional Skew = -0.3) log-skew=-0.39
Curve 4 Log-Pearson III (Fixed Parameters) Fixed log-skew = -1.0000
Curve 5 Log-Pearson III (with one low outlier treated) station log-skew = -1.4999
Curve 6 Log-Pearson III (with no outliers treated) station log- slew = -1.5348

Figure 3



Curve 1 - Upper 95% confidence for standard LPIII Distribution skew = -0.6584, n = 62 years
Curve 2 - Standard Bulletin 17B (with regional skew set at -0.3) , 2 low outliers treated
log-mean = 3.0981, log-std = 0.8185, log-skew = -0.5506, n = 62 years
Curves 3- Standard LPIII and EMA LPIII (Exactly the same with no outliers detected)
log-mean = 3.0981, log-std = 0.8185, log-skew = -0.6584, n = 62 years
Curve 4 - Log-Pearson with fixed log-skew = -1.0000 (other parameters same as curve 2)
Curve 5 - Lower 95% confidence for standard LPIII Distribution skew = -0.6584, n = 62 years

Figure 4



Combined gauges near Matilija Reservoir
Top End Fitting of Peak Flow Data
Top 20 peak flows plotted with return periods based on Weibull plotting positions with 62 years of systematic data
100000
10000
1000
Peak Flow (cfs)
10-yr
20-yr
50-yr
100-yr
500-yr
Top 20 Peaks Fitted
y = 5422.3Ln(x) - 1179.1
R^2 = 0.9619
Q100 = 26,200 cfs
Q500 = 34,900 cfs
Top 10 Peaks Fitted
y = 4309.7Ln(x) + 2187.9
R^2 = 0.9676
Q100 = 22,000 cfs
Q500 = 29,000 cfs
Top 7 Peaks Fitted
y = 3937.3Ln(x) + 3432.5
R^2 = 0.9558
Q100 = 21,600 cfs
Q500 = 27,900cfs
In the regression equatons:
x = return period (years)
Ln(x) = natural log of x
y = peak flow (cfs)
1
10
100
1000
Return Period (years)

Figure 5

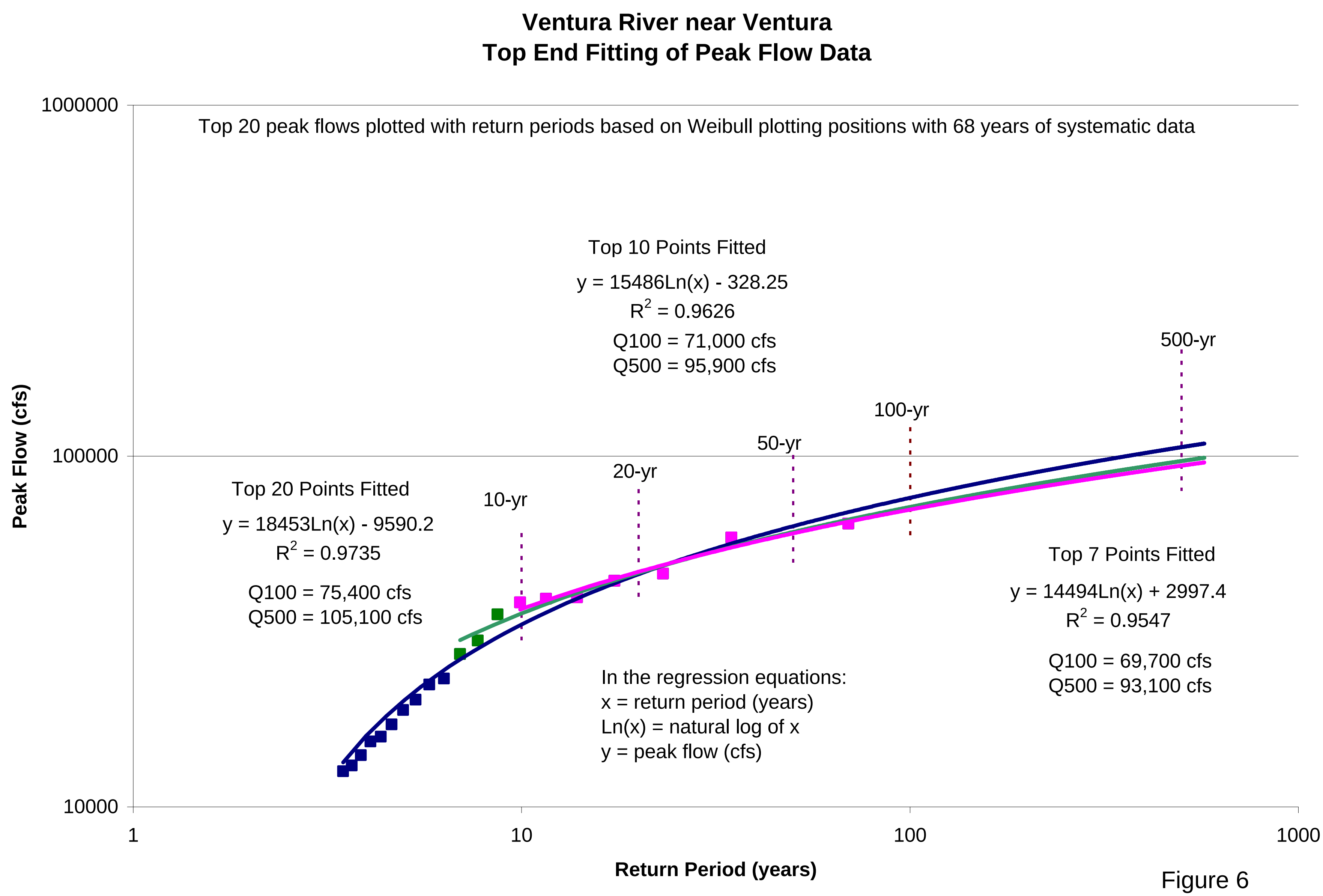

Ventura River near Ventura
Top End Fitting of Peak Flow Data
Top 20 peak flows plotted with return periods based on Weibull plotting positions with 68 years of systematic data
1000000
100000
10000
Peak Flow (cfs)
Top 10 Points Fitted
y = 15486Ln(x) - 328.25
R^2 = 0.9626
Q100 = 71,000 cfs
Q500 = 95,900 cfs
500-yr
100-yr
50-yr
20-yr
10-yr
Top 20 Points Fitted
y = 18453Ln(x) - 9590.2
R^2 = 0.9735
Q100 = 75,400 cfs
Q500 = 105,100 cfs
Top 7 Points Fitted
y = 14494Ln(x) + 2997.4
R^2 = 0.9547
Q100 = 69,700 cfs
Q500 = 93,100 cfs
In the regression equations:
x = return period (years)
Ln(x) = natural log of x
y = peak flow (cfs)
1
10
100
1000
Return Period (years)

Figure 6

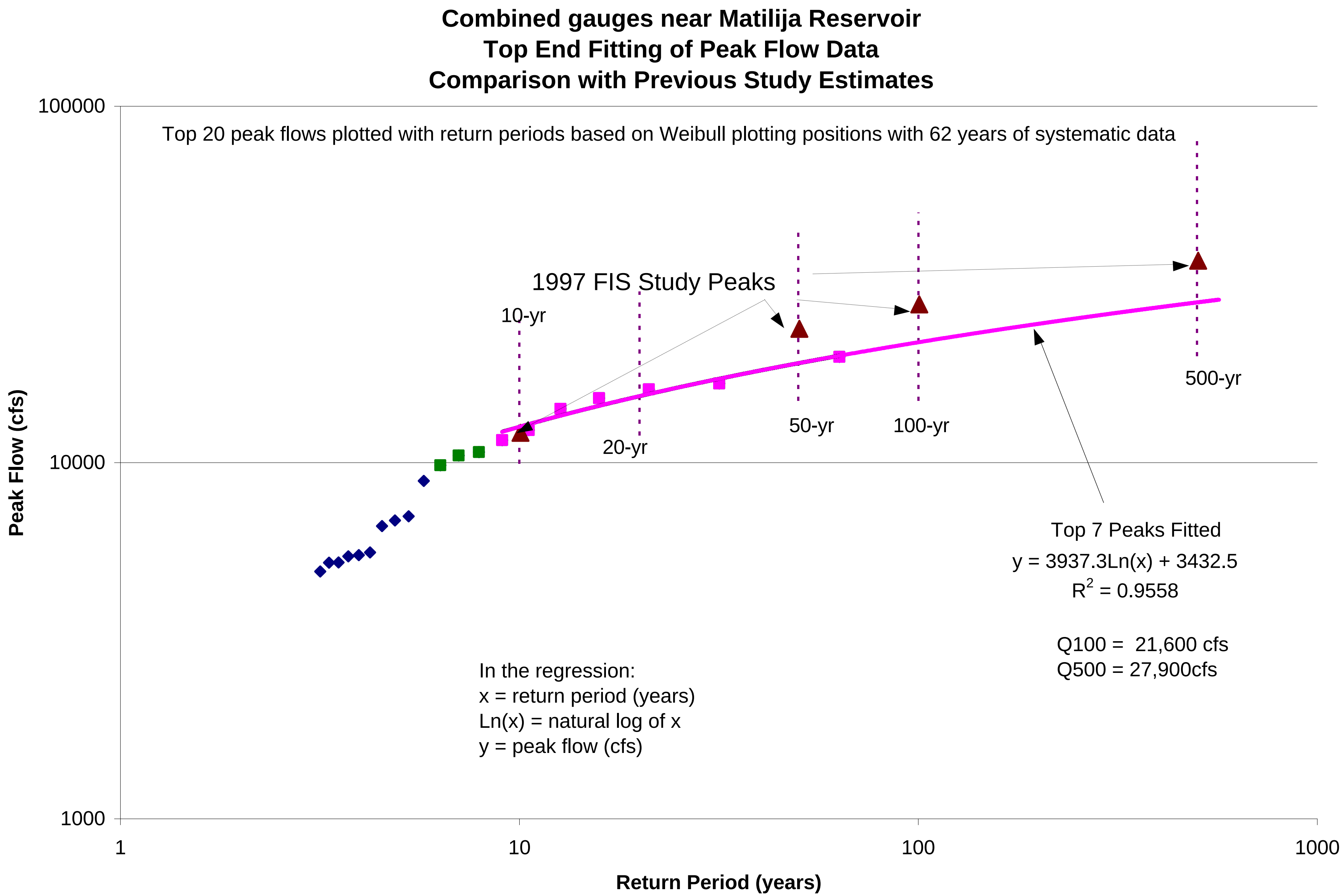
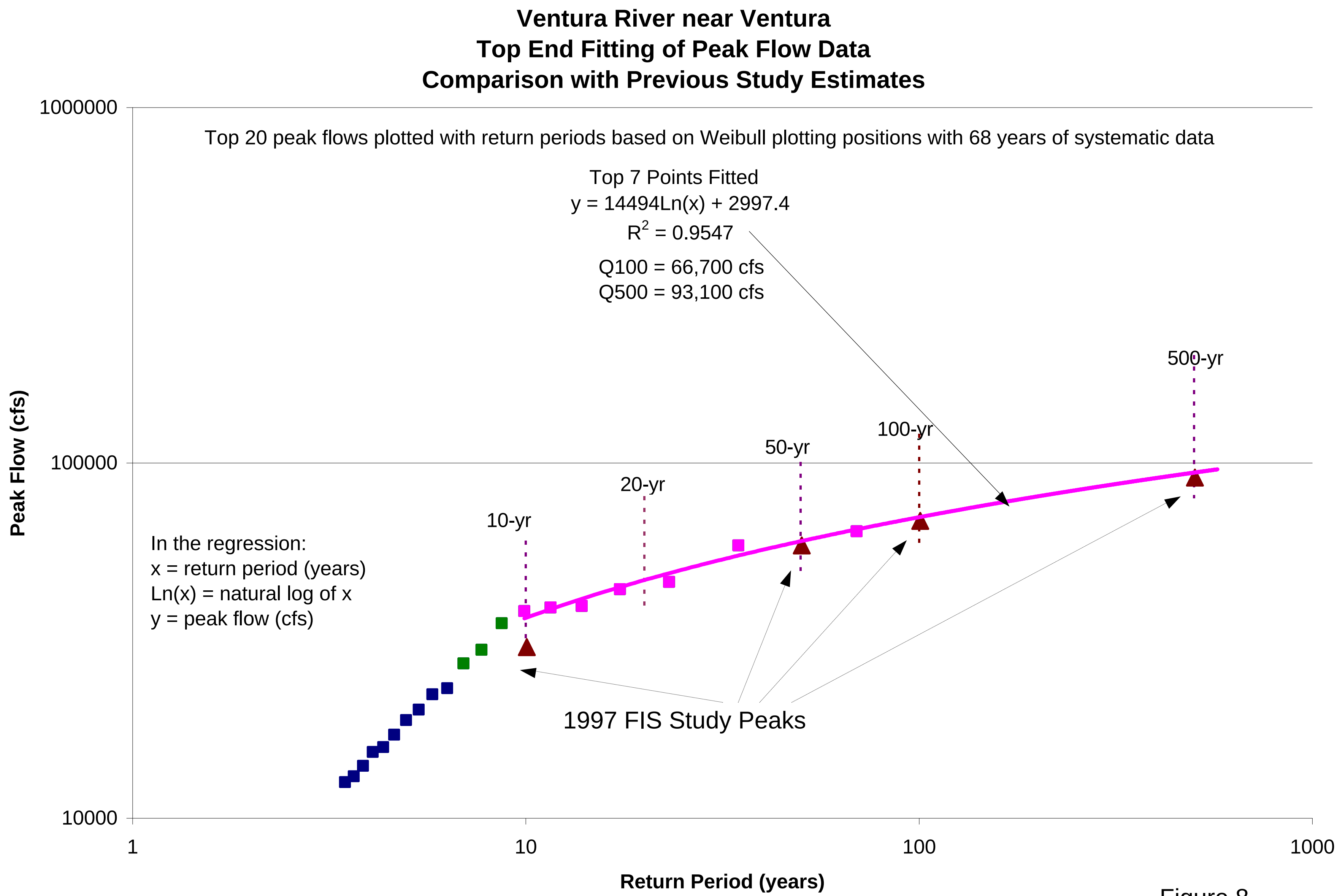
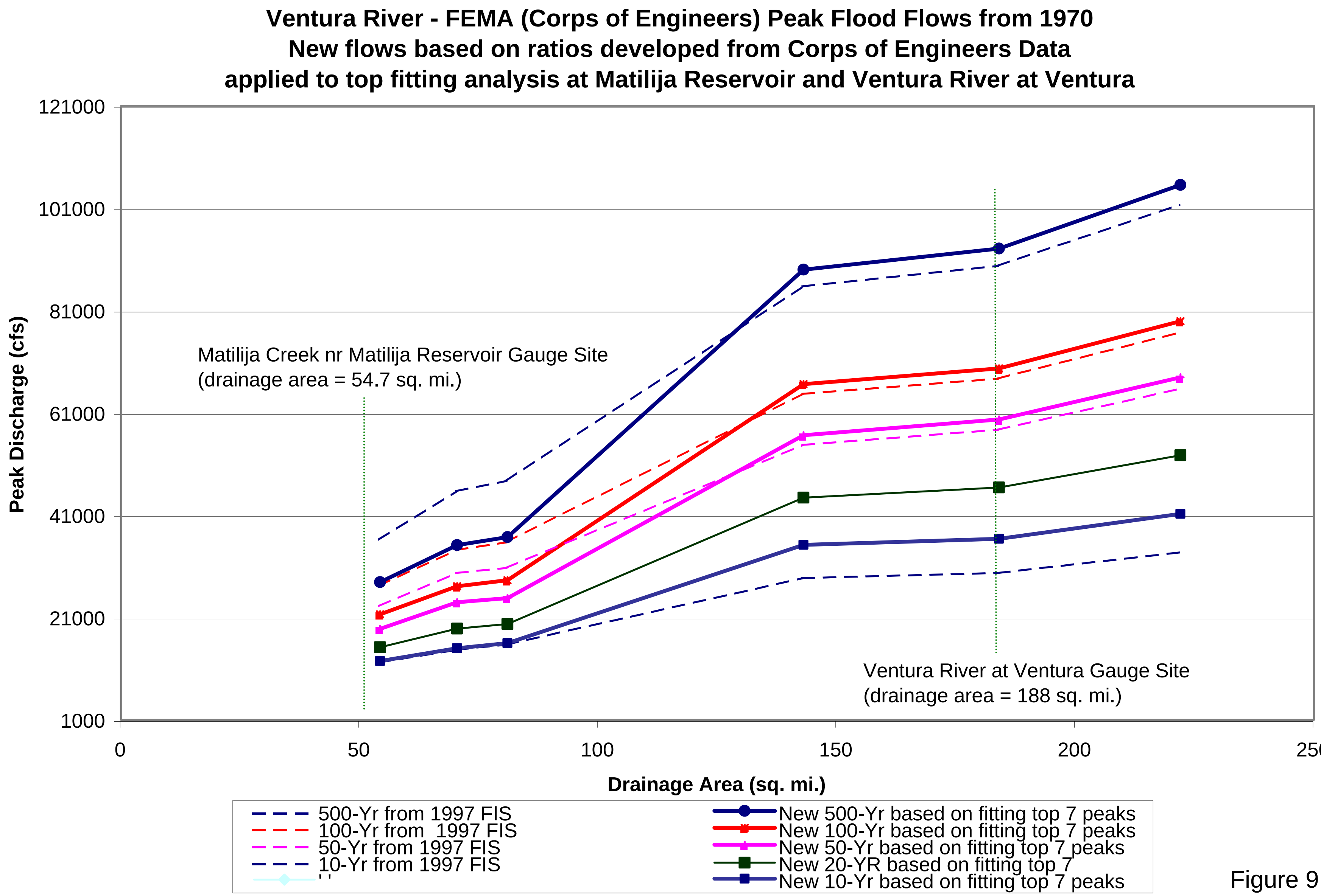

Combined gauges near Matilija Reservoir
Top End Fitting of Peak Flow Data
Comparison with Previous Study Estimates
100000
10000
1000
Peak Flow (cfs)
Top 20 peak flows plotted with return periods based on Weibull plotting positions with 62 years of systematic data
1997 FIS Study Peaks
10-yr
20-yr
50-yr
100-yr
500-yr
Top 7 Peaks Fitted
y = 3937.3Ln(x) + 3432.5
R2 = 0.9558
Q100 = 21,600 cfs
Q500 = 27,900cfs
In the regression:
x = return period (years)
Ln(x) = natural log of x
y = peak flow (cfs)
1
10
100
1000
Return Period (years)

Figure 7



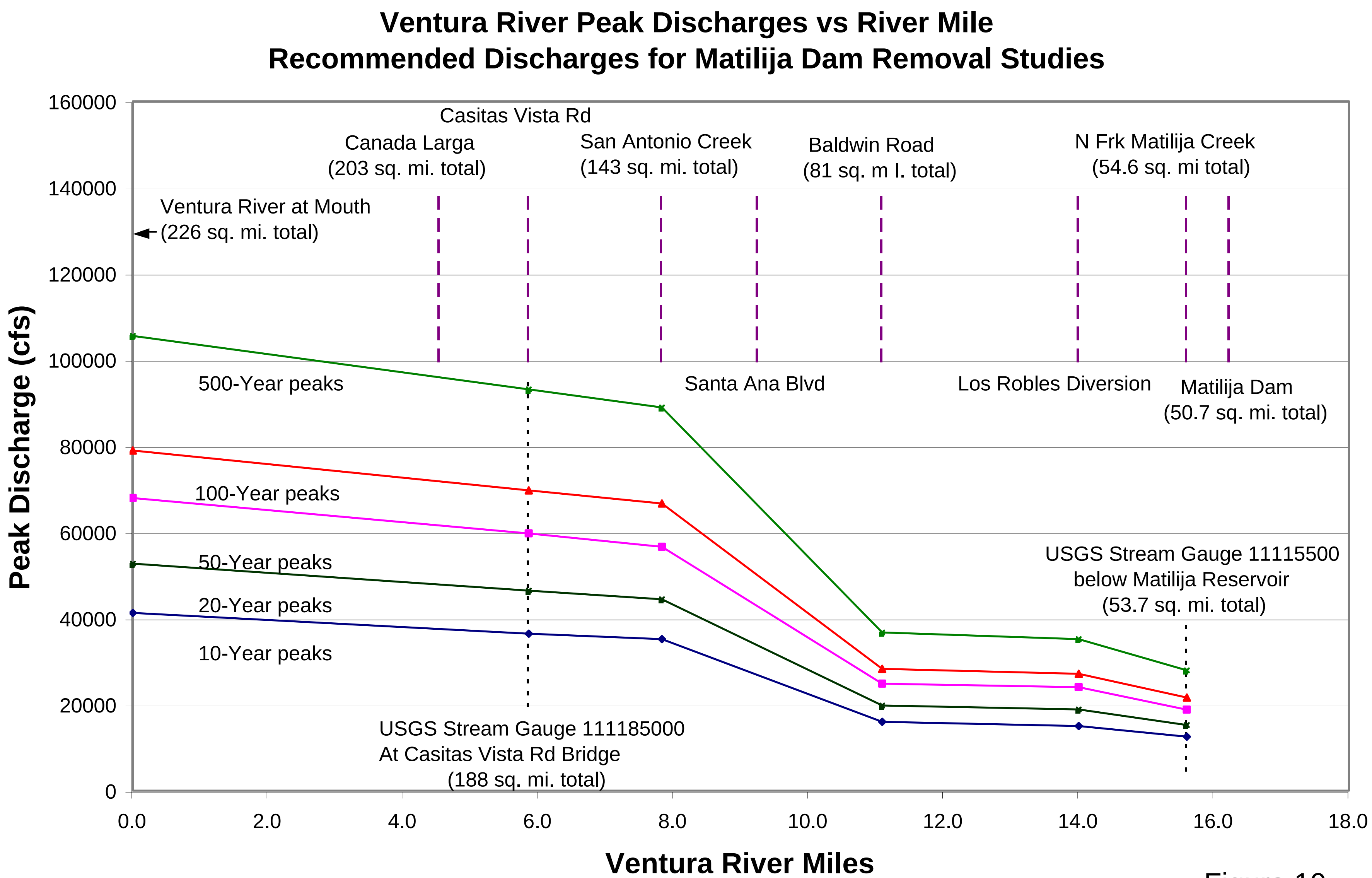
Ventura River near Ventura
Top End Fitting of Peak Flow Data
Comparison with Previous Study Estimates
Top 20 peak flows plotted with return periods based on Weibull plotting positions with 68 years of systematic data
Top 7 Points Fitted
y = 14494Ln(x) + 2997.4
R2 = 0.9547
Q100 = 66,700 cfs
Q500 = 93,100 cfs
In the regression:
x = return period (years)
Ln(x) = natural log of x
y = peak flow (cfs)
10-yr
20-yr
50-yr
100-yr
500-yr
1997 FIS Study Peaks
Peak Flow (cfs)
1000000
100000
10000
1
10
100
1000
Return Period (years)

Figure 8



Ventura River - FEMA (Corps of Engineers) Peak Flood Flows from 1970
New flows based on ratios developed from Corps of Engineers Data
applied to top fitting analysis at Matilija Reservoir and Ventura River at Ventura
121000
101000
81000
61000
41000
21000
1000
Peak Discharge (cfs)
0
50
100
150
200
250
Drainage Area (sq. mi.)
Matilija Creek nr Matilija Reservoir Gauge Site
(drainage area = 54.7 sq. mi.)
Ventura River at Ventura Gauge Site
(drainage area = 188 sq. mi.)
500-Yr from 1997 FIS
100-Yr from 1997 FIS
50-Yr from 1997 FIS
10-Yr from 1997 FIS
New 500-Yr based on fitting top 7 peaks
New 100-Yr based on fitting top 7 peaks
New 50-Yr based on fitting top 7 peaks
New 20-YR based on fitting top 7
New 10-Yr based on fitting top 7 peaks

Figure 9



Ventura River Peak Discharges vs River Mile
Recommended Discharges for Matilija Dam Removal Studies
Peak Discharge (cfs)
Ventura River Miles
160000
140000
120000
100000
80000
60000
40000
20000
0
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
Casitas Vista Rd
Canada Larga
(203 sq. mi. total)
San Antonio Creek
(143 sq. mi. total)
Baldwin Road
(81 sq. m I. total)
N Frk Matilija Creek
(54.6 sq. mi total)
Ventura River at Mouth
(226 sq. mi. total)
500-Year peaks
Santa Ana Blvd
Los Robles Diversion
Matilija Dam
(50.7 sq. mi. total)
100-Year peaks
50-Year peaks
20-Year peaks
10-Year peaks
USGS Stream Gauge 11115500
below Matilija Reservoir
(53.7 sq. mi. total)
USGS Stream Gauge 111185000
At Casitas Vista Rd Bridge
(188 sq. mi. total)

Figure 10

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14. Exhibit B. Flow Duration Curves by Month

Matilija Creek ab Reservoir at Matilija Hot Springs

USGS Gauge Number 11114500

Daily Flows from 1949 to 1969

Drainage Area = 15.6 square miles

Gauge Datum = 1.160.20 feet

Percent of Time Flow is Below This Value	Flow Values (cfs) January	February	March	April	May	June	July	August	September	October	November	December
0	1.7	2.3	2.6	1.7	1.4	0.6	0.5	0.4	0.4	0.3	0.6	1.1
10	3.4	4.0	4.0	4.2	3.3	1.9	1.1	0.8	0.6	0.61	0.9	1.6
20	4.0	5.5	6.2	5.8	4.0	2.6	1.4	0.9	0.8	0.8	1.2	2.0
30	4.5	7.6	8.8	7.3	5.8	3.0	1.7	1.1	0.8	0.9	1.5	2.7
40	7.0	9.7	10	8.7	7.2	4.0	2.1	1.4	1.0	1.3	1.7	3.8
50	8.0	14	12	12	8.1	5.0	2.5	1.9	1.6	1.5	2.1	4.2
60	9.7	17	17	15	11	6.0	3.0	2.1	1.8	1.8	2.6	4.5
70	11	34	27	25	14	7.2	3.8	2.4	2.2	2.1	4.0	7.5
80	21	50	59	53	28	14	7.7	4.5	3.4	3.2	6.5	12
90	74	159	118	92	44	25	15	10	8.3	7.2	9.1	37
91	85	197	141	100	47	26	15	10	8.3	7.4	10	39
92	100	216	160	105	50	26	16	10	8.3	7.6	11	42
93	111	234	185	115	50	27	16	11	8.3	7.7	12	47
94	139	247	207	131	54	27	16	12	8.3	8.0	13	55
95	163	274	226	151	58	28	17	12	8.7	8.0	15	63
96	214	346	265	180	68	31	18	15	9.9	8.2	19	71
97	412	500	300	221	76	33	19	18	10	8.3	29	84
98	613	863	361	375	89	36	20	19	12	8.3	124	103
99	903	1492	540	736	98	39	21	20	13	8.7	352	307
99.5	962	3230	724	896	105	42	22	20	13	8.7	789	786
99.7	4040	4183	754	1114	112	44	22	20	13	9.1	947	1331
99.9	4210	5285	1013	1812	120	45	23	21	13	9.1	1285	2421
99.95	7180	5748	1211	2171	123	46	23	21	14	9.1	1552	2656
99.99	8609	6117	1370	2458	125	46	23	21	14	9.1	1766	2843
100	8610	6210	1410	2530	126	46	23	21	14	9.1	1820	2890

MATILIJA C A MATILIJA HOT SPRINGS

USGS Gauge Number 11115500

Daily Flows from 1927 to 1988

Drainage Area = 54.7 Square Miles

Gague Datum = 900.0 feet

Percent of Time Flow is Below This Value	Flow Values (cfs) January	February	March	April	May	June	July	August	September	October	November	December
0	0.1	0.2	0.1	0.1	0.2	0.3	0.6	0.28	0.27	0.2	0.1	0.1
10	1.8	3.0	2.9	3.0	3.7	2.7	1.3	0.9	0.7	0.6	0.9	1.2
20	3.4	4.4	4.1	5.7	5.0	3.6	2.0	1.3	1.2	1.2	1.5	2.1
30	4.3	6.8	7.1	7.6	6.2	4.4	2.7	1.8	1.9	1.7	2.0	3.2
40	5.6	10	11	9.7	7.2	5.0	3.5	2.5	2.4	2.3	2.5	3.7
50	8.5	15	17	13	9.3	6.6	4.5	3.2	2.8	2.8	3.2	4.4
60	11	25	27	20	12	8.4	6.0	4.1	3.4	3.4	4.0	5.8
70	20	46	55	42	18	12	8.5	5.6	4.2	4.3	5.0	7.6
80	30	83	129	72	30	16	11	7.5	6.0	5.5	6.5	13
90	75	221	250	133	45	26	16	9.9	8.5	7.0	9.5	36
91	85	237	276	147	52	28	17	10	8.7	7.2	10	40
92	95	92	305	160	58	30	18	11	9.0	7.4	12	48
93	106	303	337	170	62	32	20	11	9.3	7.8	13	60
94	120	348	375	181	69	34	24	13	11	8.4	14	70
95	152	407	421	201	83	40	25	14	11	9.5	17	84
96	193	491	480	222	103	44	27	15	12	10	25	119
97	250	666	605	253	122	48	30	16	15	13	41	152
98	396	960	767	346	146	55	41	20	18	15	86	206
99	651	1426	1142	568	188	64	48	23	31	21	162	294
99.5	988	2492	1635	774	216	70	66	26	51	71	368	560
99.7	1520	3179	2053	888	231	89	68	37	70	99	421	634
99.9	3661	4887	5493	1068	289	222	219	51	130	166	525	1077
99.95	4237	5362	7107	1484	301	336	234	56	136	192	675	1387
99.99	7519	6072	7604	2209	326	412	238	75	177	232	1079	2597
100	8340	6250	7740	2390	332	431	239	80	187	242	1180	2900

NF MATILIJA C A MATILIJA HOT SPRINGS CA

USGS Gauge Number 11116000

Daily Flows from 1933 to 1983

Drainage Area = 15.6 Square Miles

Gauge Datum = 1,142.02 feet

Percent of Time Flow is Below This Value	Flow Values (cfs)											
	January	February	March	April	May	June	July	August	September	October	November	December
0	0.1	0.9	0.4	0.4	0.3	0.1	0.1	0.1	0.1	0.2	0.2	0.4
10	1.7	1.6	1.6	1.6	1.2	0.7	0.4	0.2	0.2	0.37	0.5	0.9
20	3.3	2.4	2.6	2.2	1.6	1.0	0.5	0.4	0.4	0.5	0.8	1.1
30	3.8	3.4	3.3	3.0	2.2	1.3	0.7	0.5	0.5	0.6	0.9	1.3
40	4.7	4.1	4.3	3.7	2.6	1.6	0.9	0.7	0.7	0.8	1.1	1.5
50	6.4	5.2	6.4	4.8	3.1	2.0	1.2	0.8	0.8	1.0	1.2	1.8
60	9.3	7.0	8.9	7.0	4.3	2.5	1.5	1.1	1.0	1.2	1.4	2.5
70	12	14	15	12	6.4	3.6	1.9	1.4	1.3	1.3	2.0	3.4
80	20	24	34	19	10	6.4	3.6	2.6	2.4	2.0	2.6	4.5
90	30	68	74	34	17	10	6.2	4.6	4.0	3.0	4.0	11
91	32	78	79	38	18	11	6.5	4.8	4.4	3.1	4.1	13
92	33	83	87	41	20	11	7.0	4.8	4.4	3.1	4.5	14
93	35	92	95	45	21	12	7.2	5.1	4.6	3.2	4.7	17
94	38	108	109	49	24	12	7.5	5.2	4.8	3.2	4.9	19
95	42	131	121	55	26	13	7.9	5.4	4.8	3.4	5.3	21
96	47	159	146	63	29	14	8.6	5.6	5.0	3.7	5.8	25
97	50	222	180	81	32	15	9.1	6.2	5.2	3.8	7.0	38
98	53	337	231	108	35	16	10	7.5	5.4	4.7	12	62
99	58	654	330	166	38	18	12	8.0	5.6	5.5	45	134
99.5	60	1161	530	244	42	20	14	9.0	5.8	6	152	171
99.7	60	1386	661	379	45	23	14	9.0	5.8	6	262	248
99.9	62	2453	1389	610	48	23	15	10	6.0	14	447	494
99.95	63	3192	1641	812	50	24	16	10	6.0	27	551	713
99.99	64	4175	2344	1346	51	24	16	10	6.0	36	736	759
100	64	4420	2520	1480	51	24	16	10	6.0	38	782	770

Ventura River near Ojai, California

USGS Gauge 11116500

Flows from 1922 to 1924

Drainage Area = 70.70 Square Miles

Gauge Datum = NA

Percent of Time Flow is Below This Value	Flow Values (cfs) January	February	March	April	May	June	July	August	September	October	November	December
0	4.7	5.0	5.5	6.5	3.2	1.0	3.9	2.7	2.4	2.5	3.2	3.5
10	5.0	5.5	10	14	4.0	2.8	4.2	3.0	2.5	2.5	3.5	4.1
20	5.5	7.0	19	21	12	7.5	4.6	3.5	3.0	3.0	3.8	4.9
30	6.0	12	33	24	13	8.5	5.5	4.0	3.5	3.5	4.5	5.0
40	12	12	37	28	15	10	5.7	4.5	4.0	4.1	4.5	5.5
50	13	22	42	31	19	10	6.5	4.5	4.3	4.5	10	10
60	14	43	50	41	24	14	9.2	5.5	4.6	8.5	12	14
70	18	45	84	68	34	22	15	11	8.0	8.5	12	15
80	38	75	136	76	48	26	18	12	8.5	10	14	19
90	43	239	163	100	56	28	20	14	10	12	17	72
91	43	252	163	103	57	28	20	14	10	12	18	75
92	44	276	163	104	58	29	20	14	10	12	18	80
93	44	298	170	110	58	30	20	14	10	12	19	82
94	44	343	172	112	58	31	20	14	10	12	20	86
95	46	385	181	116	58	31	21	14	10	12	23	104
96	48	449	188	120	59	33	23	14	10	12	27	125
97	52	430	201	124	61	34	23	14	10	12	42	152
98	57	679	227	133	63	34	23	14	10	13	58	189
99	62	760	272	142	65	34	23	15	10	13	79	300
99.5	79	806	303	144	63	34	23	16	10	13	89	378
99.7	90	848	340	151	63	34	23	16	10	13	89	445
99.9	101	889	377	158	64	34	23	16	10	14	90	512
99.95	103	900	386	159	64	34	23	16	10	14	90	528
99.99	105	908	393	161	64	34	23	16	10	14	90	542
100	106	910	395	161	64	34	23	16	10	14	90	545

Ventura River nr Meiners Oaks, CA

USGS Gauge 11116550

Flows from 1959 to 1988

Drainage Area = 76.40 Square Miles

Gauge Datum = NA

Percent of Time Flow is Below This Value	Flow Values (cfs)											
	January	February	March	April	May	June	July	August	September	October	November	December
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	0.5	1.6	0.0	0.0	0.18	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	2.3	2.4	0.7	1.8	1.6	0.2	0.0	0.0	0.0	0.0	0.0	0.10
40	4.1	3.6	2.0	4.4	3.4	1.3	0.0	0.0	0.0	0.0	0.0	0.75
50	5.8	5.5	4.7	6.4	5.8	2.2	0.0	0.0	0.0	0.0	0.04	2.1
60	7.6	8.8	7.3	8.0	7.9	3.9	0.5	0.0	0.0	0.0	0.43	5.2
70	9.6	11	9.0	10	9.8	7.0	1.7	0.5	0.1	0.0	1.7	8.0
80	13	15	12	13	13	10	5.2	3.5	1.3	0.9	4.2	12
90	18	25	23	21	19	16	14	10	9.0	4.8	9.0	21
91	19	30	28	23	20	16	16	10	9.4	6.8	9.8	22
92	19	34	50	27	28	17	20	11	10	6.8	10	23
93	20	53	131	158	34	17	21	11	10	7.8	11	24
94	21	80	183	170	47	18	23	12	11	8.7	12	25
95	23	151	259	177	61	19	26	12	11	9.4	13	26
96	26	290	369	190	91	23	28	12	12	10	18	30
97	40	419	474	209	127	29	30	14	12	10	24	31
98	210	885	562	227	154	40	32	15	13	10	28	50
99	789	3915	721	259	183	45	35	17	13	17	97	80
99.5	1132	5591	1237	306	207	106	35	18	16	25	251	116
99.7	3738	7121	1843	353	213	127	35	19	20	28	273	586
99.9	6667	8057	9155	355	265	139	35	19	24	31	727	1667
99.95	9983	9329	9192	355	280	140	35	19	30	33	1304	2638
99.99	12637	10346	9222	355	292	140	35	19	36	35	1765	3416
100	13300	10600	9230	355	295	140	35	19	37	36	1880	3610

San Antonio Creek at Casitas Springs

USGS Gauge 11117500

Flows from 1950 to 1983

Drainage Area = 51.2 square miles

Gauge Datum = 307.55 feet

Percent of Time Flow is Below This Value	Flow Values (cfs) January	February	March	April	May	June	July	August	September	October	November	December
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	0.0	0.0	0.2	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	0.0	0.0	0.6	0.4	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	0.5	0.0	1.0	0.9	0.5	0.1	0.0	0.0	0.0	0.0	0.0	0.0
40	1.2	0.0	2.3	1.8	0.8	0.2	0.0	0.0	0.0	0.0	0.0	0.1
50	2.6	0.0	4.2	3.5	1.7	0.6	0.1	0.0	0.0	0.0	0.0	0.5
60	3.4	5.0	6.5	5.0	3.3	1.9	0.8	0.6	0.4	0.15	0.44	1.3
70	4.6	6.5	9.8	6.7	4.6	3.3	1.9	1.3	1.2	0.76	1.0	2.7
80	7.3	15	25	20	12	8.8	5.7	4.2	3.3	1.9	2.9	3.9
90	17	56	92	50	25	15	10	6.3	5.0	3.8	4.2	6.5
91	23	63	100	54	26	16	10	6.9	5.3	3.9	4.6	6.9
92	28	75	108	61	27	17	11	7.3	6.1	3.9	5.0	7.0
93	33	86	118	65	29	18	11	7.5	6.1	4.2	5.1	8.1
94	46	109	132	71	31	19	12	7.8	6.5	4.2	5.7	10
95	62	153	150	81	35	20	13	8.3	6.5	4.2	6.5	12
96	85	227	179	90	40	23	13	8.8	6.9	4.2	11	15
97	133	370	219	101	44	23	14	10	6.9	4.6	18	25
98	340	572	317	119	50	27	15	14	7.8	5.0	31	66
99	719	1230	434	193	64	32	18	15	8.7	5.3	62	198
99.5	939	2012	927	418	78	36	20	15	9.8	5.7	234	377
99.7	1411	3120	1246	623	86	37	21	15	15	5.7	403	399
99.9	2721	3686	2896	1360	90	39	22	15	23	6.8	524	1464
99.95	6384	3993	3384	1625	95	39	22	20	78	9.3	1008	1818
99.99	9597	4239	3789	1837	99	39	22	25	124	11	1410	2084
100	10400	6740	3890	1890	100	39	22	26	135	12	1510	2150

Coyote Creek near Oak View CA

USGS Gague11117600

Flows from 1959 to 1988

Drainage Area = 13.20 square miles

Gauge Datum = 577.37 feet

Percent of Time Flow is Below This Value	Flow Values (cfs) January	February	March	April	May	June	July	August	September	October	November	December
0	0.20	0.30	0.10	0.10	0.10	0.00	0.00	0.00	0.00	0.00	0.06	0.09
10	0.48	0.6	0.53	0.53	0.33	0.20	0.10	0.05	0.05	0.08	0.10	0.32
20	0.73	0.97	0.9	0.82	0.57	0.31	0.13	0.10	0.10	0.10	0.20	0.40
30	0.95	1.2	1.3	1.1	0.75	0.46	0.22	0.13	0.12	0.20	0.30	0.56
40	1.1	1.5	1.8	1.3	0.93	0.62	0.34	0.20	0.19	0.22	0.39	0.8
50	1.3	2.0	2.5	1.7	1.1	0.70	0.41	0.26	0.2	0.27	0.46	0.9
60	1.7	2.9	4.6	3.1	1.3	0.85	0.50	0.31	0.24	0.33	0.56	1.0
70	2.7	4.9	8.4	5.0	2.4	1.5	0.80	0.38	0.32	0.52	0.67	1.5
80	4.6	14	18	8.1	3.6	2.3	1.3	0.76	0.67	0.64	1.0	2.3
90	12	39	35	14	5.8	3.3	2.1	1.3	1.0	1.0	1.9	5.8
91	14	52	38	15	6.1	3.4	2.2	1.4	1.1	1.0	2.1	6.9
92	17	61	41	16	6.8	3.6	2.4	1.4	1.2	1.0	2.4	8.0
93	20	76	45	18	7.0	3.7	2.4	1.5	1.2	1.0	3.3	9.6
94	27	97	51	20	7.8	3.9	2.4	1.5	1.2	1.1	4.2	11
95	40	141	60	22	8.8	4.1	2.5	1.6	1.3	1.1	5.2	14
96	75	179	75	25	10	4.7	2.6	1.6	1.3	1.1	7.9	19
97	100	242	106	33	11	5.4	2.7	1.7	1.4	1.1	12	27
98	154	407	150	36	13	6.8	2.9	1.9	1.4	1.2	20	49
99	357	837	237	52	15	7.0	3.0	2.3	1.6	1.7	74	140
99.5	474	1663	353	75	20	7.2	3.4	3.3	3.0	2.1	130	211
99.7	579	1873	589	92	22	7.9	4.0	4.4	3.2	2.8	181	385
99.9	1403	2203	1046	164	29	8.5	4.2	7.9	20	13	297	695
99.95	1951	2212	2013	201	39	8.5	4.3	10	52	43	396	907
99.99	2390	2218	2787	230	47	8.5	4.4	12	78	67	475	1077
100	2500	2220	2980	237	49	8.5	4.4	13	84	73	495	1120

Santa Ana Creek Near Oak View
USGS Gauge 11117800
Flows from 1959 to 1988
Drainage Area = 9.11 square miles
Gauge Datum = 612.43 feet

Percent of Time Flow is Below This Value	Flow Values (cfs)											
	January	February	March	April	May	June	July	August	September	October	November	December
0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0.1	0.1	0.07	0	0	0	0	0	0	0	0
20	0.2	0.56	0.3	0.20	0.08	0	0	0	0	0	0	0
30	0.6	0.91	0.93	0.49	0.2	0.06	0	0	0	0	0	0
40	0.79	1.2	1.3	0.73	0.32	0.10	0	0	0	0	0	0.21
50	1.1	1.9	2.3	1.1	0.56	0.14	0.01	0	0	0	0	0.47
60	1.7	3.1	4.0	2.2	0.82	0.20	0.06	0	0	0	0	0.67
70	3.0	5.2	7.5	4.5	1.5	0.40	0.10	0	0	0	0.1	0.95
80	5.3	13	15	7	2.4	1.2	0.20	0.07	0.05	0.13	0.4	2.4
90	10	36	28	12	5.0	2.1	0.90	0.29	0.14	0.44	1.2	6.5
91	12	42	31	13	5.3	2.4	0.90	0.30	0.15	0.50	1.4	7.1
92	15	52	36	14	6.2	2.6	0.96	0.30	0.18	0.50	1.6	8.5
93	19	68	38	16	7.1	2.8	1.0	0.33	0.20	0.50	1.9	10
94	22	73	44	17	7.8	3.0	1.1	0.38	0.20	0.50	3.2	13
95	33	96	52	19	8.7	3.6	1.1	0.40	0.28	0.51	4.4	16
96	45	143	62	21	9.7	3.7	1.2	0.54	0.33	0.83	7.3	31
97	74	189	71	24	10	4.0	1.2	0.60	0.44	1.2	12	31
98	150	298	95	31	11	4.3	1.4	0.74	0.52	1.5	22	56
99	313	548	166	43	13	5.0	1.6	0.91	0.59	2.0	93	138
99.5	431	996	287	59	14	5.6	2.2	1.2	0.70	3.7	152	190
99.7	456	1101	372	69	14	6.2	2.4	1.5	1.9	5.8	177	352
99.9	912	1333	873	96	16	6.7	2.6	1.9	21	16	254	739
99.95	1406	1451	1301	99	17	6.8	2.7	2.5	92	68	394	803
99.99	1801	1546	1644	102	18	6.8	2.8	3.1	149	110	506	854
100	1900	1570	1730	103	18	6.8	2.8	3.2	163	120	534	867

Coyote Creek Near Ventura, CA

USGS Gauge Number 1118000

Daily Flows from 1959 to 2000

Drainage Area = 41.20 square miles

Contributing Drainage Area = 2.00 square miles below casitas Dam since 1959

Gauge Datum = 224.95 feet

Percent of Time Flow is Below This Value	Flow Values (cfs) January	February	March	April	May	June	July	August	September	October	November	December
0	0	0.05	0.08	0	0	0	0	0	0	0	0	0
10	0.05	0.11	0.13	0.11	0.05	0	0	0	0	0	0	0
20	0.09	0.16	0.16	0.14	0.08	0.04	0	0	0	0	0	0
30	0.12	0.2	0.23	0.19	0.11	0.07	0	0	0	0	0	0.02
40	0.18	0.32	0.37	0.3	0.15	0.09	0.05	0	0	0	0	0.04
50	0.23	0.36	0.52	0.38	0.19	0.10	0.06	0.02	0	0.01	0.01	0.05
60	0.36	0.44	0.78	0.42	0.25	0.15	0.07	0.03	0.02	0.02	0.02	0.09
70	0.45	0.53	1.3	0.56	0.37	0.18	0.09	0.04	0.04	0.03	0.03	0.17
80	0.63	1.2	3	0.92	0.50	0.28	0.13	0.05	0.05	0.05	0.05	0.28
90	0.9	3.7	37	23	0.59	0.35	0.17	0.09	0.08	0.07	0.20	0.35
91	0.96	5.3	48	27	0.59	0.35	0.18	0.09	0.09	0.08	0.20	0.36
92	1.1	6.1	65	30	0.59	0.36	0.19	0.09	0.09	0.14	0.21	0.41
93	1.2	7.6	81	34	0.64	0.37	0.19	0.10	0.09	0.14	0.26	0.48
94	1.4	11	90	35	0.68	0.39	0.2	0.10	0.13	0.14	0.28	0.74
95	1.6	12	115	36	0.71	0.41	0.22	0.10	0.15	0.14	0.32	0.84
96	2	72	145	38	1.30	0.41	0.23	0.10	0.16	0.14	0.35	0.92
97	3.3	141	173	39	7	0.42	0.24	0.11	0.17	0.14	0.41	1.1
98	5.2	246	220	41	11	0.44	0.26	0.11	0.19	0.14	0.50	1.6
99	9.6	335	261	46	19	0.8	0.3	0.11	0.20	0.14	0.66	5
99.5	27	471	279	52	21	0.9	0.32	0.12	0.30	0.14	0.72	13
99.7	29	527	284	53	23	0.9	0.45	0.12	0.51	0.16	1.2	14
99.9	53	584	303	54	24	1.0	0.50	0.13	6	0.21	5	16
99.95	60	598	309	54	25	1.0	0.51	0.13	7	0.22	6	17
99.99	66	609	314	54	25	1.0	0.52	0.13	8	0.23	7	18
100	68	612	315	54	25	1.0	0.52	0.13	9	0.23	7	18

Ventura River Near Ventura

USGS Gauge Number 1118500

Daily Flows from 1929 to 2000

Drainage Area = 188.00 square miles

Gauge Datum = 200.0 feet

Percent of Time Flow is Below This Value	Flow Values (cfs) January	February	March	April	May	June	July	August	September	October	November	December
0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0.2	0.11	0.1	0	0	0	0	0	0	0
20	0.1	0.6	2.7	2.1	1	0.37	0.1	0	0	0	0	0
30	0.63	4.8	7.3	4.8	2.7	1.3	0.4	0.1	0	0	0	0.04
40	2.6	11	12	8.5	5	3	1	0.4	0.1	0	0	0.3
50	5.4	18	20	14	8.2	4.2	1.9	1	0.4	0.12	0.19	1.1
60	11	30	31	20	12	6.4	3.2	1.8	0.8	0.5	0.61	3.2
70	20	50	57	35	22	10	5.6	3.5	2.2	1.5	1.8	7
80	36	117	161	93	41	18	11	6.9	5.6	4.2	4.7	13
90	86	441	560	237	85	44	22	12	9.3	7.3	11	27
91	102	499	628	273	97	47	24	13	9.8	7.6	12	30
92	128	598	700	301	110	52	26	14	12	8	14	34
93	169	695	792	352	129	58	30	16	13	8.5	17	40
94	216	848	862	392	137	65	34	18	13	11	20	52
95	305	1100	944	447	156	70	38	20	15	14	22	63
96	464	1500	1100	496	185	75	43	23	16	16	24	95
97	771	2000	1320	563	236	83	46	27	20	19	30	140
98	1420	2870	1800	679	274	115	51	31	22	22	62	292
99	2710	5080	2880	1240	362	143	65	35	28	26	145	545
99.5	5000	8340	5280	1840	437	180	71	37	29	33	426	919
99.7	6800	9420	6990	2130	497	200	82	38	32	57	578	1290
99.9	15600	20000	14400	3570	687	244	89	41	33	135	931	3610
99.95	16679	20600	18000	5050	703	246	89	41	34	340	1170	3700
99.99	19296	21719	18390	6789	860	252	89	41	312	465	3444	4839
100	20000	22000	18500	7260	904	254	89	41	387	500	4060	5160

Ventura River Near Ventura

USGS Gauge Number 1118500

Daily Flows from 1959 to 2000

Drainage Area = 188.00 square miles

Gauge Datum = 200.0 feet

Percent of Time Flow is Below This Value	January	February	March	April	May	June	July	August	September	October	November	December
	Flow Values (cfs)											
0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0.17	0.31	0.48	0.32	0.05	0	0	0	0	0	0
20	0.13	0.5	2.1	3.5	2.4	0.89	0.15	0	0	0	0	0
30	0.55	3.7	7.3	6	3.6	2.3	0.73	0.15	0	0	0	0.07
40	2.4	7.6	12	9.2	5.7	3.5	1.6	0.7	0.06	0	0	0.51
50	4.7	12	18	13	8.5	4.6	2.5	1.4	0.3	0.09	0.08	1.3
60	7.6	20	25	18	12	7.2	3.7	2.4	1.2	0.7	0.9	3.9
70	14	39	46	26	19	11	6.7	4.4	2.7	1.6	2.1	7
80	25	90	112	71	34	17	11	7.4	5.9	3.8	4.4	12
90	72	430	562	274	98	46	25	12	9.4	6.8	10	22
91	89	499	656	300	118	52	27	12	9.8	7.0	13	24
92	106	589	724	338	131	57	31	13	11	7.3	17	26
93	157	734	800	373	136	60	33	14	12	7.8	20	28
94	236	1020	859	418	150	68	35	16	12	8.8	23	35
95	373	1430	920	451	184	72	42	16	13	13	25	46
96	718	1750	1070	486	236	78	44	21	15	17	30	57
97	1050	2330	1190	535	274	96	48	25	19	19	52	89
98	1710	3670	1600	585	347	133	53	29	27	22	100	93
99	2884	7053	2413	675	421	177	65	32	29	27	278	461
99.5	5071	10359	3469	882	505	200	81	36	32	47	571	806
99.7	6942	16046	6135	1353	550	218	86	36	32	64	848	1227
99.9	15770	20505	13140	1782	699	246	89	38	34	129	1115	3101
99.95	17966	21189	15894	1834	776	249	89	39	170	207	2283	4175
99.99	19593	21838	17979	1839	878	253	89	40	344	313	3705	4963
100	20000	22000	18500	1840	904	254	89	40	387	340	4060	5160

15. Exhibit C. Hydraulic Properties for Current Conditions

Exhibit B3
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Standard Table 1
10-year event

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
-0.2258	41300	-18	2.53	-10.25	2.58	0.00005	1.82	22673.37	1479	0.08
-0.1784	41300	-14.25	2.5		2.62	0.000182	2.8	14767.54	1342	0.15
-0.1311	41300	-10.5	2.47		2.75	0.000608	4.2	9831.8	1204	0.26
-0.0837	41300	-4.7	2.39		3.2	0.003108	7.23	5712.95	1059	0.55
-0.0364	41300	-1.8	2.98	2.98	4.95	0.011494	11.29	3658.85	929	1.00
0.0477	41300	2.5	8.22	8.22	10.47	0.010789	12.03	3437.92	788.42	1.00
0.1052	41300	-0.3	11.08	8.22	11.61	0.001647	6.49	10071.29	2392.22	0.42
0.1383	41300	1	11.32	8.29	11.9	0.001706	6.83	10094.87	2372.43	0.43
0.1909	41300	-1.3	11.48	8.28	12.25	0.000653	7.38	8742.05	1742.01	0.47
0.191	Mult Open									
0.1945	41300	-1.3	11.7	8.29	12.42	0.000592	7.15	9130.23	1813.84	0.45
0.3579	41300	3.6	13.88	13.88	15.91	0.011275	11.44	3615.43	1683.18	1.00
0.4394	41300	0	16.27	16.18	18.42	0.003439	11.76	3510.85	2069.08	0.98
0.4395	Mult Open									
0.464	41300	0	18.41	16.18	19.32	0.000986	7.64	5406.16	2395.54	0.55
0.5204	41300	5.56	18.56	18.56	21.01	0.007435	12.8	4015.61	1618.27	0.88
0.5922	41300	6.2	21.78	21.78	24.03	0.003563	12.04	3429.73	891.1	1.00
0.5923	Mult Open									
0.6028	41300	6.2	23.66	21.77	24.62	0.001502	7.83	5274.3	2420.22	0.65
0.6629	41300	11.8	24.05	24.05	26.06	0.006144	12.18	5616.45	2372.46	0.81
0.7577	41300	15.51	27.4	27.4	29.13	0.004934	12.32	7334.67	2493.15	0.75
0.8523	41300	17.62	29.25	28.81	32.31	0.006053	14.26	3481.67	867.95	0.84
0.947	41300	19.29	32.53	31.43	35.17	0.00531	13.83	3982.32	747.2	0.79
1.0417	41300	20.72	35.19	31.52	36.98	0.002462	11.06	4471.78	522.92	0.56
1.1364	41300	22.96	36.97	36.97	41.07	0.006448	16.87	3150.15	470.9	0.89
1.2311	41300	25.25	41.57	37.74	43.12	0.002287	10.15	4384.87	424.88	0.53
1.3258	41300	30.21	42.42	40.16	44.84	0.003841	12.57	3639.71	542.87	0.68
1.4205	41300	30.59	45.15	41.18	46.24	0.001795	8.46	5293.22	602.97	0.47
1.5152	41300	36.12	46.32	44.4	47.53	0.003765	8.84	4671.34	744.75	0.62
1.6098	41300	36.91	48.1	48.1	50.97	0.009551	13.63	3111.93	602.58	0.98
1.7045	41300	41.84	52.57	52.57	55.65	0.007819	15.2	3811.72	673.42	0.94
1.7992	41300	43.24	57.05	57.05	61.08	0.007395	16.44	2912.66	407.17	0.93
1.8939	41300	43.82	61.71	57.89	63.18	0.002198	10.07	4775.82	468.58	0.52
1.9886	41300	50.21	62.97	61.1	64.73	0.004224	10.87	4256.32	629.08	0.68
2.0827	41300	53.42	65.18	64.5	67.42	0.006305	12.28	3953.72	785.91	0.82
2.178	41300	59.08	68.79	68.48	70.9	0.007282	12.48	4456.34	1061.34	0.87
2.2727	41300	62.24	72.52	70.82	73.34	0.00329	7.28	5676.09	1096.19	0.56
2.3674	41300	68.02	75.3	75.3	76.95	0.012136	10.3	4011	1225.65	1.00
2.4621	41300	70.56	80.52	79.9	81.69	0.007486	8.71	4743.74	1297.38	0.80
2.5568	41300	72.7	85.72	85.72	89.2	0.008005	15.37	3178.32	527.76	0.94
2.6515	41300	79.32	91.68	91.68	95.6	0.007318	16.34	3024.85	430.16	0.92
2.7462	41300	81.71	95.06	94.34	99.02	0.006413	15.99	2663.68	321.15	0.88
2.8409	41300	82.54	99.24	97.2	101.81	0.004481	12.88	3206.41	325.17	0.72
2.9356	41300	84.42	101.27		104.53	0.005651	14.49	2849.61	289.2	0.81
3.0303	41300	91.69	105.87	105.87	110.77	0.008621	17.76	2325.65	238.13	1.00
3.125	41300	95.51	110.67	110.67	116.17	0.008416	18.82	2194.29	200.2	1.00
3.1546	41300	99.3	112.14	112.14	117.6	0.002796	18.75	2202.79	201.39	1.00
3.1547	Bridge									
3.1591	41300	99.3	114.09	112.12	118	0.00169	15.86	2603.5	208.34	0.79
3.178	41300	100.18	114.04	113.25	118.65	0.006852	17.23	2397.41	214.96	0.91
3.2197	41300	100.57	115.9	115.9	121.03	0.008502	18.18	2271.86	221.39	1.00
3.4091	41300	108.96	124.25		126.56	0.003559	12.2	3386.94	322.24	0.65
3.5038	41300	114.22	126.1	126.1	130.11	0.009125	16.08	2568.88	320.4	1.00
3.5985	41300	118.58	131.72	131.72	136.32	0.009005	17.22	2398.52	263.83	1.01
3.6932	41300	121.48	136.41	135.52	140.24	0.006638	15.71	2629.07	266.18	0.88
3.7879	41300	125.17	139.72		143.58	0.006671	15.76	2620.41	266.55	0.89
3.8826	41300	125.36	143.27	141.79	146.66	0.005559	14.8	2815.69	320.84	0.81
3.9773	41300	135.97	147.86	147.86	152.32	0.008751	16.93	2439.26	274.06	1.00
4.072	41300	138.06	152.98	152.98	157.91	0.008799	17.82	2317.01	235.34	1.00
4.1667	41300	142.6	158.33		160.87	0.003687	12.79	3274.81	342.62	0.67
4.2614	41300	146.37	160.52		163.25	0.005832	13.3	3224.12	455.04	0.81
4.3561	41300	151.11	163.84		166.73	0.007808	13.66	3023.43	433.37	0.91
4.4508	41300	153.7	167.8	167.8	171.38	0.009312	15.2	2734.36	388.4	1.00
4.5455	41300	160.32	172.79	172.79	176.99	0.008995	16.45	2511.2	300.87	1.00
4.6402	36400	161.55	176.98	176.21	180.92	0.006743	15.95	2321.31	305.91	0.89
4.7348	36400	166.42	180.27	179.42	184.34	0.006756	16.19	2268.33	286.95	0.90
4.8295	36400	168.97	183.68		187.57	0.006111	15.81	2302.02	217.28	0.86
4.9242	36400	170.99	186.38	185.93	191.23	0.007494	17.67	2059.95	191.12	0.95
5.0189	36400	173.4	189.74	188.87	194.87	0.006771	18.18	2002.14	161.58	0.91

Exhibit B3

River Sta	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
	(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
5.1136	36400	174.41	195.26		196.59	0.00149	9.64	4752.71	475.27	0.44
5.303	36400	183.19	197.12	197.12	200.37	0.009896	14.45	2519.1	389.83	1.00
5.3977	36400	188.74	203.87	203.87	206.64	0.010599	13.36	2724.49	504.39	1.01
5.4924	36400	196.48	208.25	208.25	212.06	0.009089	15.67	2340.06	342.66	0.99
5.5871	36400	200.76	212.69		215.34	0.004674	13.08	2785.6	300.84	0.74
5.6818	36400	203.2	215.75	215.75	220.26	0.00818	17.11	2218.1	269.95	0.98
5.7765	36400	204.47	219.37	218.54	223.83	0.006298	17.03	2238.45	222.18	0.88
5.8712	36400	206.39	222.57	221.64	227.07	0.006629	17.03	2137.12	189.67	0.89
5.9301	36400	211.19	227.66		228.86	0.001351	8.85	4486.42	476.75	0.42
5.9475	36400	212.6	227.57	223.21	229.1	0.00072	9.92	3668.31	376.84	0.52
5.9476	Bridge									
5.9536	36400	212.6	227.7	223.22	229.19	0.000695	9.81	3708.76	393.39	0.51
5.9742	36400	212.75	227.7		229.44	0.003029	10.59	3455.88	429	0.60
6.0606	36400	214.97	229.38	227.65	231.08	0.004325	10.48	3480.7	477.53	0.68
6.1553	36400	219.92	232.18		233.24	0.004076	8.26	4410.02	829.37	0.63
6.3447	36400	230.8	240.55	240.55	243.05	0.013519	13.22	3113.08	709.93	1.11
6.4394	35200	238.45	245.84	245.08	247.77	0.006701	11.19	3234.91	730.44	0.82
6.5341	35200	238.98	249.33	249.3	251.66	0.00861	12.42	3248.15	868.15	0.92
6.6288	35200	244.07	253.34		255.24	0.005875	11.26	3616.22	834.71	0.78
6.7235	35200	245.77	256.56	255.45	258.1	0.005391	9.97	3531.73	614.54	0.73
6.8182	35200	251.59	260.25	260.25	262.35	0.010824	11.64	3064.94	805.88	0.99
6.8389	35200	251.95	261.89	260.89	263.24	0.005483	9.36	3817.69	795.45	0.73
6.9129	35200	252.41	264.35	264.12	266.22	0.009924	10.98	3205.18	760.76	0.94
7.0076	35200	255.42	268.76	268.1	270.51	0.007442	10.6	3321.02	665.99	0.84
7.1023	35200	261.9	273.17	273.1	275.05	0.011115	11.02	3195.7	822.02	0.98
7.197	35200	268.71	278.56	278.17	279.93	0.008458	9.4	3743.24	995.13	0.85
7.2917	35200	275.59	283.83	283.83	285.42	0.012154	10.14	3471.14	1084.6	1.00
7.3864	35200	278.71	288.06	287.17	289.45	0.005544	10.37	4831.52	1044.26	0.75
7.4811	35200	282.27	292.32	292.32	294.94	0.00924	13.01	2810.26	703.26	0.96
7.5758	35200	285.81	296.76	296.42	298.93	0.006802	12.05	3454.09	891.88	0.84
7.6705	35200	287.09	299.79	298.83	302.33	0.006385	12.78	2753.44	371.82	0.83
7.7652	35200	290.73	303.8	303.8	306.95	0.009782	14.24	2471.99	390.91	1.00
7.8598	35200	296.44	308.4		310.17	0.004239	10.67	3300.87	434.46	0.68
7.9545	35200	302.3	311.53		312.66	0.005623	8.53	4126.69	937.35	0.72
8.0492	16000	307.63	314.62	314.62	316.53	0.011964	11.37	1541.62	416.54	1.02
8.1439	16000	313.73	320.9	320.9	322.5	0.011876	10.21	1634.82	524.39	0.99
8.2386	16000	319.39	327.01	327.01	328.48	0.011655	10.01	1857.93	701.95	0.98
8.3333	16000	328.06	333.26	333.08	334.43	0.012147	9.13	2003.33	755.56	0.97
8.428	16000	331.94	339.87	339.68	340.76	0.013762	8.76	2342.86	839.32	1.01
8.5227	16000	338.87	345.4	345.11	346.4	0.009227	8.08	2085.7	831.32	0.85
8.6174	16000	343.61	351.66	351.66	352.8	0.013473	8.59	1889.86	885.93	1.00
8.7121	16000	349.67	357.65		358.76	0.010605	8.47	1912.66	739.09	0.91
8.8068	16000	359.02	364.43	364.1	365.33	0.01584	8.61	2310.61	1034.03	1.06
8.9015	16000	363.31	370.44		370.86	0.007577	6.17	3558.33	1454.13	0.74
8.9962	16000	371.3	374.91	373.77	375.32	0.014085	5.78	3172.85	1263.19	0.92
9.0909	16000	377.95	383.23	382.44	383.71	0.007718	6.96	3990.55	1982.29	0.77
9.1857	16000	383.92	388.84	388.26	389.35	0.016513	8.35	3434.16	1789.14	1.07
9.2804	16000	389.77	396.11	395.64	396.89	0.012894	9.49	3773.62	1613.88	1.00
9.3264	16000	393.7	398.64	398.64	400.1	0.010023	11.29	3267.6	1402.59	0.95
9.3786	16000	393.7	401.44	401.44	404.79	0.003572	14.69	1088.9	162.56	1.00
9.3787	Bridge									
9.3864	16000	393.7	403.25	401.45	405.34	0.001979	11.59	1380.25	168.17	0.71
9.4009	16000	393.7	404.7	400.81	405.95	0.002986	8.97	1783.32	185.07	0.51
9.4697	16000	398.1	406.14	406.14	408.81	0.010462	13.12	1219.93	227.22	1.00
9.5644	16000	401.18	411.95	411.95	414.32	0.010823	12.34	1296.46	273.56	1.00
9.6591	16000	409.8	418.03	418.03	420.39	0.010814	12.35	1295.65	273.73	1.00
9.7538	16000	415.67	424.88	424.88	426.74	0.011652	10.95	1461.42	391.8	1.00
9.8485	16000	422.07	430.73	430.45	432.06	0.009561	9.25	1729.52	517.11	0.89
9.9432	16000	428.28	437.04	437.04	438.57	0.012355	9.93	1611.34	523.12	1.00
10.0379	16000	434.56	443.28	443.27	444.78	0.012477	9.83	1627.57	542.24	1.00
10.1326	16000	442.92	449.96	449.88	451.1	0.012656	8.58	1865.57	772.42	0.97
10.2273	16000	448.22	454.94	454.63	456.61	0.009159	10.38	1541.76	375.27	0.90
10.322	16000	454.69	462.59	462.59	463.92	0.019301	9.98	1831.03	720.59	1.18
10.4167	16000	465.52	471.09	471.09	472.38	0.01213	9.21	1880.38	817.71	0.97
10.5114	16000	470.79	477.07	477.07	478.76	0.01214	10.78	1839.53	862.96	1.01
10.6061	16000	472.91	483.54	483.54	485.12	0.011328	10.2	1713.5	624.71	0.97
10.7008	16000	480.67	490.41	490.41	491.9	0.010958	9.93	1792.13	724.93	0.96
10.7955	16000	489.29	497.08	497.08	498.47	0.01268	9.75	1822.48	701.58	1.00
10.8902	16000	496.03	503.78	503.78	505.06	0.012677	10.44	2264.8	834.44	1.02
11.0795	16000	507.17	516.21	516.21	517.9	0.012089	10.44	1553.63	481.78	1.00
11.1742	16000	515.38	522.7	522.7	524.17	0.012651	9.72	1645.81	574.08	1.00
11.2585	16000	521.8	528.47	528.47	529.99	0.004047	9.88	1618.83	531.54	1.00

Exhibit B3

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
11.2586	Bridge									
11.2678	16000	521.8	529.65	528.45	530.39	0.001625	6.93	2307.67	650.47	0.65
11.3636	16000	528.49	537.2	537.2	539.31	0.011155	11.84	1465.14	376.1	1.00
11.4583	16000	538.05	544.73	544.73	545.82	0.013992	8.38	1934.5	915.07	1.00
11.553	16000	546.02	552.5	552.5	553.58	0.014207	8.45	1953.78	921.5	1.01
11.6477	15000	553.37	560.03	560.03	561.15	0.013817	8.48	1780.17	835.58	1.00
11.7424	15000	560.24	567.46	567.46	568.69	0.013241	8.93	1713.64	712.6	1.00
11.8371	15000	569.1	575.6	575.6	576.76	0.015111	9.34	1985.97	888.44	1.06
11.9318	15000	574.3	582.08	581.79	583.08	0.011173	8.92	2179.63	762.71	0.93
12.0265	15000	581.06	589.15	589.15	590.66	0.014444	11.35	1925.9	640.26	1.09
12.1212	15000	588.79	596.44	596.44	598.25	0.01172	10.78	1401.66	416.35	1.00
12.2159	15000	598.12	604.73	604.73	606.11	0.012454	9.58	1676.02	633.68	0.99
12.3106	15000	604.4	611.41	611.1	612.48	0.013521	9.35	1987.72	662.55	1.02
12.4053	15000	614.29	619.19	618.5	620.14	0.013811	8.82	2015.78	662.59	1.01
12.5	15000	620.6	626.65	626.12	627.77	0.01335	9.38	2066.97	786.83	1.01
12.5947	15000	623.7	633.38	633.38	635.34	0.011207	11.47	1436.72	417.37	1.00
12.6894	15000	631.62	641.86	641.86	643.03	0.013215	8.81	1823.57	847.13	0.99
12.7841	15000	642.59	650.29	650.29	651.41	0.01236	8.77	2023.97	979.26	0.97
12.8788	15000	646.75	659	659	660.38	0.011187	9.63	1777.33	686.2	0.95
12.9735	15000	657.38	667.1	667.1	668.38	0.011737	9.9	2028.55	788.71	0.97
13.0682	15000	665.48	674.53	674.53	675.77	0.012342	9.83	2234.55	1061.15	0.99
13.1629	15000	673.6	681.34	681.34	682.9	0.01078	10.99	1935.72	802.45	0.97
13.2576	15000	678.5	687.06	687.06	689.45	0.010991	12.4	1209.49	259.82	1.01
13.3523	15000	686.32	698.32	698.32	700.8	0.010135	12.64	1215	312.46	0.98
13.447	15000	696.87	705.88	705.88	707.89	0.010962	11.39	1341.84	382.99	0.99
13.5417	15000	707.18	715	715	716.96	0.011636	11.46	1408.62	368.88	1.01
13.6364	15000	713.74	723.15	723.15	725.03	0.007215	11.6	1952.63	922.76	0.84
13.7311	15000	721.31	729.57	729.57	731.43	0.010596	11.42	1715.62	886.05	0.97
13.8258	15000	725.05	735.03	735.03	737.01	0.008271	11.81	1728.82	599.77	0.89
13.9205	15000	731.78	740.84	740.84	743.07	0.009117	12.3	1438.65	360.34	0.94
14.0152	15000	739.54	748.67	748.67	750.71	0.010343	11.79	1492.85	398.99	0.97
14.1098	15000	748.6	758.03	758.03	760.39	0.011276	12.31	1218.08	260.12	1.00
14.1335	15000	748.86	760.24	760.24	763.12	0.010664	13.61	1101.92	193.99	1.01
14.1761	15000	758.84	763.86	763.67	765.57	0.01009	10.5	1428.53	366.04	0.94
14.2045	15000	760.23	769.15	769.15	770.98	0.011883	10.84	1384.38	384.1	1.01
14.2992	15000	768.39	774.99	774.99	777.35	0.010762	12.33	1216.76	258.44	1.00
14.3939	15000	772.5	780.92	780.92	783.75	0.008928	13.72	1223.22	245.11	0.95
14.4886	15000	778.63	787.93	787.93	790.84	0.009927	13.92	1195.12	233.1	1.00
14.5833	15000	783.85	794.18	794.18	798.05	0.009455	15.79	949.81	123.87	1.01
14.678	15000	790.59	800.92	800.92	804.55	0.008754	15.34	1020.8	169.36	0.97
14.7727	15000	796.21	808.1	808.1	811.9	0.00924	15.64	966	139.02	0.99
14.8674	15000	809.22	818.15	818.15	821	0.007251	14.13	1398.47	333.47	0.89
14.9621	15000	813.66	823.4	823.4	826.46	0.008959	14.25	1160.82	209.87	0.97
15.0568	15000	820.46	830.34	830.34	833.31	0.010167	13.82	1085.75	184.33	1.00
15.1515	15000	831.78	841.02	841.02	843.63	0.010426	12.96	1157.97	221.33	0.99
15.2462	15000	839.7	851.04	851.04	853.12	0.011406	11.56	1297.58	312.8	1.00
15.3409	15000	851.88	863.01	863.01	864.7	0.01191	10.44	1449.56	447.79	0.99
15.4356	15000	865.18	872.75	872.75	875.59	0.010206	13.52	1109.41	196.1	1.00
15.4979	15000	868.13	878.25	878.25	881.52	0.008049	14.72	1150.87	207.83	0.94
15.5036	15000	867.1	882.57	882.57	886.39	0.002358	19.23	2207.37	310.33	0.88
15.5038	Bridge									
15.5066	15000	867.1	886.38	882.65	887.96	0.000826	13.26	3465.93	352.5	0.54
15.5104	15000	868.48	887.68		888.27	0.000633	7.06	3573.49	335.03	0.30
15.5303	15000	869.79	887.67		888.39	0.000811	6.95	2530.62	235.82	0.33
15.625	15000	879.18	888.02	888.02	890.91	0.010202	13.62	1101.27	191.77	1.00
15.7197	15000	892.76	900.65	900.65	903.73	0.009938	14.07	1067.06	177.91	1.00
15.8144	15000	900.54	912.56	912.56	915.51	0.009742	13.96	1169.74	222.97	0.98
15.9091	12500	908.92	921.31	921.31	924.99	0.009857	15.4	811.6	112.05	1.01
16.0038	12500	925.84	935.17	935.17	938.72	0.010229	15.11	827.1	118.07	1.01
16.0985	12500	936.57	946.7	946.7	950.15	0.009865	14.92	837.84	122.32	1.00
16.1932	12500	949.56	962.42	962.42	965.45	0.010299	13.98	894.44	148.78	1.00
16.2879	12500	960.39	970.83	970.83	974.94	0.009752	16.27	768.1	93.52	1.00
16.3826	12500	973.59	984.68	984.68	988.75	0.009613	16.17	772.96	95.41	1.00

Exhibit B4
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Standard Table 1
20-year event

River Sta	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
	(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
-0.2258	52700	-18.0	2.53	-9.52	2.61	0.00008	2.3	22673.4	1479.0	0.10
-0.1784	52700	-14.3	2.48		2.68	0.00030	3.6	14743.9	1342.0	0.19
-0.1311	52700	-10.5	2.44		2.89	0.00101	5.4	9786.9	1204.0	0.33
-0.0837	52700	-4.7	2.28		3.66	0.00542	9.4	5595.8	1059.0	0.72
-0.0364	52700	-1.8	3.68	3.68	6.00	0.01085	12.2	4311.2	929.0	1.00
0.0477	52700	2.5	9.09	9.09	11.63	0.01011	12.8	4164.6	896.9	0.99
0.1052	52700	-0.3	12.16	9.41	12.72	0.00152	6.8	12700.8	2462.3	0.42
0.1383	52700	1.0	12.37	9.07	12.98	0.00159	7.2	12602.0	2377.1	0.43
0.1909	52700	-1.3	12.47	8.89	13.40	0.00070	8.2	10642.9	2123.8	0.49
0.191	Mult Open									
0.1945	52700	-1.3	12.77	9.10	13.63	0.00062	7.9	11292.9	2266.5	0.47
0.3579	52700	3.6	14.62	14.62	16.97	0.01051	12.3	4324.6	2162.1	0.99
0.4394	52700	0.0	17.08	17.06	19.51	0.00368	12.5	4207.7	2264.3	1.02
0.4395	Mult Open									
0.464	52700	0.0	19.34	17.07	20.45	0.00099	8.4	6255.7	2488.2	0.57
0.5204	52700	5.6	19.90	19.90	22.07	0.00566	12.5	6591.6	2192.3	0.79
0.5922	52700	6.2	22.91	22.91	25.08	0.00360	11.8	4454.9	1609.7	1.00
0.5923	Mult Open									
0.6028	52700	6.2	24.63	22.91	25.68	0.00131	8.2	6433.5	2668.2	0.63
0.6629	52700	11.8	25.23	25.23	26.95	0.00477	11.9	8808.6	2814.3	0.73
0.7577	52700	15.5	28.21	28.21	29.95	0.00482	12.9	9368.2	2513.8	0.75
0.8523	52700	17.6	30.93	30.93	33.87	0.00500	14.4	5423.3	1372.1	0.78
0.947	52700	19.3	33.34	32.84	36.94	0.00663	16.2	4746.5	1085.7	0.89
1.0417	52700	20.7	36.87	33.11	39.00	0.00253	12.2	5403.9	605.4	0.58
1.1364	52700	23.0	38.59	38.59	43.18	0.00624	18.2	3942.0	509.7	0.90
1.2311	52700	25.3	43.37	39.10	45.22	0.00225	11.1	5195.6	486.6	0.54
1.3258	52700	30.2	44.17	41.42	46.87	0.00358	13.5	4606.6	567.3	0.68
1.4205	52700	30.6	46.93	42.29	48.21	0.00171	9.2	6448.3	789.0	0.47
1.5152	52700	36.1	48.09	45.35	49.29	0.00273	8.8	6005.3	763.8	0.55
1.6098	52700	36.9	49.19	49.19	52.47	0.00879	14.6	3798.7	651.1	0.97
1.7045	52700	41.8	53.73	53.73	57.24	0.00769	16.5	4591.7	679.7	0.95
1.7992	52700	43.2	58.56	58.56	63.18	0.00722	17.8	3531.7	413.4	0.94
1.8939	52700	43.8	63.56	59.21	65.28	0.00216	10.9	5648.3	474.3	0.53
1.9886	52700	50.2	64.82	62.38	66.62	0.00338	11.1	5451.8	665.2	0.63
2.0827	52700	53.4	66.51	65.54	68.92	0.00551	12.9	5153.8	950.8	0.79
2.178	52700	59.1	69.70	69.49	72.16	0.00725	13.6	5453.2	1104.1	0.89
2.2727	52700	62.2	73.55	71.46	74.48	0.00296	7.7	6840.1	1169.9	0.55
2.3674	52700	68.0	75.92	75.92	77.80	0.01155	11.0	4787.3	1274.7	1.00
2.4621	52700	70.6	81.06	80.52	82.51	0.00770	9.7	5455.9	1303.5	0.83
2.5568	52700	72.7	87.12	87.12	91.00	0.00748	16.4	3936.4	546.8	0.94
2.6515	52700	79.3	93.16	93.16	97.64	0.00721	17.6	3669.4	439.4	0.94
2.7462	52700	81.7	96.31	96.09	101.34	0.00705	18.1	3069.5	328.3	0.94
2.8409	52700	82.5	101.31	98.66	104.16	0.00399	13.6	3887.9	333.6	0.70
2.9356	52700	84.4	102.94		106.80	0.00555	15.8	3381.7	357.9	0.82
3.0303	52700	91.7	107.71	107.71	113.32	0.00826	19.0	2773.3	247.9	1.00
3.125	52700	95.5	112.75	112.75	119.03	0.00806	20.1	2620.0	208.7	1.00
3.1546	52700	99.3	114.16	114.16	120.45	0.00270	20.1	2617.8	208.4	1.00
3.1547	Bridge									
3.1591	52700	99.3	116.38	114.15	120.91	0.00163	17.1	3084.2	212.6	0.79
3.178	52700	100.2	116.40		121.46	0.00626	18.1	2918.0	227.2	0.89
3.2197	52700	100.6	117.85	117.85	123.69	0.00821	19.4	2715.6	233.3	1.00
3.4091	52700	109.0	126.38		128.97	0.00331	13.0	4159.9	405.7	0.64
3.5038	52700	114.2	127.57	127.57	132.22	0.00871	17.3	3045.1	327.8	1.00
3.5985	52700	118.6	133.44	133.44	138.70	0.00862	18.4	2861.2	274.7	1.01
3.6932	52700	121.5	138.00	137.26	142.60	0.00687	17.2	3060.5	276.5	0.91
3.7879	52700	125.2	141.50		145.97	0.00656	17.0	3105.6	278.7	0.90
3.8826	52700	125.4	145.06	143.65	149.00	0.00540	16.0	3420.9	352.9	0.82
3.9773	52700	136.0	149.63	149.63	154.63	0.00844	17.9	2937.5	294.4	1.00
4.072	52700	138.1	154.92	154.92	160.46	0.00848	18.9	2791.7	252.5	1.00
4.1667	52700	142.6	160.44		163.30	0.00347	13.6	4033.6	369.8	0.67
4.2614	52700	146.4	162.56		165.34	0.00450	13.5	4173.5	478.6	0.74
4.3561	52700	151.1	165.17		168.47	0.00732	14.6	3635.6	482.8	0.90
4.4508	52700	153.7	169.10	169.10	173.25	0.00889	16.4	3251.2	402.0	1.00
4.5455	52700	160.3	175.37	175.37	179.04	0.00598	15.6	4049.0	912.5	0.85
4.6402	46400	161.6	178.86	178.86	183.14	0.00626	16.7	3228.6	724.8	0.88
4.7348	46400	166.4	182.54	182.54	186.59	0.00582	16.3	3444.2	809.6	0.85
4.8295	46400	169.0	185.14	184.35	189.99	0.00684	17.7	2628.3	262.3	0.92
4.9242	46400	171.0	188.20	188.02	193.91	0.00788	19.2	2420.3	206.2	0.99
5.0189	46400	173.4	191.63	191.19	197.86	0.00732	20.0	2316.1	171.0	0.96

Exhibit B4

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
5.1136	46400	174.4	198.21		199.55	0.00126	9.8	6246.3	531.4	0.42
5.303	46400	183.2	199.68		202.28	0.00559	12.9	3586.7	426.2	0.79
5.3977	46400	188.7	204.90	204.90	208.07	0.00979	14.3	3245.3	511.0	1.00
5.4924	46400	196.5	209.74	209.74	214.01	0.00835	16.6	2890.0	389.2	0.97
5.5871	46400	200.8	213.97		217.31	0.00509	14.7	3205.7	343.3	0.79
5.6818	46400	203.2	217.46	217.46	222.59	0.00776	18.3	2699.0	291.0	0.97
5.7765	46400	204.5	220.77	220.24	226.48	0.00713	19.3	2558.1	244.1	0.95
5.8712	46400	206.4	224.82	223.65	229.85	0.00615	18.0	2594.0	228.5	0.88
5.9301	46400	211.2	230.41		231.67	0.00114	9.2	6115.4	660.5	0.40
5.9475	46400	212.6	230.29	224.60	231.91	0.00058	10.2	4541.4	667.1	0.48
5.9476	Bridge									
5.9536	46400	212.6	230.40	224.62	232.00	0.00057	10.1	4578.3	677.0	0.47
5.9742	46400	212.8	230.49		232.11	0.00207	10.3	4944.5	562.3	0.51
6.0606	46400	215.0	231.61		233.22	0.00288	10.2	4718.9	667.5	0.58
6.1553	46400	219.9	233.68		234.72	0.00289	8.2	5836.1	1011.8	0.55
6.3447	46400	230.8	241.53	241.53	244.36	0.01179	14.0	3846.4	792.6	1.07
6.4394	44400	238.5	246.59	246.08	248.95	0.00709	12.4	3804.0	785.3	0.86
6.5341	44400	239.0	250.19	250.19	252.80	0.00817	13.3	4004.1	897.9	0.92
6.6288	44400	244.1	254.10		256.40	0.00621	12.5	4264.0	855.3	0.82
6.7235	44400	245.8	257.51	256.25	259.31	0.00521	10.8	4115.4	619.6	0.74
6.8182	44400	251.6	260.98	260.98	263.39	0.01020	12.5	3657.6	823.8	0.99
6.8389	44400	252.0	262.63	261.60	264.27	0.00555	10.3	4422.2	822.3	0.75
6.9129	44400	252.4	265.04	264.85	267.23	0.00957	11.9	3737.0	766.8	0.95
7.0076	44400	255.4	269.43	268.93	271.58	0.00793	11.8	3766.8	675.2	0.88
7.1023	44400	261.9	274.01	273.86	276.01	0.00986	11.3	3920.9	884.6	0.95
7.197	44400	268.7	279.04	278.74	280.75	0.00904	10.5	4223.0	998.0	0.90
7.2917	44400	275.6	284.58	284.58	285.96	0.00889	9.7	5380.0	2051.6	0.88
7.3864	44400	278.7	288.32	287.96	290.30	0.00758	12.4	5100.2	1050.7	0.88
7.4811	44400	282.3	293.36	293.36	296.21	0.00829	13.7	3610.4	848.2	0.93
7.5758	44400	285.8	297.47	297.41	300.18	0.00754	13.5	4141.7	1098.2	0.90
7.6705	44400	287.1	300.75	299.99	303.90	0.00686	14.2	3159.6	455.0	0.87
7.7652	44400	290.7	304.94	304.94	308.49	0.00946	15.1	2936.3	413.7	1.00
7.8598	44400	296.4	309.56		311.68	0.00424	11.7	3806.1	440.2	0.69
7.9545	44400	302.3	312.76		313.85	0.00404	8.4	5291.3	960.2	0.63
8.0492	19800	307.6	315.22	315.22	317.40	0.01145	12.2	1792.4	426.7	1.02
8.1439	19800	313.7	321.45	321.32	323.22	0.01174	10.8	1929.3	566.8	1.00
8.2386	19800	319.4	327.48	327.48	329.14	0.01153	10.7	2211.3	806.3	0.99
8.3333	19800	328.1	333.70	333.52	335.02	0.01196	9.7	2351.6	832.2	0.98
8.428	19800	331.9	340.29	339.90	341.30	0.01369	9.2	2711.8	890.1	1.02
8.5227	19800	338.9	345.79	345.51	346.95	0.00927	8.7	2433.5	955.5	0.87
8.6174	19800	343.6	352.04	352.04	353.31	0.01269	9.1	2248.0	1023.4	0.99
8.7121	19800	349.7	358.07	357.95	359.31	0.01136	9.0	2249.8	836.5	0.94
8.8068	19800	359.0	365.02	364.49	365.90	0.01473	8.5	3032.0	1404.8	1.03
8.9015	19800	363.3	370.78	369.71	371.27	0.00774	6.7	4051.4	1497.4	0.76
8.9962	19800	371.3	375.31	374.15	375.79	0.01399	6.6	3713.1	1421.6	0.95
9.0909	19800	378.0	383.59	383.03	384.11	0.00780	7.4	4715.0	2114.2	0.78
9.1857	19800	383.9	389.18	388.60	389.74	0.01612	8.8	4062.2	1939.7	1.07
9.2804	19800	389.8	396.45	396.03	397.39	0.01346	10.5	4369.4	1845.4	1.04
9.3264	19800	393.7	399.17	399.17	400.71	0.00970	11.8	4029.0	1462.5	0.95
9.3786	19800	393.7	402.51	402.51	406.31	0.00348	15.7	1265.0	283.2	1.00
9.3787	Bridge									
9.3864	19800	393.7	404.53	402.52	406.92	0.00198	12.4	1599.1	172.8	0.72
9.4009	19800	393.7	406.11	401.86	407.56	0.00309	9.7	2049.3	191.7	0.52
9.4697	19800	398.1	406.98	406.98	410.03	0.01004	14.0	1412.6	230.6	1.00
9.5644	19800	401.2	412.70	412.70	415.39	0.01034	13.2	1504.0	278.2	1.00
9.6591	19800	409.8	418.76	418.76	421.47	0.01037	13.2	1498.9	277.4	1.00
9.7538	19800	415.7	425.45	425.45	427.59	0.01120	11.8	1685.4	394.2	1.00
9.8485	19800	422.1	431.26	430.96	432.76	0.00937	9.9	2008.5	537.7	0.90
9.9432	19800	428.3	437.57	437.57	439.25	0.01197	10.4	1903.8	569.0	1.00
10.0379	19800	434.6	443.76	443.76	445.47	0.01203	10.5	1886.7	554.3	1.00
10.1326	19800	442.9	450.34	450.31	451.63	0.01249	9.1	2169.8	810.3	0.98
10.2273	19800	448.2	455.41	455.25	457.46	0.01011	11.5	1722.0	386.9	0.96
10.322	19800	454.7	463.10	463.10	464.51	0.01693	10.2	2210.6	774.7	1.13
10.4167	19800	465.5	471.55	471.55	472.95	0.01180	9.6	2278.3	918.9	0.97
10.5114	19800	470.8	477.94	477.94	479.30	0.00983	9.9	2652.1	1031.6	0.92
10.6061	19800	472.9	484.15	484.15	485.78	0.01117	10.5	2137.1	766.1	0.97
10.7008	19800	480.7	490.92	490.92	492.56	0.01034	10.5	2178.3	793.3	0.95
10.7955	19800	489.3	497.55	497.55	499.08	0.01267	10.3	2189.0	881.6	1.02
10.8902	19800	496.0	504.23	504.18	505.64	0.01299	11.0	2654.5	900.2	1.04
11.0795	19800	507.2	516.81	516.81	518.64	0.01113	10.9	1877.2	605.8	0.98
11.1742	19800	515.4	523.29	523.29	524.80	0.01174	9.9	2053.3	822.6	0.98
11.2585	19800	521.8	529.02	529.02	530.67	0.00394	10.3	1921.0	579.9	1.00

Exhibit B4

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
11.2586	Bridge									
11.2678	19800	521.8	530.35	529.00	531.13	0.00147	7.1	2793.2	704.6	0.63
11.3636	19800	528.5	537.90	537.90	540.25	0.01043	12.5	1737.0	412.4	0.99
11.4583	19800	538.1	545.10	545.10	546.30	0.01331	8.8	2279.9	968.2	1.00
11.553	19800	546.0	552.90	552.90	554.06	0.01404	8.8	2344.5	1041.8	1.01
11.6477	18800	553.4	560.49	560.49	561.66	0.01329	8.7	2192.4	967.3	0.99
11.7424	18800	560.2	568.14	568.14	569.19	0.01411	8.3	2323.3	1115.9	1.00
11.8371	18800	569.1	575.94	575.94	577.31	0.01592	10.1	2308.2	962.6	1.10
11.9318	18800	574.3	582.76	582.47	583.65	0.01071	8.4	2752.5	924.0	0.91
12.0265	18800	581.1	589.76	589.76	591.38	0.01380	11.8	2339.1	713.6	1.08
12.1212	18800	588.8	597.13	597.13	599.09	0.01115	11.3	1708.9	478.1	0.99
12.2159	18800	598.1	605.28	605.28	606.77	0.01205	10.0	2049.8	738.7	0.99
12.3106	18800	604.4	611.83	611.53	613.12	0.01395	10.2	2272.1	686.2	1.05
12.4053	18800	614.3	619.71	619.00	620.79	0.01357	9.4	2396.5	823.4	1.02
12.5	18800	620.6	627.03	626.45	628.37	0.01345	10.3	2370.0	793.6	1.04
12.5947	18800	623.7	634.17	634.17	636.25	0.01013	11.9	1812.4	541.4	0.97
12.6894	18800	631.6	642.30	642.30	643.58	0.01247	9.2	2217.3	929.4	0.98
12.7841	18800	642.6	650.68	650.68	651.92	0.01307	9.3	2424.9	1098.2	1.00
12.8788	18800	646.8	659.56	659.56	660.98	0.01234	9.9	2199.1	828.0	1.00
12.9735	18800	657.4	667.55	667.55	668.98	0.01279	10.5	2406.7	894.4	1.02
13.0682	18800	665.5	674.98	674.98	676.32	0.01196	10.4	2738.6	1193.2	1.00
13.1629	18800	673.6	682.04	682.04	683.60	0.00942	11.2	2604.0	1069.5	0.93
13.2576	18800	678.5	687.91	687.91	690.58	0.01024	13.1	1445.5	305.4	1.00
13.3523	18800	686.3	700.01	700.01	701.62	0.00892	10.5	2181.5	798.2	0.89
13.447	18800	696.9	706.70	706.70	708.84	0.00948	11.8	1703.0	492.5	0.94
13.5417	18800	707.2	715.78	715.78	717.86	0.01104	11.8	1749.1	522.3	1.00
13.6364	18800	713.7	724.48	724.48	725.71	0.00504	10.1	3678.9	1523.7	0.71
13.7311	18800	721.3	730.68	730.68	732.12	0.00686	10.4	2963.9	1329.4	0.81
13.8258	18800	725.1	735.76	735.76	737.92	0.00816	12.6	2263.7	881.5	0.90
13.9205	18800	731.8	741.68	741.68	744.13	0.00902	13.0	1753.7	390.6	0.94
14.0152	18800	739.5	749.38	749.38	751.67	0.01004	12.6	1779.4	413.1	0.97
14.1098	18800	748.6	758.87	758.87	761.51	0.01086	13.1	1444.1	287.1	1.00
14.1335	18800	748.9	761.66	761.66	764.35	0.01086	13.2	1427.0	270.8	1.00
14.1761	18800	758.8	764.90		766.54	0.00797	10.3	1827.5	403.7	0.85
14.2045	18800	760.2	769.75	769.75	771.86	0.01132	11.7	1614.0	387.2	1.01
14.2992	18800	768.4	775.77	775.77	778.50	0.01030	13.2	1419.8	261.8	1.00
14.3939	18800	772.5	781.94	781.94	785.10	0.00863	14.6	1482.6	262.8	0.96
14.4886	18800	778.6	788.81	788.81	792.24	0.01013	15.1	1411.0	262.0	1.02
14.5833	18800	783.9	795.57	795.57	799.89	0.00907	16.7	1128.0	131.2	1.00
14.678	18800	790.6	802.24	802.24	806.28	0.00795	16.3	1266.2	200.4	0.95
14.7727	18800	796.2	809.54	809.54	813.68	0.00860	16.4	1190.1	172.3	0.98
14.8674	18800	809.2	819.19	819.19	822.36	0.00703	15.1	1774.2	392.5	0.90
14.9621	18800	813.7	824.52	824.52	827.90	0.00873	15.1	1408.6	229.7	0.97
15.0568	18800	820.5	831.33	831.33	834.73	0.00986	14.8	1269.9	189.7	1.01
15.1515	18800	831.8	842.20	842.20	844.76	0.01071	12.8	1472.0	295.8	1.00
15.2462	18800	839.7	851.89	851.89	854.08	0.01123	11.9	1583.1	362.5	1.00
15.3409	18800	851.9	863.67	863.67	865.50	0.01157	10.9	1758.8	496.7	0.99
15.4356	18800	865.2	873.72	873.72	876.95	0.00986	14.4	1303.6	207.0	1.00
15.4979	18800	868.1	879.31	879.31	883.09	0.00795	16.0	1375.3	215.2	0.95
15.5036	18800	867.1	883.97	883.97	888.21	0.00245	20.8	2655.4	330.5	0.91
15.5038	Bridge									
15.5066	18800	867.1	888.24	884.05	889.95	0.00083	14.2	4130.4	360.9	0.55
15.5104	18800	868.5	889.60		890.28	0.00065	7.7	4227.2	344.1	0.31
15.5303	18800	869.8	889.57		890.40	0.00081	7.6	2985.1	243.8	0.33
15.625	18800	879.2	889.01	889.01	892.29	0.00977	14.5	1293.4	197.5	1.00
15.7197	18800	892.8	901.68	901.68	905.21	0.00945	15.1	1254.0	185.9	1.00
15.8144	18800	900.5	913.60	913.60	916.91	0.00911	14.9	1408.0	237.3	0.97
15.9091	15200	908.9	922.39	922.39	926.50	0.00937	16.3	935.0	115.9	1.00
16.0038	15200	925.8	936.22	936.22	940.18	0.00977	16.0	952.5	120.4	1.00
16.0985	15200	936.6	947.80	947.80	951.56	0.00954	15.6	977.0	130.3	1.00
16.1932	15200	949.6	963.68	963.68	966.61	0.01040	13.7	1107.0	191.3	1.01
16.2879	15200	960.4	972.07	972.07	976.63	0.00953	17.1	887.2	97.8	1.00
16.3826	15200	973.6	985.92	985.92	990.42	0.00935	17.0	893.2	99.5	1.00

Exhibit B5
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Standard Table 1
50-yr event

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
-0.2258	67900	-18.0	2.53	-8.77	2.67	0.00014	3.0	22673.4	1479.0	0.13
-0.1784	67900	-14.3	2.45		2.79	0.00050	4.6	14702.5	1342.0	0.25
-0.1311	67900	-10.5	2.37		3.13	0.00172	7.0	9703.1	1204.0	0.43
-0.0837	67900	-4.7	2.03	2.03	4.55	0.01059	12.7	5328.5	1059.0	1.00
-0.0364	67900	-1.8	4.53	4.53	7.28	0.01031	13.3	5100.5	929.0	1.00
0.0477	67900	2.5	10.15	10.15	13.02	0.00896	13.6	5226.5	1118.1	0.96
0.1052	67900	-0.3	13.44	10.24	14.05	0.00143	7.3	15872.1	2479.6	0.41
0.1383	67900	1.0	13.64	10.16	14.29	0.00149	7.6	15615.3	2382.8	0.42
0.1909	67900	-1.3	13.65	10.41	14.80	0.00075	9.2	13477.3	2633.5	0.52
0.191	Mult Open									
0.1945	67900	-1.3	14.08	10.42	15.11	0.00064	8.7	14639.9	2754.8	0.48
0.3579	67900	3.6	15.53	15.53	18.26	0.00975	13.3	5266.4	2596.0	0.98
0.4394	67900	0.0	18.02	18.02	20.82	0.00332	13.4	5058.3	2357.5	1.00
0.4395	Mult Open									
0.464	67900	0.0	21.18	17.98	22.32	0.00075	8.6	7927.4	2844.2	0.51
0.5204	67900	5.6	21.18	21.07	23.19	0.00472	12.6	9799.2	2626.0	0.74
0.5922	67900	6.2	23.88	23.88	26.21	0.00351	12.3	5537.4	2560.5	1.00
0.5923	Mult Open									
0.6028	67900	6.2	25.61	23.76	26.85	0.00124	8.9	7626.7	2774.9	0.63
0.6629	67900	11.8	25.96	25.94	27.85	0.00506	13.0	10861.2	2820.1	0.76
0.7577	67900	15.5	28.94	28.94	30.89	0.00524	14.2	11212.9	2529.3	0.79
0.8523	67900	17.6	32.66	32.66	35.30	0.00405	14.4	8843.3	2193.7	0.72
0.947	67900	19.3	35.30	35.30	38.55	0.00514	16.0	7218.7	1335.3	0.81
1.0417	67900	20.7	37.50	34.99	40.67	0.00357	14.9	5803.7	685.9	0.69
1.1364	67900	23.0	40.47	40.47	45.67	0.00607	19.6	4948.8	564.0	0.90
1.2311	67900	25.3	45.48	40.70	47.70	0.00222	12.2	6342.6	686.5	0.55
1.3258	67900	30.2	46.26	43.81	49.29	0.00334	14.4	5840.4	618.1	0.67
1.4205	67900	30.6	49.11	43.63	50.55	0.00157	9.8	8370.3	987.1	0.46
1.5152	67900	36.1	50.26	46.41	51.47	0.00206	8.9	7684.6	787.8	0.50
1.6098	67900	36.9	50.45	50.45	54.27	0.00829	15.8	4625.8	663.9	0.97
1.7045	67900	41.8	55.07	55.07	59.16	0.00766	18.0	5511.9	687.2	0.97
1.7992	67900	43.2	60.35	60.35	65.70	0.00709	19.3	4278.4	420.7	0.95
1.8939	67900	43.8	65.79	60.72	67.81	0.00212	11.9	6862.5	648.5	0.54
1.9886	67900	50.2	67.15	63.70	69.00	0.00268	11.3	7054.6	706.8	0.58
2.0827	67900	53.4	68.47	67.05	70.78	0.00416	12.9	7085.8	1041.5	0.71
2.178	67900	59.1	70.87	70.54	73.60	0.00682	14.6	6754.6	1119.5	0.88
2.2727	67900	62.2	74.71	72.25	75.79	0.00277	8.4	8223.6	1204.4	0.55
2.3674	67900	68.0	76.62	76.62	78.84	0.01089	11.9	5685.2	1281.8	1.00
2.4621	67900	70.6	81.70	81.26	83.51	0.00798	10.8	6294.8	1309.1	0.87
2.5568	67900	72.7	88.62	88.62	93.11	0.00739	17.8	4762.9	556.3	0.95
2.6515	67900	79.3	94.89	94.89	100.08	0.00719	19.2	4436.1	450.2	0.96
2.7462	67900	81.7	98.12	98.12	104.20	0.00708	20.0	3672.4	338.7	0.96
2.8409	67900	82.5	103.71	100.44	106.95	0.00364	14.5	4699.4	341.5	0.69
2.9356	67900	84.4	105.01		109.50	0.00529	17.1	4148.1	382.4	0.83
3.0303	67900	91.7	109.93	109.93	116.36	0.00791	20.4	3335.8	259.6	1.00
3.125	67900	95.5	115.23	115.23	122.44	0.00774	21.6	3150.1	218.7	1.00
3.1546	67900	99.3	116.55	116.55	123.90	0.00260	21.8	3121.9	213.0	1.00
3.1547	Bridge									
3.1591	67900	99.3	119.25	116.53	124.46	0.00154	18.3	3703.9	219.3	0.79
3.178	67900	100.2	119.39		124.85	0.00554	18.8	3621.6	242.5	0.85
3.2197	67900	100.6	120.14	120.14	126.86	0.00791	20.8	3264.2	245.1	1.00
3.4091	67900	109.0	128.91	124.80	131.81	0.00297	13.7	5248.8	451.1	0.63
3.5038	67900	114.2	129.58	129.34	134.77	0.00778	18.3	3715.9	337.8	0.97
3.5985	67900	118.6	135.51	135.51	141.54	0.00824	19.7	3443.4	287.8	1.00
3.6932	67900	121.5	139.84	139.36	145.43	0.00716	19.0	3581.0	288.4	0.95
3.7879	67900	125.2	143.69		148.83	0.00632	18.2	3731.6	293.1	0.90
3.8826	67900	125.4	147.28	145.71	151.75	0.00517	17.1	4238.8	381.7	0.82
3.9773	67900	136.0	151.83	151.83	157.24	0.00781	18.7	3702.1	384.5	0.98
4.072	67900	138.1	157.16	157.16	163.44	0.00817	20.1	3376.1	269.2	1.00
4.1667	67900	142.6	162.97		166.19	0.00326	14.5	4994.5	390.2	0.66
4.2614	67900	146.4	165.16		168.00	0.00345	13.7	5492.8	621.9	0.67
4.3561	67900	151.1	167.13		170.62	0.00601	15.0	4600.0	515.2	0.85
4.4508	67900	153.7	170.74	170.74	175.51	0.00831	17.6	3962.4	572.0	0.99
4.5455	67900	160.3	177.11	177.11	180.91	0.00555	16.2	5663.3	944.9	0.83
4.6402	59700	161.6	181.24	181.24	185.01	0.00479	16.4	5432.4	961.1	0.79
4.7348	59700	166.4	184.63	184.63	188.47	0.00491	16.5	5314.5	945.8	0.80
4.8295	59700	169.0	187.68	187.68	192.67	0.00606	18.1	3826.9	605.7	0.88
4.9242	59700	171.0	190.67	190.67	196.99	0.00776	20.2	2971.8	278.0	0.99
5.0189	59700	173.4	193.93	193.93	201.40	0.00773	21.9	2723.1	182.6	1.00

Exhibit B5

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
5.1136	59700	174.4	201.68		203.11	0.00111	10.3	8246.0	664.9	0.40
5.303	59700	183.2	202.93		205.14	0.00319	12.0	5129.2	692.4	0.62
5.3977	59700	188.7	206.20	206.20	209.80	0.00885	15.2	3975.5	775.5	0.98
5.4924	59700	196.5	211.45	211.45	216.30	0.00783	17.8	3577.9	413.9	0.97
5.5871	59700	200.8	215.32	214.17	219.65	0.00565	16.7	3713.8	401.9	0.85
5.6818	59700	203.2	219.48	219.48	225.35	0.00729	19.7	3317.5	334.6	0.97
5.7765	59700	204.5	223.19	223.19	229.64	0.00671	20.6	3218.2	292.0	0.94
5.8712	59700	206.4	226.73	226.23	232.96	0.00655	20.1	3126.6	402.8	0.93
5.9301	59700	211.2	233.80		235.04	0.00091	9.3	8386.9	680.9	0.37
5.9475	59700	212.6	233.69	226.30	235.25	0.00044	10.2	7889.9	927.7	0.43
5.9476	Bridge									
5.9536	59700	212.6	233.98	226.30	235.48	0.00044	10.3	7115.5	939.7	0.43
5.9742	59700	212.8	234.28		235.71	0.00133	9.9	7131.6	585.3	0.43
6.0606	59700	215.0	235.06		236.39	0.00162	9.4	7134.7	713.3	0.46
6.1553	59700	219.9	236.38		237.22	0.00153	7.4	8635.4	1062.7	0.42
6.3447	59700	230.8	242.58	242.58	245.85	0.01089	15.1	4845.6	1149.4	1.05
6.4394	56600	238.5	247.52	247.20	250.36	0.00729	13.7	4561.2	843.4	0.89
6.5341	56600	239.0	251.16	251.12	254.15	0.00794	14.4	4893.1	927.6	0.93
6.6288	56600	244.1	255.02		257.80	0.00650	13.8	5065.3	885.2	0.85
6.7235	56600	245.8	258.62	257.22	260.77	0.00512	11.8	4806.3	625.3	0.75
6.8182	56600	251.6	261.87	261.87	264.65	0.00954	13.4	4414.5	886.5	0.98
6.8389	56600	252.0	263.54	262.48	265.51	0.00560	11.3	5177.5	854.7	0.77
6.9129	56600	252.4	265.88	265.64	268.47	0.00925	12.9	4383.3	774.1	0.96
7.0076	56600	255.4	270.22	269.83	272.90	0.00840	13.1	4307.0	684.1	0.92
7.1023	56600	261.9	274.96	274.58	277.16	0.00844	11.9	4760.7	889.2	0.90
7.197	56600	268.7	279.55	279.45	281.77	0.01007	12.0	4736.2	1001.3	0.97
7.2917	56600	275.6	285.10	285.08	286.75	0.00931	10.7	6509.0	2221.4	0.92
7.3864	56600	278.7	289.03	289.02	291.34	0.00802	13.7	6442.0	1670.6	0.92
7.4811	56600	282.3	294.62	294.62	297.64	0.00726	14.3	4794.4	990.9	0.90
7.5758	56600	285.8	298.67	298.67	301.46	0.00670	14.1	5487.7	1140.8	0.86
7.6705	56600	287.1	301.68	301.68	305.75	0.00782	16.3	3668.3	674.7	0.95
7.7652	56600	290.7	306.31	306.31	310.34	0.00876	16.1	3543.4	516.4	0.99
7.8598	56600	296.4	310.88		313.48	0.00434	12.9	4392.0	446.6	0.72
7.9545	56600	302.3	314.33		315.40	0.00292	8.3	6816.5	985.5	0.56
8.0492	24800	307.6	315.97	315.97	318.45	0.01072	13.0	2128.7	487.4	1.01
8.1439	24800	313.7	322.16	322.16	324.06	0.01084	11.2	2391.8	713.8	0.98
8.2386	24800	319.4	328.10	328.10	329.91	0.01040	11.3	2743.7	930.3	0.96
8.3333	24800	328.1	334.08	334.07	335.73	0.01307	10.8	2692.8	937.9	1.04
8.428	24800	331.9	340.86	340.30	341.98	0.01229	9.7	3232.6	931.0	0.99
8.5227	24800	338.9	346.18	346.04	347.59	0.00989	9.7	2826.7	1058.1	0.91
8.6174	24800	343.6	352.51	352.51	353.91	0.01180	9.6	2749.6	1100.2	0.97
8.7121	24800	349.7	358.57	358.54	359.93	0.01228	9.4	2693.3	978.6	0.98
8.8068	24800	359.0	365.53	364.94	366.41	0.01308	8.5	3795.5	1590.1	0.98
8.9015	24800	363.3	371.13	370.14	371.73	0.00836	7.3	4592.2	1558.4	0.80
8.9962	24800	371.3	375.81	374.61	376.36	0.01319	7.4	4461.9	1569.3	0.95
9.0909	24800	378.0	383.94	383.32	384.54	0.00812	7.9	5479.4	2181.7	0.81
9.1857	24800	383.9	389.59	389.18	390.17	0.01524	8.9	4887.4	2065.8	1.05
9.2804	24800	389.8	396.83	396.72	398.03	0.01469	11.8	5099.6	1977.4	1.11
9.3264	24800	393.7	399.84	399.84	401.42	0.00912	12.3	5070.1	1656.0	0.94
9.3786	24800	393.7	403.81	403.81	408.14	0.00338	16.7	1484.9	438.8	1.00
9.3787	Bridge									
9.3864	24800	393.7	406.47	403.82	409.01	0.00178	12.8	1940.1	178.3	0.68
9.4009	24800	393.7	407.99	403.15	409.62	0.00306	10.3	2417.9	200.9	0.52
9.4697	24800	398.1	408.72	408.00	411.60	0.00710	13.6	1820.8	238.7	0.87
9.5644	24800	401.2	413.58	413.58	416.70	0.00987	14.2	1750.5	280.6	1.00
9.6591	24800	409.8	419.66	419.66	422.78	0.00988	14.2	1751.0	281.9	1.00
9.7538	24800	415.7	426.16	426.16	428.63	0.01059	12.6	1969.0	397.6	1.00
9.8485	24800	422.1	431.92	431.58	433.61	0.00912	10.5	2374.0	570.8	0.90
9.9432	24800	428.3	438.14	438.14	440.07	0.01144	11.1	2236.2	595.0	1.00
10.0379	24800	434.6	444.40	444.40	446.26	0.01154	11.0	2261.1	602.6	1.00
10.1326	24800	442.9	450.74	450.72	452.28	0.01240	10.0	2493.1	813.6	1.00
10.2273	24800	448.2	456.34	456.34	458.48	0.01097	11.8	2111.7	509.7	1.00
10.322	24800	454.7	463.72	463.72	465.17	0.01549	10.3	2722.3	887.0	1.10
10.4167	24800	465.5	472.09	472.09	473.60	0.01121	10.1	2809.2	1050.0	0.97
10.5114	24800	470.8	478.40	478.40	479.93	0.01060	10.7	3157.4	1132.4	0.96
10.6061	24800	472.9	484.70	484.70	486.56	0.01069	11.2	2579.6	869.5	0.97
10.7008	24800	480.7	491.51	491.51	493.33	0.00967	11.1	2671.3	869.9	0.94
10.7955	24800	489.3	498.13	498.13	499.78	0.01144	10.7	2733.2	971.5	0.99
10.8902	24800	496.0	504.74	504.74	506.30	0.01324	11.7	3134.0	977.6	1.07
11.0795	24800	507.2	517.49	517.49	519.50	0.01051	11.5	2310.2	665.9	0.97
11.1742	24800	515.4	523.93	523.93	525.49	0.01060	10.1	2654.8	1007.8	0.95
11.2585	24800	521.8	529.66	529.66	531.44	0.00385	10.7	2319.4	652.5	1.00

Exhibit B5

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
11.2586	Bridge									
11.2678	24800	521.8	530.62	529.65	531.70	0.00189	8.3	2983.8	716.2	0.72
11.3636	24800	528.5	538.76	538.76	541.36	0.00959	13.3	2129.3	510.4	0.97
11.4583	24800	538.1	545.56	545.51	546.88	0.01209	9.3	2737.1	1023.4	0.97
11.553	24800	546.0	553.33	553.33	554.61	0.01358	9.2	2825.9	1154.0	1.01
11.6477	24000	553.4	560.99	560.99	562.26	0.01310	9.1	2709.1	1116.3	1.00
11.7424	24000	560.2	568.57	568.57	569.72	0.01475	8.7	2851.1	1341.0	1.03
11.8371	24000	569.1	576.72	576.72	577.98	0.01258	9.7	3133.9	1263.3	1.00
11.9318	24000	574.3	583.07	582.82	584.24	0.01315	9.6	3043.8	966.6	1.01
12.0265	24000	581.1	590.71	590.71	592.22	0.01168	11.5	3077.4	839.4	1.01
12.1212	24000	588.8	598.12	598.12	600.04	0.01086	11.2	2247.7	624.4	0.98
12.2159	24000	598.1	605.92	605.92	607.50	0.01197	10.3	2554.8	837.9	1.00
12.3106	24000	604.4	612.36	612.14	613.91	0.01434	11.2	2642.3	719.7	1.09
12.4053	24000	614.3	620.32	619.84	621.56	0.01327	10.2	2927.6	921.3	1.03
12.5	24000	620.6	627.49	627.42	629.14	0.01372	11.4	2734.2	798.5	1.08
12.5947	24000	623.7	635.07	635.07	637.30	0.00904	12.5	2371.3	663.5	0.94
12.6894	24000	631.6	642.78	642.78	644.24	0.01206	9.9	2688.0	1000.4	0.99
12.7841	24000	642.6	651.24	651.24	652.53	0.01196	9.6	3073.8	1228.1	0.98
12.8788	24000	646.8	660.10	660.10	661.70	0.01245	10.6	2662.5	907.4	1.01
12.9735	24000	657.4	668.13	668.13	669.70	0.01280	11.2	2959.1	1052.9	1.03
13.0682	24000	665.5	675.65	675.65	676.93	0.01082	10.4	3588.8	1351.4	0.96
13.1629	24000	673.6	682.70	682.70	684.37	0.00914	11.9	3351.3	1194.3	0.93
13.2576	24000	678.5	689.03	689.03	691.94	0.00917	13.8	1823.8	377.1	0.97
13.3523	24000	686.3	700.73	700.73	702.37	0.00996	10.8	2871.1	1107.9	0.94
13.447	24000	696.9	707.68	707.68	709.87	0.00928	12.1	2283.7	683.0	0.94
13.5417	24000	707.2	716.71	716.71	718.91	0.00924	12.3	2321.1	725.9	0.94
13.6364	24000	713.7	724.98	724.98	726.32	0.00593	10.9	4464.6	1593.1	0.77
13.7311	24000	721.3	731.27	731.27	732.83	0.00689	11.2	3782.5	1425.0	0.82
13.8258	24000	725.1	737.17	737.17	738.80	0.00594	11.5	3756.0	1210.4	0.78
13.9205	24000	731.8	743.66	743.66	745.52	0.00738	11.5	2864.9	843.3	0.85
14.0152	24000	739.5	750.25	750.25	752.86	0.00975	13.5	2147.7	430.9	0.98
14.1098	24000	748.6	760.41	760.41	762.74	0.00768	12.4	2282.1	715.9	0.87
14.1335	24000	748.9	763.10	763.10	765.63	0.00947	12.8	2009.0	543.6	0.94
14.1761	24000	758.8	765.71		767.63	0.00776	11.1	2161.8	416.4	0.86
14.2045	24000	760.2	770.50	770.50	772.96	0.01074	12.6	1906.4	391.1	1.00
14.2992	24000	768.4	776.76	776.76	779.93	0.00979	14.3	1680.8	266.0	1.00
14.3939	24000	772.5	783.10	783.10	786.75	0.00856	15.8	1800.9	281.9	0.97
14.4886	24000	778.6	790.32	790.32	793.82	0.00848	15.4	1862.9	315.2	0.96
14.5833	24000	783.9	797.25	797.25	802.12	0.00877	17.7	1355.5	140.1	1.00
14.678	24000	790.6	803.89	803.89	808.35	0.00716	17.2	1628.2	239.0	0.93
14.7727	24000	796.2	811.54	811.54	815.73	0.00696	16.6	1656.6	291.6	0.91
14.8674	24000	809.2	820.72	820.72	823.85	0.00596	15.5	2532.5	569.3	0.85
14.9621	24000	813.7	825.88	825.88	829.63	0.00845	16.0	1736.0	252.3	0.97
15.0568	24000	820.5	832.62	832.62	836.49	0.00932	15.8	1519.0	196.9	1.00
15.1515	24000	831.8	843.27	843.27	846.05	0.01036	13.4	1807.3	333.9	1.00
15.2462	24000	839.7	852.74	852.74	855.22	0.01086	12.6	1904.2	407.9	1.00
15.3409	24000	851.9	864.42	864.42	866.42	0.01131	11.4	2157.6	555.7	1.00
15.4356	24000	865.2	875.00	875.00	878.62	0.00943	15.3	1580.4	227.6	1.00
15.4979	24000	868.1	880.82	880.82	885.03	0.00731	17.0	1718.1	238.8	0.94
15.5036	24000	867.1	885.88	885.88	890.34	0.00238	22.1	3309.5	350.3	0.91
15.5038	Bridge									
15.5066	24000	867.1	890.27	885.94	892.23	0.00088	15.6	4873.9	370.0	0.58
15.5104	24000	868.5	891.79		892.59	0.00069	8.5	5037.2	399.5	0.32
15.5303	24000	869.8	891.73		892.74	0.00084	8.4	3522.5	253.7	0.35
15.625	24000	879.2	891.29		894.20	0.00609	13.7	1755.4	207.6	0.82
15.7197	24000	892.8	902.99	902.99	907.05	0.00887	16.2	1503.6	195.8	0.99
15.8144	24000	900.5	914.82	914.82	918.65	0.00874	16.1	1713.0	260.0	0.98
15.9091	18800	908.9	923.69	923.69	928.36	0.00895	17.3	1088.6	120.0	1.00
16.0038	18800	925.8	937.47	937.47	941.97	0.00947	17.0	1103.4	122.6	1.00
16.0985	18800	936.6	949.10	949.10	953.21	0.00929	16.3	1156.7	142.9	1.00
16.1932	18800	949.6	964.63	964.63	967.91	0.00986	14.5	1298.8	208.1	1.00
16.2879	18800	960.4	973.57	973.57	978.67	0.00932	18.1	1037.9	103.0	1.01
16.3826	18800	973.6	987.40	987.40	992.43	0.00912	18.0	1044.6	104.4	1.00

Exhibit B6
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Standard Table 1
100-year event

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
-0.2258	78900	-18.0	2.53	-8.36	2.72	0.00018	3.5	22673.4	1479.0	0.16
-0.1784	78900	-14.3	2.43		2.88	0.00068	5.4	14665.1	1342.0	0.29
-0.1311	78900	-10.5	2.30		3.34	0.00238	8.2	9622.0	1204.0	0.51
-0.0837	78900	-4.7	2.54	2.54	5.34	0.01036	13.4	5871.0	1059.0	1.01
-0.0364	78900	-1.8	5.11	5.11	8.15	0.00997	14.0	5641.6	929.0	1.00
0.0477	78900	2.5	10.88	10.88	13.91	0.00821	14.1	6120.4	1359.3	0.94
0.1052	78900	-0.3	14.26	10.74	14.90	0.00138	7.6	17914.1	2483.1	0.41
0.1383	78900	1.0	14.45	10.47	15.13	0.00145	7.9	17548.2	2386.4	0.42
0.1909	78900	-1.3	14.42	10.86	15.68	0.00076	9.7	15598.4	2858.4	0.53
0.191	Mult Open									
0.1945	78900	-1.3	15.05	10.81	16.13	0.00061	9.0	17472.6	3057.5	0.48
0.3579	78900	3.6	16.16	16.16	19.11	0.00924	13.9	5947.0	3256.1	0.97
0.4394	78900	0.0	18.61	18.61	21.70	0.00321	14.1	5594.4	2416.1	1.00
0.4395	Mult Open									
0.464	78900	0.0	22.02	18.59	23.30	0.00075	9.1	8692.5	2907.2	0.52
0.5204	78900	5.6	22.19	21.60	23.92	0.00381	12.2	12503.3	2756.0	0.67
0.5922	78900	6.2	24.39	24.39	26.95	0.00340	12.8	6148.6	2637.5	1.00
0.5923	Mult Open									
0.6028	78900	6.2	26.26	24.36	27.63	0.00122	9.4	8403.6	2805.5	0.63
0.6629	78900	11.8	26.69	26.40	28.46	0.00455	13.0	12926.2	2826.1	0.73
0.7577	78900	15.5	29.39	29.39	31.50	0.00556	15.1	12351.0	2531.5	0.82
0.8523	78900	17.6	33.44	33.44	36.10	0.00398	14.8	10585.5	2253.7	0.72
0.947	78900	19.3	36.09	36.09	39.56	0.00522	16.8	8289.6	1380.4	0.82
1.0417	78900	20.7	38.09	36.13	42.01	0.00420	16.6	6252.8	807.4	0.76
1.1364	78900	23.0	41.69	41.69	47.34	0.00603	20.6	5692.0	665.1	0.91
1.2311	78900	25.3	46.95	41.78	49.35	0.00215	12.8	7589.0	948.8	0.55
1.3258	78900	30.2	47.67	44.85	50.89	0.00318	15.0	6741.5	656.1	0.67
1.4205	78900	30.6	50.61	44.51	52.10	0.00145	10.1	9857.5	999.8	0.45
1.5152	78900	36.1	51.67	47.09	52.93	0.00178	9.0	8852.7	901.8	0.47
1.6098	78900	36.9	51.29	51.29	55.47	0.00803	16.6	5188.8	693.8	0.97
1.7045	78900	41.8	55.96	55.96	60.44	0.00766	19.0	6125.1	692.1	0.98
1.7992	78900	43.2	61.75	61.75	67.37	0.00663	19.9	4980.8	643.3	0.93
1.8939	78900	43.8	67.22	61.80	69.43	0.00210	12.5	7810.4	677.3	0.54
1.9886	78900	50.2	68.67	64.56	70.57	0.00240	11.5	8147.1	731.0	0.56
2.0827	78900	53.4	69.88	67.71	72.08	0.00342	12.7	8566.5	1052.5	0.66
2.178	78900	59.1	71.78	71.22	74.59	0.00624	14.9	7781.0	1131.5	0.86
2.2727	78900	62.2	75.48	72.79	76.67	0.00268	8.8	9153.4	1213.4	0.55
2.3674	78900	68.0	77.10	77.10	79.54	0.01052	12.5	6294.4	1286.5	1.00
2.4621	78900	70.6	82.13	81.71	84.19	0.00815	11.5	6851.5	1312.9	0.89
2.5568	78900	72.7	89.61	89.61	94.52	0.00736	18.7	5314.3	562.6	0.96
2.6515	78900	79.3	96.07	96.07	101.70	0.00710	20.1	4973.1	457.6	0.96
2.7462	78900	81.7	99.46	99.46	106.11	0.00685	20.9	4132.8	346.4	0.96
2.8409	78900	82.5	105.28	101.69	108.80	0.00350	15.1	5239.1	346.6	0.68
2.9356	78900	84.4	106.43		111.30	0.00509	17.8	4703.2	397.8	0.82
3.0303	78900	91.7	111.39	111.39	118.37	0.00773	21.2	3720.4	267.3	1.00
3.125	78900	95.5	116.87	116.87	124.70	0.00758	22.5	3513.5	225.2	1.00
3.1546	78900	99.3	118.18	118.18	126.20	0.00254	22.7	3471.8	216.8	1.00
3.1547	Bridge									
3.1591	78900	99.3	121.18	118.17	126.84	0.00150	19.1	4131.7	223.8	0.78
3.178	78900	100.2	121.42		127.10	0.00513	19.1	4123.8	252.6	0.83
3.2197	78900	100.6	121.67	121.65	128.95	0.00773	21.7	3644.3	253.0	1.01
3.4091	78900	109.0	130.63	126.21	133.70	0.00276	14.2	6060.1	485.9	0.61
3.5038	78900	114.2	131.09	130.54	136.49	0.00703	18.6	4232.4	345.4	0.94
3.5985	78900	118.6	136.88	136.88	143.42	0.00804	20.5	3843.8	296.4	1.00
3.6932	78900	121.5	141.03	140.75	147.30	0.00738	20.1	3926.6	296.1	0.97
3.7879	78900	125.2	145.17		150.72	0.00614	18.9	4172.4	302.8	0.90
3.8826	78900	125.4	148.73	147.17	153.55	0.00495	17.8	4822.7	433.4	0.82
3.9773	78900	136.0	153.12	153.12	158.92	0.00750	19.4	4208.2	401.6	0.98
4.072	78900	138.1	158.64	158.64	165.40	0.00799	20.9	3781.9	280.3	1.00
4.1667	78900	142.6	164.64		168.09	0.00315	15.1	5658.2	403.7	0.66
4.2614	78900	146.4	166.96		169.78	0.00292	13.8	6835.1	777.0	0.63
4.3561	78900	151.1	168.59	167.03	172.12	0.00517	15.1	5446.6	726.4	0.80
4.4508	78900	153.7	172.68	172.68	176.83	0.00605	16.6	5649.9	1048.7	0.87
4.5455	78900	160.3	178.06	178.06	182.10	0.00559	17.0	6574.8	984.7	0.84
4.6402	69700	161.6	182.20	182.20	186.20	0.00480	17.2	6358.7	969.2	0.80
4.7348	69700	166.4	185.68	185.68	189.68	0.00481	17.2	6338.3	995.8	0.80
4.8295	69700	169.0	189.19	189.19	194.20	0.00560	18.4	4789.9	661.4	0.86
4.9242	69700	171.0	192.40	192.40	198.97	0.00702	20.6	3516.7	369.4	0.96
5.0189	69700	173.4	195.78	195.78	203.79	0.00757	22.7	3069.7	191.9	1.00

Exhibit B6

River Sta	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
	(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
5.1136	69700	174.4	204.06		205.45	0.00099	10.3	9890.4	736.0	0.39
5.303	69700	183.2	205.14		207.13	0.00235	11.5	6834.5	799.3	0.55
5.3977	69700	188.7	207.22	207.22	210.91	0.00776	15.5	4852.5	872.3	0.94
5.4924	69700	196.5	212.61	212.61	217.86	0.00759	18.6	4070.9	444.5	0.96
5.5871	69700	200.8	216.16	215.49	221.26	0.00611	18.2	4054.6	407.7	0.89
5.6818	69700	203.2	220.88	220.88	227.24	0.00692	20.6	3797.0	350.0	0.96
5.7765	69700	204.5	224.46	224.46	231.72	0.00697	22.0	3598.4	305.7	0.97
5.8712	69700	206.4	228.91	228.58	234.92	0.00554	19.9	4056.1	436.7	0.87
5.9301	69700	211.2	235.52		236.87	0.00090	9.8	9570.9	692.8	0.37
5.9475	69700	212.6	235.41	227.44	237.07	0.00043	10.6	9551.3	998.0	0.43
5.9476	Bridge									
5.9536	69700	212.6	235.78	227.49	237.36	0.00042	10.7	8876.9	1007.5	0.43
5.9742	69700	212.8	236.11		237.62	0.00123	10.2	8205.2	591.8	0.42
6.0606	69700	215.0	236.90		238.24	0.00138	9.5	8453.2	718.9	0.43
6.1553	69700	219.9	238.12		238.92	0.00117	7.3	10519.8	1106.8	0.38
6.3447	69700	230.8	243.84	243.84	246.71	0.00783	14.3	6396.9	1302.3	0.92
6.4394	66600	238.5	248.04	248.04	251.39	0.00795	14.9	5011.5	878.7	0.94
6.5341	66600	239.0	251.93	251.43	255.34	0.00797	15.4	5703.1	1288.8	0.95
6.6288	66600	244.1	255.83	255.33	259.03	0.00661	14.8	5898.0	1241.2	0.87
6.7235	66600	245.8	259.60	257.96	261.94	0.00480	12.3	5425.3	646.0	0.74
6.8182	66600	251.6	262.56	262.56	265.59	0.00906	14.1	5049.9	941.5	0.97
6.8389	66600	252.0	264.16	263.15	266.45	0.00576	12.2	5731.0	908.2	0.79
6.9129	66600	252.4	266.53	266.31	269.41	0.00903	13.6	4883.0	779.6	0.96
7.0076	66600	255.4	270.82	270.54	273.91	0.00872	14.1	4715.3	690.7	0.95
7.1023	66600	261.9	275.71	275.18	278.05	0.00761	12.3	5425.2	892.9	0.88
7.197	66600	268.7	279.99	279.99	282.56	0.01043	12.9	5172.6	1004.0	1.00
7.2917	66600	275.6	285.68	285.39	287.36	0.00816	10.8	7820.1	2266.7	0.87
7.3864	66600	278.7	289.52	289.52	292.16	0.00856	14.8	7291.8	1807.1	0.96
7.4811	66600	282.3	295.41	295.41	298.66	0.00703	14.9	5595.3	1017.8	0.90
7.5758	66600	285.8	299.38	299.38	302.40	0.00669	14.8	6300.8	1148.5	0.88
7.6705	66600	287.1	303.05	303.05	307.09	0.00661	16.3	4718.5	822.9	0.89
7.7652	66600	290.7	307.50	307.50	311.05	0.00689	15.6	5220.0	1061.7	0.89
7.8598	66600	296.4	310.71		314.44	0.00637	15.5	4315.2	445.8	0.87
7.9545	66600	302.3	315.49		316.57	0.00246	8.4	7972.9	1006.9	0.52
8.0492	28300	307.6	316.50	316.50	319.12	0.01004	13.4	2394.4	515.8	0.99
8.1439	28300	313.7	322.58	322.58	324.58	0.01017	11.5	2716.1	788.5	0.96
8.2386	28300	319.4	328.54	328.54	330.39	0.00948	11.5	3166.0	989.2	0.94
8.3333	28300	328.1	334.38	334.38	336.18	0.01313	11.4	2975.0	984.4	1.06
8.428	28300	331.9	341.16	340.53	342.41	0.01213	10.2	3508.8	948.3	1.00
8.5227	28300	338.9	346.46	346.36	348.00	0.00997	10.1	3132.6	1125.1	0.93
8.6174	28300	343.6	352.74	352.74	354.29	0.01220	10.1	3014.4	1175.4	1.00
8.7121	28300	349.7	358.92	358.85	360.30	0.01177	9.5	3049.4	1054.1	0.97
8.8068	28300	359.0	365.75	365.45	366.71	0.01337	8.9	4146.0	1643.6	1.00
8.9015	28300	363.3	371.45	370.26	372.06	0.00832	7.3	5104.4	1664.1	0.80
8.9962	28300	371.3	376.08	374.86	376.68	0.01318	7.9	4889.4	1642.2	0.97
9.0909	28300	378.0	384.18	383.59	384.81	0.00815	8.2	6007.2	2221.4	0.82
9.1857	28300	383.9	389.82	389.28	390.43	0.01506	9.1	5370.8	2168.4	1.05
9.2804	28300	389.8	397.07	396.98	398.39	0.01514	12.4	5581.0	2102.5	1.14
9.3264	28300	393.7	400.19	400.19	401.85	0.00919	12.8	5673.7	1803.8	0.95
9.3786	28300	393.7	404.66	404.66	409.33	0.00333	17.3	1632.4	551.0	1.00
9.3787	Bridge									
9.3864	28300	393.7	407.11	404.67	410.06	0.00196	13.8	2054.9	179.0	0.72
9.4009	28300	393.7	408.95	403.98	410.77	0.00324	10.8	2613.5	207.4	0.54
9.4697	28300	398.1	409.77	408.66	412.66	0.00617	13.7	2072.8	243.5	0.82
9.5644	28300	401.2	414.17	414.17	417.56	0.00960	14.8	1914.8	282.2	1.00
9.6591	28300	409.8	420.26	420.26	423.64	0.00958	14.8	1920.4	284.9	1.00
9.7538	28300	415.7	426.63	426.63	429.31	0.01034	13.1	2153.4	400.4	1.00
9.8485	28300	422.1	432.37	432.00	434.16	0.00880	10.7	2641.4	594.9	0.90
9.9432	28300	428.3	438.51	438.51	440.59	0.01105	11.6	2457.5	606.0	0.99
10.0379	28300	434.6	444.74	444.74	446.78	0.01133	11.5	2470.9	608.5	1.00
10.1326	28300	442.9	451.03	451.03	452.70	0.01199	10.4	2728.7	816.0	1.00
10.2273	28300	448.2	456.75	456.75	459.07	0.01076	12.2	2331.8	553.4	1.00
10.322	28300	454.7	464.04	464.04	465.56	0.01599	10.5	3018.9	974.4	1.12
10.4167	28300	465.5	472.42	472.42	474.02	0.01084	10.4	3165.4	1119.7	0.96
10.5114	28300	470.8	478.71	478.71	480.35	0.01041	11.1	3516.5	1172.0	0.96
10.6061	28300	472.9	485.02	485.02	487.06	0.01075	11.8	2869.5	949.6	0.99
10.7008	28300	480.7	491.87	491.87	493.82	0.00947	11.6	2993.6	934.4	0.94
10.7955	28300	489.3	498.46	498.46	500.23	0.01108	11.2	3048.4	982.8	0.99
10.8902	28300	496.0	504.96	504.96	506.75	0.01415	12.5	3354.3	998.7	1.11
11.0795	28300	507.2	517.86	517.86	520.04	0.01046	12.0	2566.9	704.7	0.98
11.1742	28300	515.4	524.24	524.24	525.91	0.01034	10.5	2972.2	1025.7	0.95
11.2585	28300	521.8	530.08	530.08	531.92	0.00377	10.9	2597.9	698.8	1.00

Exhibit B6

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
11.2586	Bridge									
11.2678	28300	521.8	531.01	530.06	532.18	0.00187	8.7	3266.1	732.9	0.72
11.3636	28300	528.5	539.31	539.31	542.06	0.00913	13.7	2433.9	601.5	0.96
11.4583	28300	538.1	545.87	545.78	547.25	0.01119	9.5	3068.7	1060.1	0.95
11.553	28300	546.0	553.60	553.60	554.95	0.01321	9.4	3143.5	1208.3	1.01
11.6477	27100	553.4	561.22	561.22	562.57	0.01291	9.4	2975.5	1160.4	1.00
11.7424	27100	560.2	568.75	568.75	569.99	0.01502	9.0	3102.0	1403.5	1.05
11.8371	27100	569.1	576.93	576.93	578.28	0.01365	10.2	3400.3	1354.4	1.04
11.9318	27100	574.3	583.40	583.04	584.61	0.01236	9.8	3375.8	1009.5	0.99
12.0265	27100	581.1	590.89	590.89	592.55	0.01430	12.1	3239.4	901.8	1.11
12.1212	27100	588.8	598.51	598.51	600.51	0.01081	11.4	2507.2	677.0	0.98
12.2159	27100	598.1	606.22	606.22	607.89	0.01194	10.6	2813.4	872.9	1.00
12.3106	27100	604.4	612.77	612.54	614.30	0.01431	11.0	2958.2	801.2	1.08
12.4053	27100	614.3	620.69	620.09	622.14	0.01429	11.0	3284.8	970.3	1.08
12.5	27100	620.6	627.85	627.69	629.58	0.01262	11.7	3021.2	802.4	1.05
12.5947	27100	623.7	635.49	635.49	637.86	0.00876	12.9	2654.7	674.8	0.94
12.6894	27100	631.6	643.06	643.06	644.60	0.01176	10.2	2966.7	1039.9	0.99
12.7841	27100	642.6	651.44	651.44	652.85	0.01270	10.1	3327.8	1282.0	1.01
12.8788	27100	646.8	660.48	660.48	662.08	0.01177	10.6	3024.2	988.8	0.99
12.9735	27100	657.4	668.45	668.45	670.09	0.01258	11.4	3320.7	1157.7	1.03
13.0682	27100	665.5	675.87	675.87	677.25	0.01153	10.8	3894.8	1416.4	0.99
13.1629	27100	673.6	682.92	682.92	684.79	0.00990	12.7	3619.0	1230.4	0.97
13.2576	27100	678.5	689.73	689.73	692.65	0.00889	13.8	2112.6	443.7	0.96
13.3523	27100	686.3	701.22	701.22	702.76	0.00907	10.5	3481.1	1355.8	0.90
13.447	27100	696.9	708.18	708.18	710.41	0.00884	12.2	2660.1	843.8	0.93
13.5417	27100	707.2	716.94	716.94	719.47	0.01011	13.2	2497.1	798.2	0.99
13.6364	27100	713.7	725.34	725.34	726.63	0.00614	11.0	5047.5	1674.6	0.78
13.7311	27100	721.3	731.55	731.55	733.20	0.00711	11.7	4177.2	1450.8	0.84
13.8258	27100	725.1	737.56	737.56	739.18	0.00612	11.7	4233.7	1260.9	0.79
13.9205	27100	731.8	744.04	744.04	745.97	0.00776	11.9	3193.7	885.1	0.87
14.0152	27100	739.5	750.64	750.64	753.52	0.01013	14.3	2316.7	443.3	1.01
14.1098	27100	748.6	760.91	760.91	763.31	0.00750	12.7	2649.2	746.6	0.87
14.1335	27100	748.9	764.04	764.04	766.19	0.00867	12.0	2683.6	873.3	0.90
14.1761	27100	758.8	765.95		768.18	0.00865	12.0	2262.7	421.9	0.91
14.2045	27100	760.2	770.93	770.93	773.58	0.01043	13.1	2073.4	393.3	1.00
14.2992	27100	768.4	777.30	777.30	780.73	0.00960	14.9	1825.4	268.2	1.00
14.3939	27100	772.5	783.91	783.91	787.66	0.00799	16.1	2040.3	302.8	0.95
14.4886	27100	778.6	790.79	790.79	794.70	0.00899	16.4	2014.1	332.3	1.00
14.5833	27100	783.9	798.16	798.16	803.33	0.00845	18.3	1491.1	157.5	1.00
14.678	27100	790.6	804.79	804.79	809.46	0.00681	17.7	1869.9	299.1	0.92
14.7727	27100	796.2	812.39	812.39	816.75	0.00665	17.1	1929.7	348.0	0.90
14.8674	27100	809.2	821.62	821.62	824.62	0.00531	15.5	3055.2	590.6	0.81
14.9621	27100	813.7	826.61	826.61	830.57	0.00832	16.5	1924.7	264.0	0.97
15.0568	27100	820.5	833.33	833.33	837.46	0.00910	16.3	1661.2	201.5	1.00
15.1515	27100	831.8	843.80	843.80	846.75	0.01024	13.8	1988.0	349.7	1.00
15.2462	27100	839.7	853.23	853.23	855.83	0.01050	13.0	2112.7	444.4	1.00
15.3409	27100	851.9	864.81	864.81	866.91	0.01113	11.7	2376.7	578.8	1.00
15.4356	27100	865.2	875.73	875.73	879.51	0.00913	15.6	1750.8	239.8	0.99
15.4979	27100	868.1	881.45	881.45	886.10	0.00749	17.9	1872.9	248.5	0.96
15.5036	27100	867.1	886.71	886.71	891.47	0.00246	23.1	3600.6	354.0	0.94
15.5038	Bridge									
15.5066	27100	867.1	891.37	886.75	893.47	0.00091	16.3	5288.1	387.7	0.59
15.5104	27100	868.5	892.94		893.84	0.00073	9.0	5503.3	407.4	0.33
15.5303	27100	869.8	892.88		893.99	0.00086	8.8	3831.3	290.0	0.35
15.625	27100	879.2	892.54		895.36	0.00494	13.5	2029.3	237.1	0.76
15.7197	27100	892.8	903.71	903.71	908.07	0.00862	16.8	1647.4	201.3	0.99
15.8144	27100	900.5	915.64	915.64	919.61	0.00812	16.4	1934.9	278.2	0.95
15.9091	21600	908.9	924.62	924.62	929.70	0.00873	18.1	1201.8	123.0	1.00
16.0038	21600	925.8	938.38	938.38	943.28	0.00928	17.8	1215.8	124.2	1.00
16.0985	21600	936.6	949.93	949.93	954.39	0.00900	17.0	1275.6	146.0	1.00
16.1932	21600	949.6	965.29	965.29	968.85	0.00950	15.1	1439.0	215.7	1.00
16.2879	21600	960.4	974.69	974.69	980.12	0.00910	18.7	1154.6	106.8	1.00
16.3826	21600	973.6	988.49	988.49	993.87	0.00893	18.6	1160.3	108.0	1.00

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Exhibit B7
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Standard Table 1
500-year event

River Sta	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
	(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
-0.2258	105500	-18.0	2.53	-7.41	2.87	0.00033	4.7	22673.4	1479.0	0.21
-0.1784	105500	-14.3	2.34		3.15	0.00125	7.3	14545.0	1342.0	0.39
-0.1311	105500	-10.5	2.07		4.05	0.00470	11.3	9341.7	1204.0	0.71
-0.0837	105500	-4.7	3.75	3.75	7.13	0.00964	14.8	7149.1	1059.0	1.00
-0.0364	105500	-1.8	6.40	6.40	10.09	0.00942	15.4	6838.4	929.0	1.00
0.0477	105500	2.5	12.52	12.52	15.77	0.00684	14.8	8699.4	1745.8	0.89
0.1052	105500	-0.3	15.97	11.82	16.70	0.00134	8.2	22172.3	2490.5	0.41
0.1383	105500	1.0	16.15	12.26	16.93	0.00143	8.7	21604.1	2394.0	0.43
0.1909	105500	-1.3	16.05	12.58	17.55	0.00078	10.8	20714.6	3458.5	0.55
0.191	Mult Open									
0.1945	105500	-1.3	16.70	12.48	17.99	0.00064	10.1	23023.7	3654.4	0.50
0.3579	105500	3.6	17.45	17.45	20.98	0.00872	15.2	7363.5	3462.2	0.97
0.4394	105500	0.0	19.94	19.94	23.68	0.00302	15.5	6797.9	2548.8	1.00
0.4395	Mult Open									
0.464	105500	0.0	23.87	19.90	25.47	0.00075	10.2	10389.8	3043.4	0.53
0.5204	105500	5.6	24.60	22.59	25.89	0.00243	11.2	19482.4	2960.2	0.56
0.5922	105500	6.2	25.50	25.50	28.58	0.00320	14.1	7482.0	2762.1	1.00
0.5923	Mult Open									
0.6028	105500	6.2	28.24	25.49	29.72	0.00095	9.8	10810.3	2853.3	0.58
0.6629	105500	11.8	28.93	27.34	30.22	0.00289	11.9	19286.3	2844.5	0.60
0.7577	105500	15.5	30.44	30.36	32.82	0.00599	16.7	14994.1	2536.7	0.87
0.8523	105500	17.6	34.75	34.75	37.73	0.00428	16.4	13553.4	2260.8	0.76
0.947	105500	19.3	37.84	37.84	41.66	0.00522	18.3	10789.6	1456.8	0.84
1.0417	105500	20.7	39.57	39.57	44.88	0.00516	19.6	7650.9	1031.6	0.85
1.1364	105500	23.0	45.30	45.30	50.55	0.00460	20.7	8903.8	1032.6	0.83
1.2311	105500	25.3	49.45	44.22	52.33	0.00223	14.3	10075.7	1003.7	0.57
1.3258	105500	30.2	50.00	47.38	54.04	0.00340	17.0	8387.5	762.4	0.70
1.4205	105500	30.6	53.63	46.95	55.29	0.00133	10.9	12917.0	1025.4	0.44
1.5152	105500	36.1	54.63	48.59	56.01	0.00145	9.5	11684.1	1024.5	0.44
1.6098	105500	36.9	53.94	53.47	58.16	0.00590	16.9	7397.3	898.6	0.86
1.7045	105500	41.8	57.93	57.93	63.30	0.00763	21.0	7496.9	703.0	1.01
1.7992	105500	43.2	64.69	64.69	70.68	0.00573	21.0	7032.8	707.9	0.90
1.8939	105500	43.8	69.93	64.08	72.68	0.00218	14.1	9729.5	726.9	0.57
1.9886	105500	50.2	71.67	66.49	73.79	0.00208	12.2	10611.8	895.9	0.54
2.0827	105500	53.4	72.77	69.58	75.00	0.00268	13.1	11706.4	1139.1	0.60
2.178	105500	59.1	74.20	72.65	76.96	0.00473	15.1	10554.3	1200.5	0.78
2.2727	105500	62.2	77.31	73.98	78.70	0.00240	9.5	11401.4	1237.6	0.54
2.3674	105500	68.0	78.46	78.14	81.12	0.00838	13.1	8067.8	1348.5	0.93
2.4621	105500	70.6	82.81	82.77	85.69	0.00971	13.6	7763.9	1375.0	0.99
2.5568	105500	72.7	91.75	91.75	97.63	0.00734	20.7	6532.8	576.1	0.98
2.6515	105500	79.3	98.56	98.56	105.27	0.00696	22.2	6163.2	515.0	0.98
2.7462	105500	81.7	102.42	102.42	110.32	0.00645	23.0	5183.7	362.7	0.96
2.8409	105500	82.5	108.80	104.33	112.91	0.00323	16.3	6516.3	426.9	0.67
2.9356	105500	84.4	109.72		115.25	0.00458	19.1	6032.8	410.2	0.80
3.0303	105500	91.7	114.63	114.63	122.75	0.00736	22.9	4614.4	284.5	1.00
3.125	105500	95.5	120.47	120.47	129.60	0.00727	24.3	4351.0	239.7	1.00
3.1546	105500	99.3	121.81	121.81	131.27	0.00242	24.7	4272.8	225.3	1.00
3.1547	Bridge									
3.1591	105500	99.3	139.41	121.78	141.21	0.00025	11.4	15945.7	1401.1	0.35
3.178	105500	100.2	140.15		141.49	0.00054	9.9	16007.0	969.2	0.30
3.2197	105500	100.6	140.49		141.63	0.00054	9.5	18187.9	1100.4	0.30
3.4091	105500	109.0	140.90		142.40	0.00084	10.4	13797.5	811.4	0.36
3.5038	105500	114.2	140.60		143.40	0.00188	13.5	8486.8	662.1	0.53
3.5985	105500	118.6	139.91	139.91	147.51	0.00761	22.1	4770.3	314.5	1.00
3.6932	105500	121.5	144.19	144.19	151.34	0.00683	21.5	5084.0	483.4	0.96
3.7879	105500	125.2	147.50	147.00	154.74	0.00667	21.6	4896.3	319.3	0.96
3.8826	105500	125.4	152.26	150.06	157.53	0.00416	18.8	6485.1	496.1	0.77
3.9773	105500	136.0	155.77	155.77	162.60	0.00696	21.1	5333.5	466.5	0.97
4.072	105500	138.1	161.89	161.89	169.60	0.00745	22.3	4769.3	374.2	0.99
4.1667	105500	142.6	168.11		172.20	0.00301	16.5	7588.8	719.8	0.66
4.2614	105500	146.4	171.06		173.70	0.00208	13.7	10419.1	958.6	0.55
4.3561	105500	151.1	172.64		175.43	0.00295	13.9	9838.2	1170.7	0.63
4.4508	105500	153.7	174.84	174.84	179.31	0.00561	17.6	7953.8	1082.8	0.86
4.5455	105500	160.3	180.03	180.03	184.55	0.00559	18.4	8545.6	1006.2	0.86
4.6402	93100	161.6	184.14	184.14	188.64	0.00487	18.8	8322.2	1065.4	0.82
4.7348	93100	166.4	187.28	187.28	192.16	0.00536	19.5	7980.4	1059.2	0.86
4.8295	93100	169.0	191.87	191.87	197.20	0.00530	19.5	6715.9	928.2	0.85
4.9242	93100	171.0	197.72	197.72	202.69	0.00374	18.6	7774.8	1357.5	0.74
5.0189	93100	173.4	199.62	199.62	208.73	0.00728	24.2	3842.2	211.1	1.00

Exhibit B7

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
5.1136	93100	174.4	208.99		210.37	0.00084	10.6	14295.7	1060.3	0.37
5.303	93100	183.2	209.89		211.57	0.00142	10.8	10941.5	975.2	0.45
5.3977	93100	188.7	209.90		213.32	0.00517	15.2	7252.5	917.1	0.80
5.4924	93100	196.5	215.29	215.29	221.04	0.00656	19.6	5511.9	604.0	0.93
5.5871	93100	200.8	218.10	218.10	224.77	0.00671	20.9	4858.7	421.3	0.95
5.6818	93100	203.2	224.10	224.10	231.27	0.00604	22.0	5005.3	404.5	0.93
5.7765	93100	204.5	228.24	228.24	235.97	0.00589	23.0	4906.1	366.7	0.92
5.8712	93100	206.4	232.08	231.70	238.76	0.00505	21.4	5515.9	471.8	0.85
5.9301	93100	211.2	239.25		240.80	0.00087	10.7	12211.1	747.5	0.38
5.9475	93100	212.6	239.17	229.92	240.97	0.00039	11.3	13357.1	1055.3	0.42
5.9476	Bridge									
5.9536	93100	212.6	240.08	229.94	241.69	0.00035	11.1	13313.9	1136.9	0.40
5.9742	93100	212.8	240.28		241.92	0.00104	10.8	10818.5	679.4	0.40
6.0606	93100	215.0	241.09		242.43	0.00102	9.6	11489.7	731.4	0.39
6.1553	93100	219.9	242.18		242.91	0.00072	7.0	15172.5	1169.3	0.31
6.3447	93100	230.8	245.16	245.16	248.48	0.00764	15.6	8201.0	1404.5	0.93
6.4394	89000	238.5	249.83	249.83	253.48	0.00689	15.8	7034.2	1266.6	0.90
6.5341	89000	239.0	253.58	253.58	257.15	0.00687	16.1	7879.3	1339.7	0.90
6.6288	89000	244.1	257.45	257.45	260.96	0.00610	15.9	8002.1	1337.3	0.86
6.7235	89000	245.8	260.74	259.51	263.98	0.00564	14.5	6339.0	1085.0	0.82
6.8182	89000	251.6	263.93	263.89	267.49	0.00846	15.4	6385.6	1013.9	0.97
6.8389	89000	252.0	265.41	264.55	268.34	0.00608	13.8	6897.6	984.6	0.83
6.9129	89000	252.4	267.82	267.67	271.35	0.00876	15.1	5897.4	790.7	0.97
7.0076	89000	255.4	272.06	272.06	276.01	0.00911	15.9	5583.7	704.6	1.00
7.1023	89000	261.9	277.27	276.27	279.92	0.00639	13.1	6824.7	900.5	0.83
7.197	89000	268.7	281.09	281.09	284.20	0.00980	14.2	6290.2	1010.9	1.00
7.2917	89000	275.6	286.79	286.35	288.53	0.00683	11.3	10342.5	2290.6	0.83
7.3864	89000	278.7	291.31	291.31	293.77	0.00644	14.8	11052.2	2225.7	0.86
7.4811	89000	282.3	297.18	297.18	300.24	0.00562	15.1	9124.8	1759.0	0.83
7.5758	89000	285.8	300.56	300.56	303.66	0.00642	15.7	9268.4	1690.2	0.88
7.6705	89000	287.1	304.81	304.81	308.95	0.00598	17.2	7158.0	1117.5	0.87
7.7652	89000	290.7	309.39	309.39	313.28	0.00616	16.6	7374.6	1196.0	0.87
7.8598	89000	296.4	311.89	311.77	317.19	0.00785	18.5	4858.5	478.0	0.98
7.9545	89000	302.3	318.15		319.23	0.00174	8.3	10725.7	1056.8	0.46
8.0492	36700	307.6	318.46		320.77	0.00612	12.6	3518.3	597.7	0.81
8.1439	36700	313.7	323.38	323.38	325.67	0.00952	12.4	3350.9	796.2	0.96
8.2386	36700	319.4	329.29	329.29	331.42	0.00916	12.5	3934.7	1061.5	0.94
8.3333	36700	328.1	335.32	335.32	337.18	0.01071	11.6	3980.0	1134.5	0.98
8.428	36700	331.9	341.54	341.23	343.28	0.01459	12.0	3888.3	1009.0	1.11
8.5227	36700	338.9	347.26	347.10	348.90	0.00882	10.6	4116.8	1361.2	0.89
8.6174	36700	343.6	353.43	353.43	355.09	0.01102	10.5	3855.2	1270.3	0.97
8.7121	36700	349.7	359.54	359.54	361.08	0.01246	10.0	3750.0	1237.3	1.00
8.8068	36700	359.0	366.42	365.97	367.38	0.01216	8.9	5334.4	1890.3	0.97
8.9015	36700	363.3	371.99		372.71	0.00900	7.9	6096.1	2064.0	0.84
8.9962	36700	371.3	376.77	375.38	377.48	0.01258	9.0	6083.1	1786.8	0.98
9.0909	36700	378.0	384.66	384.04	385.42	0.00846	9.0	7104.2	2322.8	0.85
9.1857	36700	383.9	390.45	389.69	391.07	0.01443	9.0	6869.7	2574.4	1.03
9.2804	36700	389.8	397.59	397.21	399.25	0.01637	14.0	6697.2	2234.6	1.21
9.3264	36700	393.7	401.05	401.05	402.79	0.00857	13.5	7396.7	2139.6	0.94
9.3786	36700	393.7	407.03	407.03	411.91	0.00280	17.8	2446.1	657.5	0.93
9.3787	Bridge									
9.3864	36700	393.7	414.98	406.56	415.95	0.00051	9.3	7963.4	836.5	0.37
9.4009	36700	393.7	416.47	405.80	416.55	0.00018	3.4	30491.4	2359.8	0.14
9.4697	36700	398.1	416.45	410.18	416.72	0.00051	5.5	19430.4	2247.6	0.26
9.5644	36700	401.2	415.47	415.47	419.49	0.00911	16.1	2284.2	285.6	1.00
9.6591	36700	409.8	421.59	421.59	425.55	0.00906	16.0	2302.3	291.5	1.00
9.7538	36700	415.7	427.67	427.67	430.82	0.00983	14.3	2573.0	406.8	1.00
9.8485	36700	422.1	433.31	432.85	435.35	0.00796	11.5	3203.5	605.0	0.88
9.9432	36700	428.3	439.33	439.33	441.77	0.01041	12.6	2960.3	630.3	0.99
10.0379	36700	434.6	445.55	445.55	447.92	0.01065	12.4	2969.7	622.6	1.00
10.1326	36700	442.9	451.66	451.66	453.64	0.01139	11.3	3247.6	821.2	1.00
10.2273	36700	448.2	457.80	457.80	460.35	0.00989	12.9	2934.1	607.9	0.98
10.322	36700	454.7	464.65	464.65	466.42	0.01578	11.3	3631.2	1053.7	1.13
10.4167	36700	465.5	473.09	473.09	474.89	0.01034	11.1	3976.4	1285.1	0.96
10.5114	36700	470.8	479.34	479.34	481.30	0.01058	12.3	4279.1	1256.1	0.99
10.6061	36700	472.9	486.07	486.07	488.12	0.00839	12.0	4025.8	1196.9	0.90
10.7008	36700	480.7	492.71	492.71	494.89	0.00870	12.4	3835.4	1052.8	0.93
10.7955	36700	489.3	499.16	499.16	501.21	0.01053	12.1	3742.7	1002.5	0.99
10.8902	36700	496.0	505.76	505.76	507.79	0.01278	13.4	4183.4	1069.8	1.09
11.0795	36700	507.2	518.85	518.85	521.20	0.00932	12.5	3343.4	866.8	0.95
11.1742	36700	515.4	524.95	524.95	526.83	0.01003	11.2	3721.4	1094.3	0.95
11.2585	36700	521.8	530.82	530.82	532.96	0.00359	11.7	3125.4	724.6	1.00

Exhibit B7

River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
11.2586	Bridge									
11.2678	36700	521.8	531.86	530.79	533.23	0.00185	9.4	3905.3	767.5	0.73
11.3636	36700	528.5	540.72	540.72	543.46	0.00737	13.9	3528.9	965.7	0.89
11.4583	36700	538.1	546.41	546.32	548.07	0.01113	10.4	3657.5	1113.1	0.97
11.553	36700	546.0	554.18	554.18	555.68	0.01274	10.0	3876.1	1333.0	1.01
11.6477	35200	553.4	561.79	561.79	563.31	0.01262	10.0	3666.2	1289.7	1.01
11.7424	35200	560.2	569.34	569.34	570.64	0.01247	9.2	4012.2	1654.4	0.98
11.8371	35200	569.1	577.50	577.50	579.02	0.01313	10.8	4220.5	1503.6	1.04
11.9318	35200	574.3	583.95	583.70	585.43	0.01314	10.7	3957.0	1094.1	1.04
12.0265	35200	581.1	591.60	591.60	593.38	0.01630	12.5	3927.3	1072.2	1.17
12.1212	35200	588.8	599.37	599.37	601.61	0.01028	12.2	3126.8	781.7	0.98
12.2159	35200	598.1	606.91	606.91	608.83	0.01146	11.4	3428.3	921.3	1.00
12.3106	35200	604.4	613.42	613.24	615.23	0.01501	11.9	3509.3	876.7	1.12
12.4053	35200	614.3	621.37	620.52	623.03	0.01351	11.8	3954.7	1002.1	1.08
12.5	35200	620.6	628.42	628.42	630.63	0.01338	13.2	3483.3	808.7	1.11
12.5947	35200	623.7	636.48	636.48	639.18	0.00832	13.9	3334.0	696.8	0.94
12.6894	35200	631.6	643.67	643.67	645.46	0.01141	11.0	3635.1	1132.7	0.99
12.7841	35200	642.6	652.04	652.04	653.61	0.01204	10.7	4136.5	1382.0	1.01
12.8788	35200	646.8	661.18	661.18	662.97	0.01151	11.2	3782.1	1173.9	1.00
12.9735	35200	657.4	669.14	669.14	670.98	0.01218	12.3	4166.2	1342.1	1.04
13.0682	35200	665.5	676.43	676.43	678.02	0.01182	11.8	4727.2	1543.2	1.02
13.1629	35200	673.6	683.98	683.98	685.73	0.00814	12.7	5055.4	1453.3	0.90
13.2576	35200	678.5	691.07	691.07	694.24	0.00823	14.5	2793.1	624.1	0.94
13.3523	35200	686.3	701.92	701.92	703.59	0.00922	11.2	4524.8	1606.0	0.92
13.447	35200	696.9	709.57	709.57	711.52	0.00661	11.8	4327.3	1492.8	0.82
13.5417	35200	707.2	718.68	718.68	720.61	0.00584	12.0	4877.3	1785.3	0.79
13.6364	35200	713.7	725.87	725.87	727.30	0.00771	12.0	5982.2	1832.6	0.87
13.7311	35200	721.3	732.35	732.35	734.10	0.00693	12.4	5405.9	1608.7	0.85
13.8258	35200	725.1	738.30	738.30	740.07	0.00694	12.7	5181.8	1303.5	0.85
13.9205	35200	731.8	744.79	744.79	747.07	0.00826	13.1	3883.0	928.8	0.92
14.0152	35200	739.5	751.98	751.98	755.18	0.00918	15.1	2993.6	571.2	0.98
14.1098	35200	748.6	762.23	762.23	764.63	0.00663	13.0	3765.5	936.5	0.83
14.1335	35200	748.9	765.03	765.03	767.34	0.00799	12.6	3583.1	950.9	0.88
14.1761	35200	758.8	766.73	766.72	769.57	0.01035	13.5	2602.3	461.8	1.00
14.2045	35200	760.2	772.07	772.07	775.07	0.00928	13.9	2591.5	568.3	0.97
14.2992	35200	768.4	778.65	778.65	782.66	0.00911	16.1	2189.8	273.9	1.00
14.3939	35200	772.5	785.71	785.71	789.80	0.00725	16.9	2639.8	355.0	0.92
14.4886	35200	778.6	792.76	792.76	796.62	0.00716	16.6	2766.7	424.9	0.92
14.5833	35200	783.9	800.45	800.45	806.18	0.00761	19.3	1903.2	195.0	0.97
14.678	35200	790.6	807.42	807.42	811.96	0.00529	17.9	2808.9	414.2	0.84
14.7727	35200	796.2	814.87	814.87	818.97	0.00509	17.1	3026.1	487.5	0.81
14.8674	35200	809.2	822.94	822.94	826.25	0.00538	16.7	3846.3	601.2	0.83
14.9621	35200	813.7	828.50	828.50	832.77	0.00763	17.2	2491.7	359.2	0.95
15.0568	35200	820.5	835.11	835.11	839.76	0.00841	17.3	2065.1	255.5	0.99
15.1515	35200	831.8	845.09	845.09	848.38	0.00952	14.6	2472.7	428.0	0.99
15.2462	35200	839.7	854.58	854.58	857.21	0.00991	13.1	2823.5	629.3	0.98
15.3409	35200	851.9	865.63	865.63	868.09	0.01074	12.6	2861.7	600.5	1.00
15.4356	35200	865.2	877.34	877.34	881.62	0.00832	16.6	2183.8	288.5	0.98
15.4979	35200	868.1	883.85	883.85	888.52	0.00600	18.3	2641.2	347.0	0.89
15.5036	35200	867.1	888.65	888.65	894.15	0.00265	25.6	4295.4	362.7	0.99
15.5038	Bridge									
15.5066	35200	867.1	894.03	888.72	896.52	0.00098	18.2	6349.4	408.1	0.62
15.5104	35200	868.5	895.98		896.98	0.00072	9.7	6751.6	415.7	0.34
15.5303	35200	869.8	895.84		897.16	0.00086	9.7	4809.8	346.4	0.36
15.625	35200	879.2	895.64		898.31	0.00328	13.2	2791.8	256.1	0.65
15.7197	35200	892.8	905.49	905.49	910.52	0.00807	18.1	2016.7	214.9	0.98
15.8144	35200	900.5	917.32	917.32	921.79	0.00761	17.6	2412.7	289.2	0.95
15.9091	27900	908.9	926.61	926.61	932.49	0.00825	19.5	1452.9	129.3	0.99
16.0038	27900	925.8	940.28	940.28	945.99	0.00894	19.2	1455.1	127.6	1.00
16.0985	27900	936.6	951.62	951.62	956.87	0.00858	18.4	1527.5	152.2	1.00
16.1932	27900	949.6	966.67	966.67	970.81	0.00891	16.4	1745.6	231.3	0.99
16.2879	27900	960.4	976.93	976.93	983.07	0.00879	19.9	1402.3	114.5	1.00
16.3826	27900	973.6	990.69	990.69	996.81	0.00865	19.8	1406.0	115.3	1.00

Exhibit B10
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Distributed values
10-year event

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
-0.2258		41300			1.82	
-0.1784		41300			2.8	
-0.1311		41300			4.2	
-0.0837		41300			7.23	
-0.0364		41300			11.29	
0.0477	0.27	41296.95	2.78	0.67	12.03	0.62
0.1052	171.82	32913.11	8215.07	1.32	6.49	1.69
0.1383	616.48	32837.98	7845.54	1.64	6.83	1.6
0.1909		37300.37	3999.63		7.38	1.09
0.191	Mult Open					
0.1945		37163.02	4136.98		7.15	1.05
0.3579		41296.73	3.27		11.44	0.6
0.4394		41300			11.76	
0.4395	Mult Open					
0.464		41300			7.64	
0.5204	324.6	39756.44	1218.97	2.19	12.8	1.6
0.5922		41300			12.04	
0.5923	Mult Open					
0.6028		41300			7.83	
0.6629	2648.8	35684.15	2967.05	3.31	12.18	1.57
0.7577	4728.05	29894.59	6677.35	2.91	12.32	2.03
0.8523	1285.98	39991.63	22.4	2.01	14.26	0.59
0.947	4950.39	36344.98	4.63	3.69	13.83	0.33
1.0417	2486.63	38813.37		2.59	11.06	
1.1364	3089.96	38106.08	103.96	3.75	16.87	1.54
1.2311	1227.83	40072.17		2.81	10.15	
1.3258	485.23	40613.85	200.92	1.59	12.57	1.92
1.4205	285.14	40265.43	749.43	1.89	8.46	1.95
1.5152	0.01	41299.99		0.22	8.84	
1.6098	11.15	41094.76	194.09	0.86	13.63	2.3
1.7045	228.45	34889.9	6181.65	2.14	15.2	4.39
1.7992		39549.07	1750.93		16.44	3.45
1.8939		38257.64	3042.36		10.07	3.12
1.9886		39505.55	1794.45		10.87	2.89
2.0827	862.5	39365.31	1072.19	2.57	12.28	2.59
2.178	103.96	35502.09	5693.96	1.08	12.48	3.76
2.2727		41300			7.28	
2.3674		41300			10.3	
2.4621		41300			8.71	
2.5568	2234.61	39065.39		3.51	15.37	
2.6515	2319.61	38980.38		3.63	16.34	
2.7462	152.1	41141.29	6.61	1.76	15.99	1.42
2.8409		41300			12.88	
2.9356		41300			14.49	
3.0303		41300			17.76	
3.125		41300			18.82	
3.1546		41300			18.75	
3.1547	Bridge					
3.1591		41300			15.86	
3.178		41300			17.23	
3.2197		41300			18.18	
3.4091	0.38	41299.63		0.29	12.2	
3.5038		41300			16.08	
3.5985		41300			17.22	
3.6932		41300			15.71	
3.7879		41300			15.76	
3.8826		41275.73	24.27		14.8	0.92
3.9773		41300			16.93	
4.072		41300			17.82	
4.1667	49.29	41250.71		1.02	12.79	
4.2614	71.81	41024.45	203.74	1.72	13.3	2.09
4.3561		41300			13.66	
4.4508	53.12	41246.88		2.6	15.2	
4.5455		41300			16.45	
4.6402	81.11	36314.15	4.74	2.33	15.95	0.47
4.7348		36384.9	15.1		16.19	0.72
4.8295		36400			15.81	
4.9242		36400			17.67	
5.0189		36400			18.18	

Exhibit B10

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
5.1136	2945.05	33454.95		2.3	9.64	
5.303		36400			14.45	
5.3977		36400			13.36	
5.4924		36381.9	18.1		15.67	1
5.5871		36398.86	1.14		13.08	0.43
5.6818	315.78	36084.22		2.89	17.11	
5.7765	380.21	36019.79		3.07	17.03	
5.8712		36400			17.03	
5.9301	484.04	35868.44	47.52	1.22	8.85	1.27
5.9475		36400			9.92	
5.9476	Bridge					
5.9536		36400			9.81	
5.9742	8.41	36391.59		0.41	10.59	
6.0606		36388.6	11.4		10.48	1.5
6.1553		36395.85	4.15		8.26	0.96
6.3447		32882.51	3517.49		13.22	5.62
6.4394		35105.4	94.6		11.19	0.97
6.5341	355.78	34177.82	666.41	2.08	12.42	2.05
6.6288	395.03	34027.51	777.46	2.19	11.26	1.88
6.7235		35200			9.97	
6.8182	26.74	35149.4	23.86	1.03	11.64	1.25
6.8389		35112.14	87.86		9.36	1.33
6.9129		35200			10.98	
7.0076		35200			10.6	
7.1023		35198.13	1.87		11.02	1.33
7.197		35200			9.4	
7.2917		35200			10.14	
7.3864		28488.2	6711.8		10.37	3.22
7.4811		35071.57	128.43		13.01	1.12
7.5758		33785.51	1414.49		12.05	2.17
7.6705		35200			12.78	
7.7652		35200			14.24	
7.8598	0.18	35199.82		0.32	10.67	
7.9545		35200			8.53	
8.0492	873.48	15126.52		4.13	11.37	
8.1439	264.97	15735.03		2.82	10.21	
8.2386	79.27	14987.39	933.34	1.56	10.01	3.01
8.3333	1970.35	14029.65		4.22	9.13	
8.428	5851.41	10148.59		4.94	8.76	
8.5227	313.47	15686.53		2.17	8.08	
8.6174	23.58	15960.7	15.72	1.52	8.59	0.95
8.7121		15959.8	40.2		8.47	1.43
8.8068	226.08	10578.7	5195.22	2.51	8.61	5.24
8.9015	1988.26	9755.29	4256.45	2.99	6.17	3.24
8.9962	1169.04	2950.37	11880.59	3.66	5.78	5.07
9.0909	2279.38	9164.7	4555.92	2.69	6.96	2.5
9.1857	4847.12	5070.34	6082.54	4.21	8.35	3.63
9.2804	1677.72	8318.05	6004.22	2.94	9.49	2.58
9.3264		11715.23	4284.77		11.29	1.92
9.3786		16000			14.69	
9.3787	Bridge					
9.3864		16000			11.59	
9.4009		16000			8.97	
9.4697		16000			13.12	
9.5644		16000			12.34	
9.6591		16000			12.35	
9.7538		16000			10.95	
9.8485		16000			9.25	
9.9432		16000			9.93	
10.0379		16000			9.83	
10.1326		16000			8.58	
10.2273		16000			10.38	
10.322		12393.83	3606.17		9.98	6.12
10.4167		15631.31	368.69		9.21	2.01
10.5114		14960.51	1039.49		10.78	2.3
10.6061	108.23	15508.18	383.58	1.63	10.2	3.02
10.7008	310.76	15575.75	113.49	2.2	9.93	1.38
10.7955	35.46	14840.82	1123.72	1.15	9.75	4.16
10.8902	1060.26	11476.09	3463.65	3.28	10.44	4.11
11.0795		15956.18	43.82		10.44	1.77
11.1742	0.15	15999.85		0.28	9.72	
11.2585		16000			9.88	

Exhibit B10

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
11.2586	Bridge					
11.2678		16000			6.93	
11.3636	53.35	15502.56	444.09	2.39	11.84	3.33
11.4583	0.04	15926.35	73.61	0.28	8.38	2.12
11.553	3.04	15603.74	393.23	0.64	8.45	3.81
11.6477	12.44	14987.56		0.94	8.48	
11.7424		14902.35	97.65		8.93	2.17
11.8371	183.22	12467.27	2349.52	2.9	9.34	4
11.9318		11445.86	3554.14		8.92	3.96
12.0265	35.94	10654.3	4309.76	2.15	11.35	4.44
12.1212	14.76	14985.24		1.31	10.78	
12.2159	453.54	14544.61	1.86	2.93	9.58	0.65
12.3106	0.72	10563.23	4436.05	0.79	9.35	5.18
12.4053		8831.69	6168.32		8.82	6.08
12.5	0.85	11682.04	3317.11	1.15	9.38	4.04
12.5947	58.76	14320.41	620.83	2.32	11.47	3.81
12.6894	294.96	14645.96	59.09	2.19	8.81	2.27
12.7841	852.56	13931.58	215.86	2.57	8.77	2.1
12.8788	714.37	14285.63		2.43	9.63	
12.9735	2697.2	12302.8		3.43	9.9	
13.0682	2826.43	12173.57		2.84	9.83	
13.1629	2852.96	12147.04		3.44	10.99	
13.2576	0.01	15000		0.15	12.4	
13.3523	31.78	14968.22		1.04	12.64	
13.447	39.27	14960.73		1.38	11.39	
13.5417	632.11	14367.89		4.08	11.46	
13.6364	1600.76	13399.24		2.01	11.6	
13.7311	1254.69	13745.31		2.45	11.42	
13.8258	1398.17	13601.83		2.42	11.81	
13.9205	777.11	14222.89		2.75	12.3	
14.0152	942.04	14057.96		3.14	11.79	
14.1098		15000			12.31	
14.1335		15000			13.61	
14.1761		15000			10.5	
14.2045		15000			10.84	
14.2992		15000			12.33	
14.3939	453.57	14546.43		2.79	13.72	
14.4886	525.5	14474.5		3.38	13.92	
14.5833		15000			15.79	
14.678	3.44	14903.51	93.05	1	15.34	2.02
14.7727		14990.28	9.72		15.64	1.25
14.8674	763.67	13700.03	536.3	3.81	14.13	2.35
14.9621	437.13	14546.97	15.9	3.37	14.25	1.55
15.0568		15000			13.82	
15.1515		14999.31	0.69		12.96	0.79
15.2462		15000			11.56	
15.3409		14981.92	18.08		10.44	1.27
15.4356		15000			13.52	
15.4979	66.54	14567.37	366.09	2.28	14.72	2.77
15.5036	2298.66	9829.47	2871.87	2.75	19.23	3.34
15.5038	Bridge					
15.5066	3170.56	8515.66	3313.78	2.1	13.26	2.52
15.5104	2151.04	11253.64	1595.32	1.74	7.06	2.16
15.5303	506.86	14236.51	256.63	1.52	6.95	1.71
15.625		15000			13.62	
15.7197		14998.7	1.3		14.07	0.94
15.8144		14621.81	378.19		13.96	3.1
15.9091		12500			15.4	
16.0038		12500			15.11	
16.0985		12500			14.92	
16.1932		12500			13.98	
16.2879		12500			16.27	
16.3826		12500			16.17	

Exhibit B11
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Distributed values
20-year event

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
-0.2258		52700			2.32	
-0.1784		52700			3.57	
-0.1311		52700			5.38	
-0.0837		52700			9.42	
-0.0364		52700			12.22	
0.0477	5.08	52631.55	63.37	1.4	12.81	1.2
0.1052	301.41	39780.72	12617.88	1.57	6.84	1.89
0.1383	950.02	39262.23	12487.75	1.88	7.18	1.88
0.1909		46804.62	5895.38		8.2	1.19
0.191	Mult Open					
0.1945		46556.55	6143.45		7.89	1.14
0.3579		52631.45	68.55		12.32	1.27
0.4394		52700			12.52	
0.4395	Mult Open					
0.464		52700			8.42	
0.5204	743.38	46854.44	5102.18	1.86	12.52	2.08
0.5922		52700			11.83	
0.5923	Mult Open					
0.6028		52700			8.19	
0.6629	3314.1	40673.6	8712.31	2.72	11.9	2.09
0.7577	6531.35	34411.17	11757.48	3.19	12.94	2.52
0.8523	3739.46	47979.07	981.47	2.58	14.42	1.52
0.947	6208	46206.21	285.8	3.76	16.19	1.19
1.0417	4055.76	48639.41	4.84	2.91	12.17	0.48
1.1364	5273.45	46961.93	464.62	4.56	18.16	2.34
1.2311	1867.03	50803.48	29.49	3.23	11.1	0.71
1.3258	1564.78	50660.39	474.82	2.46	13.45	2.31
1.4205	305.29	51366.19	1028.52	1.14	9.2	1.73
1.5152	5.82	52694.18		0.93	8.78	
1.6098	147.24	52091.28	461.48	1.78	14.62	3.02
1.7045	578.01	43119.78	9002.21	3.03	16.46	5.05
1.7992		49433.41	3266.59		17.78	4.35
1.8939		48298.14	4401.86		10.94	3.56
1.9886		49724.79	2975.21		11.07	3.1
2.0827	1606.01	49102.55	1991.45	2.62	12.89	2.73
2.178	524.87	44253.04	7922.09	2.01	13.61	4.08
2.2727	19.05	52680.95		0.63	7.74	
2.3674		52700			11.01	
2.4621		52700			9.66	
2.5568	3927.9	48772.09		4.09	16.38	
2.6515	4112.64	48587.37		4.49	17.64	
2.7462	484.54	52190.81	24.66	2.8	18.09	2.05
2.8409		52700			13.55	
2.9356		52654.62	45.38		15.77	1.06
3.0303		52700			19	
3.125		52700			20.11	
3.1546		52700			20.13	
3.1547	Bridge					
3.1591		52700			17.09	
3.178		52700			18.06	
3.2197		52700			19.41	
3.4091	119.32	52580.68		1.21	12.95	
3.5038		52700			17.31	
3.5985		52700			18.42	
3.6932		52700			17.22	
3.7879		52700			16.97	
3.8826		52415.51	284.49		15.96	2.07
3.9773		52700			17.94	
4.072		52700			18.88	
4.1667	403.04	52296.96		2.05	13.63	
4.2614	284.74	51814.55	600.72	2.53	13.5	2.68
4.3561	28.66	52671.34		1.21	14.58	
4.4508	115.4	52584.59		3.01	16.37	
4.5455		51473.25	1226.75		15.55	1.66
4.6402	226.65	45510.37	662.98	3.03	16.74	1.52
4.7348		45298.72	1101.28		16.34	1.64
4.8295		46399.13	0.87		17.67	0.3
4.9242		46400			19.17	
5.0189		46400			20.03	

Exhibit B11

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
5.1136	5276.25	41123.75		2.56	9.83	
5.303		46400			12.94	
5.3977		46400			14.3	
5.4924		46153.29	246.71		16.64	2.12
5.5871	0	46333.52	66.48	0.08	14.68	1.32
5.6818	729.72	45670.28		3.57	18.31	
5.7765	635.35	45764.65		3.36	19.32	
5.8712	17.19	46382.81		1	18	
5.9301	1639.58	44631.86	128.56	1.41	9.17	1.5
5.9475		46400			10.22	
5.9476	Bridge					
5.9536		46400			10.13	
5.9742	822.17	45573.94	3.89	1.57	10.32	0.87
6.0606	150.6	46213.76	35.64	0.92	10.19	1.71
6.1553	181.1	46196.84	22.06	1.01	8.19	1.28
6.3447		42601.56	3798.44		14.02	4.7
6.4394		43974.84	425.15		12.4	1.65
6.5341	764.31	42152.4	1483.28	2.67	13.29	2.71
6.6288	672.3	42160.51	1567.2	2.6	12.48	2.5
6.7235		44400			10.79	
6.8182	147.29	44157.28	95.43	1.97	12.5	1.91
6.8389		44186.26	213.74		10.29	1.69
6.9129		44400			11.88	
7.0076		44400			11.79	
7.1023		44393.38	6.62		11.33	1.75
7.197		44400			10.51	
7.2917	2643.64	41756.36		2.47	9.69	
7.3864		35633.13	8766.87		12.44	3.92
7.4811		43600.06	799.94		13.65	1.92
7.5758		42176.52	2223.48		13.54	2.17
7.6705		44352.41	47.59		14.24	1.06
7.7652		44400			15.12	
7.8598	8.72	44391.28		1.13	11.69	
7.9545		44400			8.39	
8.0492	1106.72	18693.28		4.32	12.17	
8.1439	395.4	19404.6		3.1	10.77	
8.2386	175.75	18379.78	1244.47	1.76	10.71	3.15
8.3333	2387.17	17412.83		4.24	9.73	
8.428	7047.23	12752.77		5.29	9.24	
8.5227	456.89	19343.11		2.07	8.74	
8.6174	48.13	19689.44	62.43	1.77	9.08	1.19
8.7121		19706.87	93.13		8.95	1.96
8.8068	412.19	13924.56	5463.25	2.86	8.49	4.38
8.9015	2452.18	12222.97	5124.85	3.26	6.67	3.49
8.9962	1268.36	4521	14010.64	3.09	6.57	5.36
9.0909	2930.77	11101.92	5767.31	2.95	7.39	2.6
9.1857	5780.17	6329.05	7690.78	4.35	8.82	3.82
9.2804	2173.69	10306.18	7320.13	3.09	10.45	2.73
9.3264	0.07	13797.38	6002.55	0.23	11.84	2.1
9.3786		19800			15.65	
9.3787	Bridge					
9.3864		19800			12.38	
9.4009		19800			9.66	
9.4697		19800			14.02	
9.5644	0	19800		0.28	13.17	
9.6591	0.01	19799.99		0.37	13.21	
9.7538		19800			11.75	
9.8485		19800			9.86	
9.9432	0.54	19799.46		0.56	10.41	
10.0379		19800			10.49	
10.1326		19800			9.13	
10.2273		19800			11.5	
10.322		15732.48	4067.52		10.24	6.04
10.4167		19151.96	648.04		9.64	2.22
10.5114		17429.38	2370.62		9.93	2.65
10.6061	264.59	18936.36	599.05	1.98	10.47	3.08
10.7008	510.59	18984.75	304.65	2.45	10.5	1.89
10.7955	122.73	18249.54	1427.73	1.61	10.25	4.29
10.8902	1497.19	13859.76	4443.06	3.63	11.02	4.51
11.0795		19675.83	124.17		10.9	1.72
11.1742	50.7	19749.3		0.9	9.89	
11.2585		19800			10.31	
11.2586	Bridge					
11.2678		19800			7.09	
11.3636	91.78	19066.15	642.07	2.66	12.54	3.52

Exhibit B11

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
11.4583	2.66	19678.2	119.14	0.81	8.83	2.46
11.553	25.13	19313.24	461.63	1.4	8.77	3.74
11.6477	59.17	18738.49	2.35	1.66	8.7	0.74
11.7424		18581.97	218.03		8.28	2.74
11.8371	261.92	15538.08	3000	3	10.13	4.36
11.9318		13856.24	4943.76		8.43	4.46
12.0265	70.85	13174.1	5555.05	2.28	11.8	4.66
12.1212	71.58	18728.42		1.62	11.25	
12.2159	684.7	18083.44	31.86	3.29	9.96	1.17
12.3106	4.22	13432.04	5363.74	1.23	10.21	5.63
12.4053		11633.31	7166.69		9.43	6.16
12.5	2.22	14641.25	4156.54	1.47	10.27	4.41
12.5947	114.57	17756.8	928.63	2.7	11.9	3.35
12.6894	488.5	18216.22	95.27	2.37	9.23	2.58
12.7841	1272.85	17171.45	355.7	2.9	9.33	2.45
12.8788	1268.2	17531.8		2.99	9.88	
12.9735	3772.47	15027.53		3.85	10.53	
13.0682	4026.4	14773.6		3.06	10.38	
13.1629	4183.84	14616.16		3.22	11.22	
13.2576	16.37	18783.63		1.15	13.12	
13.3523	971.71	17828.29		2.05	10.45	
13.447	202.49	18597.51	0	1.62	11.79	0.12
13.5417	878.07	17921.66	0.26	3.81	11.81	0.54
13.6364	4296.58	14503.42		1.91	10.11	
13.7311	2901.53	15898.47		2.01	10.43	
13.8258	2352.16	16434.08	13.76	2.51	12.58	0.76
13.9205	1340.83	17459.17		3.26	13.01	
14.0152	1445.92	17354.08		3.6	12.6	
14.1098	2.3	18797.7		0.74	13.05	
14.1335	0.16	18799.85		0.41	13.18	
14.1761		18800			10.29	
14.2045		18800			11.65	
14.2992		18800			13.24	
14.3939	827.81	17972.19		3.3	14.59	
14.4886	751.05	18048.95		3.43	15.14	
14.5833		18800			16.67	
14.678	38.53	18491.07	270.4	1.74	16.25	2.55
14.7727	1.44	18710.76	87.8	1.08	16.37	1.93
14.8674	1105.34	16623.92	1070.74	4.32	15.12	2.56
14.9621	735.77	18004.31	59.92	3.99	15.05	2.14
15.0568		18800			14.8	
15.1515		18786.33	13.67		12.83	1.69
15.2462		18800			11.88	
15.3409		18728.54	71.46		10.86	2.08
15.4356	0.18	18799.82		0.44	14.43	
15.4979	140.72	17961.79	697.49	2.76	15.95	3.52
15.5036	3276.31	11640.5	3883.19	3.07	20.81	3.78
15.5038	Bridge					
15.5066	4476.21	10021.75	4302.04	2.39	14.18	2.78
15.5104	3168.43	13541.34	2090.23	2.01	7.65	2.38
15.5303	841.15	17588.77	370.08	1.8	7.57	1.89
15.625		18800			14.54	
15.7197		18786.75	13.25		15.08	1.63
15.8144		18158.08	641.92		14.86	3.45
15.9091		15199.03	0.97		16.27	1.05
16.0038		15200			15.96	
16.0985		15200			15.56	
16.1932		15200			13.73	
16.2879		15200			17.13	
16.3826		15200			17.02	

Exhibit B14

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
11.2586	Bridge					
11.2678		36700			9.4	
11.3636	426.67	33351.66	2921.67	2.71	13.9	3
11.4583	112.19	36242.04	345.77	1.7	10.4	3.26
11.553	194.51	35645.61	859.88	2.66	9.99	3.68
11.6477	334.64	34764.59	100.77	2.88	9.95	1.81
11.7424	1.35	34706.11	492.55	0.48	9.21	2.04
11.8371	1169.23	28082	5948.78	2.5	10.81	5.15
11.9318	15.32	26351.21	8833.46	1.15	10.72	5.95
12.0265	349.58	22674.16	12176.26	3.15	12.54	6.06
12.1212	890.66	34309.34		2.91	12.16	
12.2159	1671.01	33110.43	418.56	4.28	11.41	3.09
12.3106	72.05	25666.19	9461.76	2.55	11.89	7.15
12.4053		24725.27	10474.73		11.79	5.64
12.5	15.69	27419.97	7764.34	2.39	13.18	5.56
12.5947	401.82	31278.61	3519.58	3.35	13.92	3.64
12.6894	1656.19	33274.91	268.9	3.13	11.01	3.2
12.7841	3278.21	30715.97	1205.82	3.66	10.67	3.33
12.8788	3396.72	31803.28		3.58	11.23	
12.9735	8482.1	26717.83	0.08	4.27	12.25	0.16
13.0682	10357.5	24842.5		3.96	11.75	
13.1629	11386.18	23813.81		3.59	12.67	
13.2576	1089.63	34110.37		2.47	14.5	
13.3523	5191.63	30008.37		2.83	11.16	
13.447	3483.85	31687.7	28.45	2.14	11.81	1.89
13.5417	4558.39	30336.62	304.99	2.08	11.97	1.98
13.6364	13639.93	21560.07		3.26	11.97	
13.7311	10111.39	25088.61		2.99	12.41	
13.8258	10325.06	24247.52	627.42	3.4	12.67	2.76
13.9205	5497.12	29685.09	17.8	3.43	13.1	1.29
14.0152	3602.05	31597.94		4	15.1	
14.1098	3329.84	31866.96	3.2	2.53	13.03	0.93
14.1335	2483.34	32716.66		2.51	12.61	
14.1761		35200			13.53	
14.2045	62.1	35137.9		0.95	13.91	
14.2992		35200			16.07	
14.3939	3042.47	32157.53		4.11	16.93	
14.4886	3428.41	31763.74	7.85	4.1	16.56	0.68
14.5833	210.93	34989.07		2.44	19.26	
14.678	889.55	31995.88	2314.58	3.63	17.9	2.98
14.7727	83.77	31823.76	3292.47	2.68	17.07	2.91
14.8674	2452.47	26207.91	6539.63	5.26	16.74	3.6
14.9621	2217.25	32406.69	576.06	5.39	17.21	2.92
15.0568		35147.96	52.04		17.31	1.48
15.1515	3.42	35007.68	188.9	0.69	14.61	2.66
15.2462	224.8	34918.29	56.91	1.9	13.06	1.78
15.3409		34813.36	386.64		12.63	3.64
15.4356	143.92	35037.46	18.62	2.13	16.64	1.81
15.4979	1243.68	31335.3	2621.02	2.92	18.31	5.21
15.5036	8579.85	18458.46	8161.68	4.37	25.61	5.06
15.5038	Bridge					
15.5066	10100.45	16449.08	8650.47	3.23	18.15	3.73
15.5104	8015.1	22915.55	4269.35	2.66	9.74	3.09
15.5303	2788.07	31471.74	940.19	2.34	9.72	2.48
15.625	55.51	34907.39	237.09	1.97	13.17	2.1
15.7197		34929.97	270.03		18.06	3.25
15.8144		32663.28	2536.72		17.57	4.58
15.9091		27828.02	71.97		19.48	2.93
16.0038		27900			19.17	
16.0985		27872.89	27.11		18.4	2.2
16.1932	107.65	27792.35		2.29	16.36	
16.2879		27900			19.9	
16.3826		27900			19.84	

Exhibit B12
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Distributed values
50-year event

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
-0.2258		67900			2.99	
-0.1784		67900			4.62	
-0.1311		67900			7	
-0.0837		67900			12.74	
-0.0364		67900			13.31	
0.0477	22.76	67446.61	430.62	1.95	13.64	1.59
0.1052	493.24	48810.5	18596.26	1.81	7.28	2.09
0.1383	1416.62	47531.14	18952.25	2.12	7.6	2.18
0.1909		59634.79	8265.21		9.19	1.18
0.191	Mult Open					
0.1945		58980.45	8919.55		8.71	1.13
0.3579		67575.17	324.83		13.29	1.78
0.4394		67900			13.42	
0.4395	Mult Open					
0.464		67900			8.57	
0.5204	1883.02	54935.44	11081.55	2.28	12.6	2.4
0.5922		67900			12.26	
0.5923	Mult Open					
0.6028		67900			8.9	
0.6629	4837.47	48266.91	14795.62	3.21	12.97	2.63
0.7577	9053.93	40732.59	18113.48	3.72	14.2	3.06
0.8523	7565.3	55534.66	4800.04	3.2	14.35	1.84
0.947	10620.61	54535.52	2743.87	4.14	16.02	2.2
1.0417	5645.56	62224.91	29.53	3.57	14.88	0.75
1.1364	8350.31	58289.54	1260.15	5.38	19.64	2.94
1.2311	2768.03	64812.16	319.81	3.68	12.19	1.16
1.3258	3374.1	63571.79	954.12	3.24	14.42	2.44
1.4205	1126.52	64958.46	1815.01	1.72	9.84	1.62
1.5152	37.01	67862.98	0	1.33	8.86	0.09
1.6098	534.08	66477.52	888.4	2.87	15.84	3.66
1.7045	1143.52	53884.21	12872.27	3.89	17.97	5.8
1.7992		62375.62	5524.38		19.31	5.27
1.8939	117.87	61498.88	6283.25	0.79	11.92	4.04
1.9886		63097.3	4802.71		11.27	3.3
2.0827	3486.63	60616.08	3797.29	3.25	12.89	2.9
2.178	1368.72	54954.27	11577	2.86	14.57	4.62
2.2727	150.07	67749.93		1.23	8.36	
2.3674		67900			11.94	
2.4621		67900			10.79	
2.5568	6492.25	61407.75		4.94	17.81	
2.6515	6725.13	61174.87		5.39	19.19	
2.7462	1196.22	66624.58	79.2	3.89	19.97	2.74
2.8409		67900			14.45	
2.9356		67435.56	464.44		17.07	2.36
3.0303		67900			20.35	
3.125		67900			21.55	
3.1546		67900			21.75	
3.1547	Bridge					
3.1591		67900			18.33	
3.178		67900			18.75	
3.2197		67900			20.8	
3.4091	781.63	67118.37		2.18	13.73	
3.5038		67900			18.27	
3.5985		67900			19.72	
3.6932		67900			18.96	
3.7879		67900			18.2	
3.8826		66894.33	1005.67		17.09	3.11
3.9773	138.52	67761.48		1.86	18.68	
4.072		67900			20.11	
4.1667	1160.31	66739.69		2.91	14.52	
4.2614	699.97	65869.97	1330.06	2.7	13.73	3.05
4.3561	253.57	67645.63	0.8	2.56	15.04	0.38
4.4508	245.54	67628.95	25.5	3.5	17.55	0.67
4.5455		62850.3	5049.7		16.24	2.82
4.6402	481.31	53997.81	5220.88	3.37	16.36	2.62
4.7348	19.55	54156.94	5523.52	0.82	16.48	2.76
4.8295		58485.8	1214.2		18.11	2.03
4.9242		59690.36	9.64		20.18	0.71
5.0189		59700			21.92	

Exhibit B12

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
5.1136	7974.45	51725.55		2.49	10.25	
5.303	100.1	59599.9		0.71	11.95	
5.3977	39.18	59659.01	1.81	0.71	15.24	0.4
5.4924		58828.57	871.43		17.8	3.2
5.5871	44.5	59395.43	260.06	1.4	16.73	1.97
5.6818	1468.26	58225.03	6.71	4.19	19.68	0.81
5.7765	1483.47	58216.53		3.74	20.63	
5.8712	280.47	59419.53		1.68	20.08	
5.9301	4626.44	54776.4	297.16	1.98	9.33	1.69
5.9475	2117.17	57582.84		0.94	10.21	
5.9476	Bridge					
5.9536	919.33	58780.67		0.67	10.25	
5.9742	3031.3	56634.38	34.32	2.24	9.85	1.24
6.0606	1717.96	57886.9	95.14	1.88	9.39	1.76
6.1553	1134.25	58477.11	88.65	1.6	7.44	1.42
6.3447	31.52	54910.14	4758.34	1.06	15.09	4.04
6.4394	1.98	55421.9	1176.12	0.6	13.66	2.35
6.5341	1395.75	52464.24	2740.01	3.22	14.39	3.37
6.6288	1097.78	52685.95	2816.27	2.98	13.83	3.17
6.7235		56600			11.78	
6.8182	377.88	56011.58	210.54	2.75	13.44	1.89
6.8389		56135.94	464.06		11.33	2.06
6.9129		56600			12.91	
7.0076		56600			13.14	
7.1023		56583.87	16.13		11.91	2.06
7.197		56600			11.95	
7.2917	4115.29	52484.71		2.57	10.69	
7.3864	1041.68	43675.69	11882.63	1.76	13.68	4.47
7.4811		54124.37	2475.63		14.26	2.48
7.5758		51237.76	5362.24		14.06	2.91
7.6705		56293.64	306.36		16.25	1.5
7.7652		56564.41	35.58		16.13	0.97
7.8598	36.92	56563.08		1.69	12.94	
7.9545		56600			8.3	
8.0492	1431.5	23368.5		4.36	12.98	
8.1439	471.13	24328.87		2.21	11.17	
8.2386	391.64	22634.4	1773.96	1.9	11.27	3.35
8.3333	2918.44	21881.56		4.33	10.84	
8.428	8354.26	16445.71	0.03	5.46	9.67	0.26
8.5227	744.21	24055.79		2.21	9.66	
8.6174	93.32	24478.38	228.3	2.03	9.56	1.6
8.7121		24617.59	182.41		9.39	2.51
8.8068	605.14	17598.6	6596.26	3.05	8.53	4.3
8.9015	3100.12	15392.02	6307.86	3.66	7.29	3.86
8.9962	1705.49	6849.07	16245.44	3.01	7.39	5.47
9.0909	3712.65	13499.34	7588.01	3.26	7.94	2.87
9.1857	7193.11	7725.58	9881.32	4.64	8.88	4
9.2804	3028.88	13011.44	8759.69	3.48	11.75	2.81
9.3264	36.93	16370.6	8392.46	0.98	12.32	2.27
9.3786		24800			16.7	
9.3787	Bridge					
9.3864		24800			12.78	
9.4009		24800			10.26	
9.4697		24800			13.62	
9.5644	0.4	24799.6		0.88	14.17	
9.6591	0.57	24799.43		0.97	14.17	
9.7538		24800			12.6	
9.8485		24800			10.45	
9.9432	9.41	24790.59		1.13	11.13	
10.0379		24800			10.97	
10.1326		24800			9.95	
10.2273	2.09	24797.91		0.59	11.76	
10.322		20013.48	4786.52		10.32	6.12
10.4167		23691.37	1108.63		10.08	2.42
10.5114		21260.62	3539.38		10.67	3.04
10.6061	517.05	23420.46	862.49	2.4	11.22	3.11
10.7008	845.57	23311.05	643.37	2.81	11.14	2.31
10.7955	343.28	22627.39	1829.34	1.81	10.71	4.25
10.8902	2095.57	16947.26	5757.17	4	11.7	4.96
11.0795	0	24426.12	373.88	0.11	11.45	2.1
11.1742	368.08	24431.93		1.55	10.11	
11.2585		24800			10.69	

Exhibit B12

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
11.2586	Bridge					
11.2678		24800			8.31	
11.3636	151.04	23643.07	1005.89	2.77	13.25	3.46
11.4583	16.23	24599.32	184.44	1.17	9.26	2.75
11.553	66.8	24184.16	549.04	1.93	9.17	3.59
11.6477	140.84	23839.79	19.37	2.22	9.06	1.25
11.7424		23714.63	285.37		8.65	2.62
11.8371	325.49	19656.75	4017.77	1.67	9.74	4.36
11.9318	0	17719.83	6280.17	0.17	9.64	5.21
12.0265	171.24	16429.87	7398.89	2.69	11.52	4.66
12.1212	298.26	23701.74		2.3	11.19	
12.2159	1021.07	22836.75	142.17	3.71	10.33	2.08
12.3106	15.34	17371.96	6612.69	1.71	11.15	6.15
12.4053		15569.94	8430.06		10.19	6.02
12.5	5.05	18698.76	5296.19	1.82	11.39	4.86
12.5947	197.46	22209.72	1592.81	2.95	12.45	3.06
12.6894	805.8	23048.46	145.74	2.65	9.87	2.9
12.7841	1922.22	21495.94	581.85	3.15	9.6	2.59
12.8788	1892.21	22107.79		3.34	10.55	
12.9735	5182.91	18817.09		4.06	11.18	
13.0682	6161.32	17838.68		3.3	10.36	
13.1629	6232.08	17767.92		3.36	11.89	
13.2576	158.6	23841.4		1.77	13.75	
13.3523	2116.47	21883.53		2.53	10.75	
13.447	740.56	23257.41	2.03	2.1	12.05	1.11
13.5417	1438.45	22544.55	17	3.09	12.25	1.13
13.6364	7004.93	16995.07		2.41	10.93	
13.7311	5008.11	18991.89		2.4	11.18	
13.8258	5055.49	18707.41	237.1	2.52	11.53	1.87
13.9205	2460.63	21539.38		2.47	11.53	
14.0152	2194.63	21805.37		4.1	13.53	
14.1098	634.28	23365.72		1.58	12.42	
14.1335	225.21	23774.79		1.44	12.83	
14.1761		24000			11.1	
14.2045		24000			12.59	
14.2992		24000			14.28	
14.3939	1409.26	22590.74		3.84	15.76	
14.4886	1409.62	22590.38		3.53	15.44	
14.5833		24000			17.71	
14.678	164.47	23177.51	658.02	2.41	17.24	3.06
14.7727	17.72	23450.98	531.3	2.04	16.6	2.26
14.8674	1583.01	19964.66	2452.33	4.66	15.49	2.71
14.9621	1176.42	22656.86	166.72	4.59	15.97	2.74
15.0568		24000			15.8	
15.1515		23952.46	47.54		13.41	2.27
15.2462	4.53	23995.47		0.79	12.64	
15.3409		23834.02	165.99		11.36	2.77
15.4356	14.67	23985.33		1.64	15.26	
15.4979	279.81	22458.3	1261.89	2.75	17	4.27
15.5036	4894.8	13816.35	5288.85	3.44	22.1	4.19
15.5038	Bridge					
15.5066	6274.47	12079.66	5645.87	2.75	15.55	3.11
15.5104	4562.43	16659.79	2777.78	2.26	8.45	2.66
15.5303	1335.68	22128.04	536.28	2.11	8.39	2.12
15.625	2.59	23997.41		1.16	13.69	
15.7197		23942.74	57.26		16.19	2.29
15.8144		22944.48	1055.52		16.05	3.73
15.9091		18791.48	8.52		17.34	1.77
16.0038		18800			17.04	
16.0985		18799.66	0.34		16.26	0.76
16.1932	4.5	18795.5		1.02	14.52	
16.2879		18800			18.11	
16.3826		18800			18	

Exhibit B13
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Distributed values
100-year event

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
-0.2258		78900			3.48	
-0.1784		78900			5.38	
-0.1311		78900			8.2	
-0.0837		78900			13.44	
-0.0364		78900			13.99	
0.0477	44.46	77696.22	1159.33	2.23	14.06	2.02
0.1052	636.2	54907.59	23356.2	1.94	7.55	2.27
0.1383	1761.78	53447.1	23691.12	2.27	7.91	2.37
0.1909		67958.05	10941.95		9.7	1.27
0.191	Mult Open					
0.1945		66905.72	11994.28		9	1.19
0.3579		78170.95	729.05		13.85	2.41
0.4394		78900			14.1	
0.4395	Mult Open					
0.464		78900			9.08	
0.5204	3023.29	58857.58	17019.13	2.58	12.15	2.62
0.5922		78900			12.83	
0.5923	Mult Open					
0.6028		78900			9.39	
0.6629	6123.96	52153.25	20622.79	3.41	12.95	2.9
0.7577	10872.97	45144.27	22882.76	4.07	15.06	3.42
0.8523	9779.54	60900.1	8220.36	3.52	14.81	2.23
0.947	13349.97	61014.12	4535.91	4.55	16.83	2.63
1.0417	6612.82	72167.76	119.42	3.74	16.58	0.92
1.1364	10670.64	66386.37	1843	5.87	20.64	2.79
1.2311	3442.2	74368.84	1088.97	3.9	12.76	1.24
1.3258	4840.18	72568.14	1491.68	3.67	14.98	2.58
1.4205	1911.67	73759.6	3228.73	2.06	10.11	1.97
1.5152	75.52	78798.05	26.43	1.5	9.01	0.49
1.6098	885.49	76773.3	1241.21	3.44	16.62	3.99
1.7045	1603.37	61558.8	15737.83	4.39	18.95	6.27
1.7992	106.45	71393.05	7400.49	0.98	19.91	5.76
1.8939	567.33	70697.59	7635.08	1.4	12.53	4.33
1.9886		72641.88	6258.13		11.47	3.45
2.0827	4944.58	68153.18	5802.24	3.49	12.74	3.22
2.178	2160.92	62327.83	14411.25	3.31	14.93	4.88
2.2727	300.61	78599.4		1.57	8.77	
2.3674		78900			12.53	
2.4621		78900			11.52	
2.5568	8462.21	70437.79		5.45	18.73	
2.6515	8769.75	70130.25		5.92	20.08	
2.7462	1863.27	76891.46	145.27	4.48	20.94	3.15
2.8409		78900			15.06	
2.9356		77930.14	969.86		17.81	2.96
3.0303		78900			21.21	
3.125		78900			22.46	
3.1546		78900			22.73	
3.1547	Bridge					
3.1591		78900			19.1	
3.178		78900			19.13	
3.2197		78900			21.65	
3.4091	1543.88	77356.13		2.56	14.18	
3.5038		78900			18.64	
3.5985		78900			20.53	
3.6932		78900			20.09	
3.7879		78899.99	0		18.91	0.18
3.8826	17.3	77210.2	1672.49	0.84	17.79	3.62
3.9773	441.07	78458.93		2.76	19.38	
4.072		78900			20.86	
4.1667	1827.53	77072.47		3.34	15.08	
4.2614	1655.65	75272.6	1971.75	2.2	13.77	3.19
4.3561	502.79	78307.16	90.05	3.08	15.13	0.85
4.4508	430.76	76796.47	1672.77	3.56	16.58	1.87
4.5455		71002.98	7897.01		16.97	3.3
4.6402	638.85	60692.15	8369	3.63	17.16	3.16
4.7348	138.8	60819.97	8741.23	1.53	17.15	3.23
4.8295		66363.07	3336.93		18.39	2.82
4.9242	9.29	69390.23	300.48	0.66	20.62	2.19
5.0189		69700			22.71	

Exhibit B13

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
5.1136	11569.58	58130.42		2.73	10.29	
5.303	1431.38	68268.62		1.64	11.46	
5.3977	587.23	68971.28	141.49	1.84	15.51	1.66
5.4924	1.1	68220.73	1478.17	0.49	18.57	3.74
5.5871	124.02	69051.68	524.29	2.08	18.19	2.63
5.6818	2144.23	67466.98	88.79	4.54	20.55	2.13
5.7765	2317.09	67382.91		4.35	21.98	
5.8712	1805.24	67894.73	0.02	2.78	19.93	0.21
5.9301	6704.04	62567.16	428.8	2.27	9.8	1.8
5.9475	3758.52	65941.48		1.12	10.64	
5.9476	Bridge					
5.9536	2345.94	67354.06		0.92	10.66	
5.9742	4499.69	65133.41	66.9	2.54	10.19	1.42
6.0606	2954.6	66601.77	143.63	2.21	9.46	1.84
6.1553	1916.85	67625.08	158.08	1.72	7.27	1.49
6.3447	348.14	62467.98	6883.89	1.69	14.32	3.76
6.4394	15.36	64759.73	1824.91	1.04	14.9	2.81
6.5341	2033.13	61823.97	2742.9	3.64	15.38	2.44
6.6288	1530.8	62128.72	2940.48	3.17	14.84	2.39
6.7235		66597.91	2.09		12.28	0.52
6.8182	602.22	65564.28	433.5	3.21	14.07	2.13
6.8389		65928.92	671.08		12.18	2.1
6.9129		66600			13.64	
7.0076		66600			14.12	
7.1023		66572.84	27.16		12.3	2.25
7.197		66600			12.88	
7.2917	5998.97	60601.03		2.69	10.84	
7.3864	1994.44	50514.36	14091.2	2.2	14.76	4.76
7.4811		62381.37	4218.63		14.92	2.98
7.5758		58668.41	7931.59		14.82	3.39
7.6705		65049.59	1550.41		16.32	2.12
7.7652	3556.62	62506.05	537.32	4.31	15.57	1.41
7.8598	38.68	66561.32		1.97	15.5	
7.9545		66599.42	0.57		8.35	0.56
8.0492	1737.06	26562.94		4.26	13.37	
8.1439	673.14	27626.87		2.19	11.47	
8.2386	662.2	25435.15	2202.65	2.06	11.48	3.5
8.3333	3352.87	24947.13		4.31	11.35	
8.428	9220.59	19077.89	1.53	5.64	10.2	0.68
8.5227	1016.33	27283.67		2.3	10.14	
8.6174	126.9	27839.93	333.17	2.22	10.07	1.72
8.7121		28047.35	252.65		9.48	2.75
8.8068	722.73	20093.38	7483.88	3.22	8.9	4.5
8.9015	3623.65	17446.59	7229.77	3.89	7.32	4.04
8.9962	2175.97	8372.19	17751.84	3.31	7.89	5.6
9.0909	4231.9	15105.71	8962.38	3.42	8.22	3.06
9.1857	8092.98	8776.09	11430.94	4.84	9.11	4.18
9.2804	3476.37	14714.57	10109.06	3.52	12.44	2.96
9.3264	107.07	18121.56	10071.37	1.16	12.79	2.42
9.3786		28300			17.34	
9.3787	Bridge					
9.3864		28300			13.77	
9.4009		28300			10.83	
9.4697		28300			13.65	
9.5644	1.27	28298.73		1.17	14.79	
9.6591	1.69	28298.31		1.26	14.75	
9.7538		28300			13.14	
9.8485		28300			10.71	
9.9432	23.97	28276.03		1.4	11.59	
10.0379		28300			11.45	
10.1326		28300			10.37	
10.2273	19.29	28280.71		0.97	12.23	
10.322		22928.53	5371.48		10.53	6.39
10.4167		26860.62	1439.38		10.41	2.46
10.5114		23937.84	4362.16		11.11	3.2
10.6061	685.51	26536.1	1078.39	2.51	11.81	3.08
10.7008	1093.88	26287.43	918.69	3.03	11.61	2.49
10.7955	566.79	25633.06	2100.15	2.17	11.15	4.31
10.8902	2463.76	19259.71	6576.54	4.28	12.54	5.29
11.0795	1.99	27712.83	585.18	0.65	11.96	2.38
11.1742	647.47	27652.53		1.9	10.51	
11.2585		28300			10.89	

Exhibit B13

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
11.2586	Bridge					
11.2678		28300			8.66	
11.3636	208.16	26728.88	1362.96	2.91	13.66	3.36
11.4583	37.45	28028.79	233.75	1.33	9.47	2.89
11.553	100.58	27579.72	619.7	2.19	9.44	3.53
11.6477	189.03	26875.54	35.42	2.43	9.35	1.45
11.7424		26772.93	327.07		9	2.59
11.8371	502.58	21946.78	4650.64	1.97	10.15	4.73
11.9318	0.7	20111.66	6987.64	0.66	9.75	5.33
12.0265	221.06	18112.99	8765.95	3.09	12.1	5.24
12.1212	448.84	26651.16		2.58	11.42	
12.2159	1207.69	25677.15	215.15	3.91	10.63	2.44
12.3106	30.65	19490.51	7578.84	2.03	10.99	6.48
12.4053		19005.67	8094.33		11.03	5.18
12.5	7.99	21115.17	5976.83	1.98	11.67	4.95
12.5947	244.02	24770.06	2085.92	3.02	12.88	3.2
12.6894	1022.75	25899.4	177.86	2.79	10.19	3.04
12.7841	2294.05	24082.79	723.16	3.38	10.07	2.81
12.8788	2308.28	24791.72		3.39	10.58	
12.9735	6082.85	21017.15		4.1	11.44	
13.0682	7320.67	19779.33		3.54	10.81	
13.1629	7351.98	19748.02		3.57	12.66	
13.2576	354.59	26745.41		2.02	13.81	
13.3523	3029.07	24070.93		2.54	10.51	
13.447	1138.03	25956.22	5.74	2.13	12.24	1.41
13.5417	1761.02	25310.76	28.22	3.19	13.17	1.2
13.6364	8981.91	18118.09		2.64	10.99	
13.7311	6283.72	20816.28		2.62	11.69	
13.8258	6606.6	20147.87	345.54	2.8	11.73	2.16
13.9205	3364.25	23735.05	0.7	2.82	11.88	0.47
14.0152	2591.07	24508.93		4.33	14.26	
14.1098	1235.77	25864.23		2	12.72	
14.1335	874.63	26225.37		1.79	11.95	
14.1761		27100			11.98	
14.2045		27100			13.07	
14.2992		27100			14.85	
14.3939	1814.93	25285.07		3.9	16.05	
14.4886	1738.87	25361.13		3.74	16.37	
14.5833	6.47	27093.53		0.98	18.25	
14.678	267.7	25860.34	971.96	2.51	17.74	3.18
14.7727	30.3	26083.53	986.17	2.3	17.08	2.53
14.8674	1851.62	21637.53	3610.85	4.75	15.45	2.86
14.9621	1449.1	25398.18	252.72	4.87	16.45	3.03
15.0568		27100			16.31	
15.1515		27025.22	74.78		13.8	2.53
15.2462	26.66	27073.34	0.01	1.21	12.95	0.15
15.3409		26872.1	227.9		11.67	3.08
15.4356	35.32	27064.68		2.03	15.61	
15.4979	391.36	25132.54	1576.1	2.95	17.91	4.67
15.5036	5888.33	15131	6080.68	3.72	23.14	4.45
15.5038	Bridge					
15.5066	7375.41	13274.11	6450.49	2.93	16.29	3.29
15.5104	5169.21	18703.09	3227.7	2.25	9.01	2.84
15.5303	1659.4	24799.16	641.44	2.24	8.84	2.24
15.625	10.15	27082.56	7.29	1.51	13.46	0.7
15.7197		27000.29	99.71		16.78	2.6
15.8144		25720.57	1379.43		16.38	3.79
15.9091		21579.29	20.71		18.1	2.19
16.0038		21600			17.77	
16.0985		21596.58	3.42		16.96	1.33
16.1932	21.88	21578.12		1.56	15.14	
16.2879		21600			18.71	
16.3826		21600			18.62	

Exhibit B14
Ventura River Hydraulic Model from below Matilija Dam to Pacific Ocean
Distributed values
500-year event

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
-0.2258		105500			4.65	
-0.1784		105500			7.25	
-0.1311		105500			11.29	
-0.0837		105500			14.76	
-0.0364		105500			15.43	
0.0477	124.66	100798.9	4576.49	2.7	14.77	2.5
0.1052	1000.56	69542.43	34957.01	2.21	8.22	2.64
0.1383	2616.81	67602.08	35281.12	2.59	8.65	2.76
0.1909		87313.49	18186.51		10.77	1.44
0.191	Mult Open					
0.1945		85988.73	19511.27		10.06	1.35
0.3579		103563	1937.04		15.21	3.5
0.4394		105500			15.52	
0.4395	Mult Open					
0.464		105500			10.15	
0.5204	5887.15	67362.6	32250.26	2.92	11.21	2.82
0.5922		105500			14.1	
0.5923	Mult Open					
0.6028		105500			9.76	
0.6629	9480.31	58790.54	37229.15	3.51	11.86	3.2
0.7577	15373.79	54883.36	35242.85	4.78	16.65	4.16
0.8523	14701.07	74034.2	16764.73	4.22	16.36	3.02
0.947	20030.54	75369.66	10099.79	5.33	18.32	3.46
1.0417	10830.52	93600.88	1068.6	4.81	19.56	1.74
1.1364	16592.21	81637.24	7270.55	6.33	20.68	3.11
1.2311	4932.38	95531.53	5036.08	4.44	14.26	2.22
1.3258	8170.75	94449.37	2879.89	4.56	16.98	2.79
1.4205	3947.16	94585.27	6967.57	2.64	10.87	2.56
1.5152	184.81	104634.7	680.49	1.16	9.48	1.39
1.6098	2134.49	100472.9	2892.64	4.3	16.87	3.06
1.7045	2846.01	79813.37	22840.63	5.35	21	7.22
1.7992	2765	91025.76	11709.24	3.09	21	6.49
1.8939	2255.28	92474.56	10770.16	2.26	14.09	4.96
1.9886	276.04	95233.18	9990.78	1.18	12.22	3.87
2.0827	7650.94	87155.63	10693.43	3.48	13.07	3.77
2.178	4520.54	79366.52	21612.94	4.01	15.12	5.17
2.2727	789.61	104708.3	2.06	2.15	9.49	0.53
2.3674	5.21	105494.8		0.56	13.09	
2.4621	8.54	105491.5		0.67	13.61	
2.5568	13487.21	92012.79		6.47	20.69	
2.6515	13786.15	91649.07	64.78	6.92	22.15	2
2.7462	3758.53	101355.1	386.38	5.57	22.98	3.94
2.8409	25.13	105474.9		0.69	16.28	
2.9356		102679.9	2820.09		19.12	4.26
3.0303		105500			22.86	
3.125		105500			24.25	
3.1546		105500			24.69	
3.1547	Bridge					
3.1591	2263.79	98108.49	5127.72	0.99	11.4	1.02
3.178	5501.23	92537.38	7461.39	1.86	9.89	2.02
3.2197	15227.3	84982.33	5290.37	2.43	9.48	1.79
3.4091	12494.33	93005.67		2.56	10.43	
3.5038	1205.02	104295		1.56	13.52	
3.5985		105500			22.12	
3.6932	307.63	105192.4		1.64	21.48	
3.7879		105476.2	23.82		21.6	1.7
3.8826	666.96	101106.7	3726.36	2.59	18.81	4.37
3.9773	1314.11	104169.8	16.07	3.39	21.11	1.47
4.072	40.64	105459.4		1.04	22.29	
4.1667	3178.05	102322		2.31	16.47	
4.2614	6173.09	95511.5	3815.41	2.68	13.69	3.37
4.3561	2156.56	98492.75	4850.7	2.78	13.86	2.48
4.4508	782.83	97483.59	7233.58	4.03	17.63	3.25
4.5455		89415.77	16084.24		18.43	4.35
4.6402	1084.92	75493.78	16521.3	3.39	18.8	4.14
4.7348	510.19	76664.94	15924.87	2.28	19.45	4.17
4.8295	16.43	83843.76	9239.82	0.51	19.47	3.88
4.9242	4912.89	85933.12	2253.99	2.36	18.6	2.11
5.0189		93100			24.23	

Exhibit B14

River Sta	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Left (ft/s)	Vel Chnl (ft/s)	Vel Right (ft/s)
5.1136	19813.4	73286.6		2.7	10.55	
5.303	6197.03	86902.97		2.16	10.76	
5.3977	3395.96	88729.37	974.68	3.12	15.18	3.05
5.4924	471.32	89168.47	3460.21	2.04	19.64	4.68
5.5871	437.98	91245.13	1416.89	3.3	20.92	3.89
5.6818	4008.75	88599.4	491.85	4.75	22.01	3.65
5.7765	5582.32	87464.05	53.64	5.2	22.99	1.94
5.8712	5891.34	87062.98	145.68	4.23	21.42	2.47
5.9301	11967.4	80281.01	851.59	2.78	10.72	2.08
5.9475	9086.13	84013.34	0.52	1.53	11.32	0.1
5.9476	Bridge					
5.9536	7633.99	85441.27	24.74	1.39	11.06	0.28
5.9742	8351.67	84512.79	235.54	3.05	10.76	1.06
6.0606	6214.96	86593.28	291.76	2.68	9.6	1.96
6.1553	4329.49	88372.66	397.86	1.91	6.99	1.56
6.3447	1332.34	80283.62	11484.05	2.51	15.64	4.53
6.4394	219.32	84228.54	4552.14	1.8	15.77	2.9
6.5341	3543.07	77937.63	7519.31	4.09	16.13	3.45
6.6288	2764.79	78740.05	7495.16	3.49	15.93	3.31
6.7235	5.88	88832.9	161.23	0.77	14.47	0.84
6.8182	1151.62	86564.2	1284.17	3.96	15.35	2.81
6.8389		87554.91	1445.1		13.84	2.52
6.9129		89000			15.09	
7.0076		89000			15.94	
7.1023		88938.37	61.63		13.08	2.59
7.197		89000			14.15	
7.2917	11301.17	77698.84		3.28	11.26	
7.3864	6964.93	62871.98	19163.09	2.89	14.75	4.38
7.4811	4205.2	76122.7	8672.1	2.48	15.1	3.63
7.5758	5634.08	70590.06	12775.85	3.5	15.74	4.02
7.6705	4119.12	80054.63	4826.24	5.06	17.17	2.87
7.7652	5438.65	80144.27	3417.09	4.27	16.63	2.67
7.8598	94.04	88905.95		1.83	18.49	
7.9545		88987.1	12.9		8.31	1.07
8.0492	2765.51	33934.5		3.32	12.63	
8.1439	1460.06	35239.94		2.9	12.38	
8.2386	1463.24	32068.13	3168.63	2.66	12.47	3.9
8.3333	4343.51	32356.49		3.66	11.59	
8.428	11517.71	25172.72	9.57	6.46	12.01	1.01
8.5227	2042.95	34657.05		2.44	10.57	
8.6174	244.2	35558.83	896.97	2.52	10.51	2.4
8.7121		36271.07	428.93		10.02	3.3
8.8068	1086.01	26175.28	9438.72	3.47	8.89	4.54
8.9015	4767.93	23166.27	8765.8	4.44	7.94	4.17
8.9962	3519.19	12800.19	20380.61	3.95	8.95	5.42
9.0909	5460.55	19180.72	12058.74	3.75	9.04	3.42
9.1857	10686.73	11281.7	14731.57	5.3	8.98	4.09
9.2804	5223.32	18912.31	12564.36	4.08	14.04	3.09
9.3264	675.93	22076.54	13947.53	1.7	13.52	2.6
9.3786		36492.46	207.54		17.78	0.53
9.3787	Bridge					
9.3864	11.85	32200	4488.14	0.25	9.27	1.01
9.4009	93.02	14588.91	22018.07	0.4	3.39	0.85
9.4697		20789.87	15910.13		5.49	1.02
9.5644	5.83	36694.17		1.68	16.09	
9.6591	7.29	36692.71		1.78	15.97	
9.7538		36700			14.26	
9.8485		36700			11.46	
9.9432	90.5	36609.5		1.91	12.57	
10.0379		36700			12.36	
10.1326		36700			11.3	
10.2273	194.55	36505.45		2.1	12.85	
10.322		30286.15	6413.85		11.33	6.69
10.4167		34219.72	2480.28		11.12	2.76
10.5114		30437.3	6262.7		12.26	3.48
10.6061	1472.11	33405.93	1821.96	3.15	12	2.35
10.7008	1767.95	33127.96	1804.1	3.42	12.42	2.77
10.7955	1200.74	32721.89	2777.36	2.85	12.09	4.51
10.8902	3268.02	24778.92	8653.06	4.18	13.41	5.57
11.0795	84.28	35333.25	1282.47	1.44	12.53	2.76
11.1742	1503.45	35196.23	0.32	2.55	11.24	0.56
11.2585		36700			11.74	

16. Exhibit D. Flood Mapping for Current Condition and With Project Condition

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17. Exhibit E. Ventura River Bed Material

Table 17.1. Ventura River bed-material sample locations.

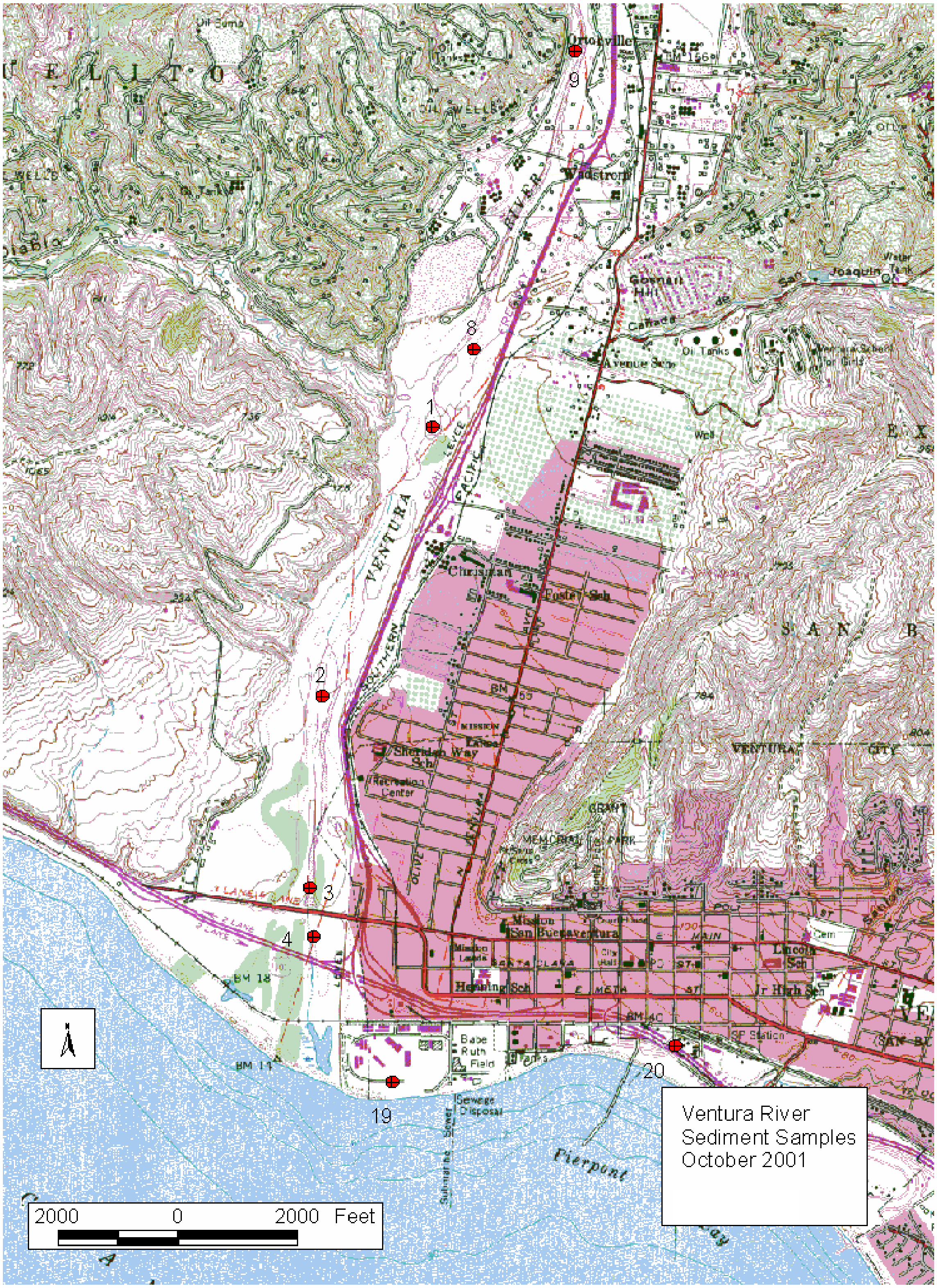
	River	Latitude			Longitude		
Sample #	Mile	degrees	minutes	seconds	degrees	minutes	seconds
19	beach	34	16	26.73	119	18	14.05
20	beach	34	16	33.37	119	17	18.06
4	0.5	34	16	50.60	119	18	29.90
3	0.6	34	16	58.60	119	18	30.80
2	1.2	34	17	30.18	119	18	28.63
1	2.2	34	18	14.53	119	18	7.80
8	2.5	34	18	27.22	119	17	59.97
9	3.4	34	19	16.50	119	17	40.70
18	4.6	34	20	14.60	119	17	48.40
7	5.1	34	20	40.93	119	17	57.31
5	6.0	34	21	15.44	119	18	33.93
6	7.5	34	22	27.64	119	18	28.88
17	8.3	34	23	9.50	119	18	42.20
16	9.8	34	24	20.30	119	18	10.92
10	11.1	34	25	26.05	119	18	8.68
13	12.8	34	26	49.00	119	17	43.77
11	13.7	34	27	32.40	119	17	29.60
12	14.4	34	28	7.38	119	17	24.61
14	15.1	34	28	43.17	119	17	32.66
15	17.9	34	29	38.44	119	19	45.95

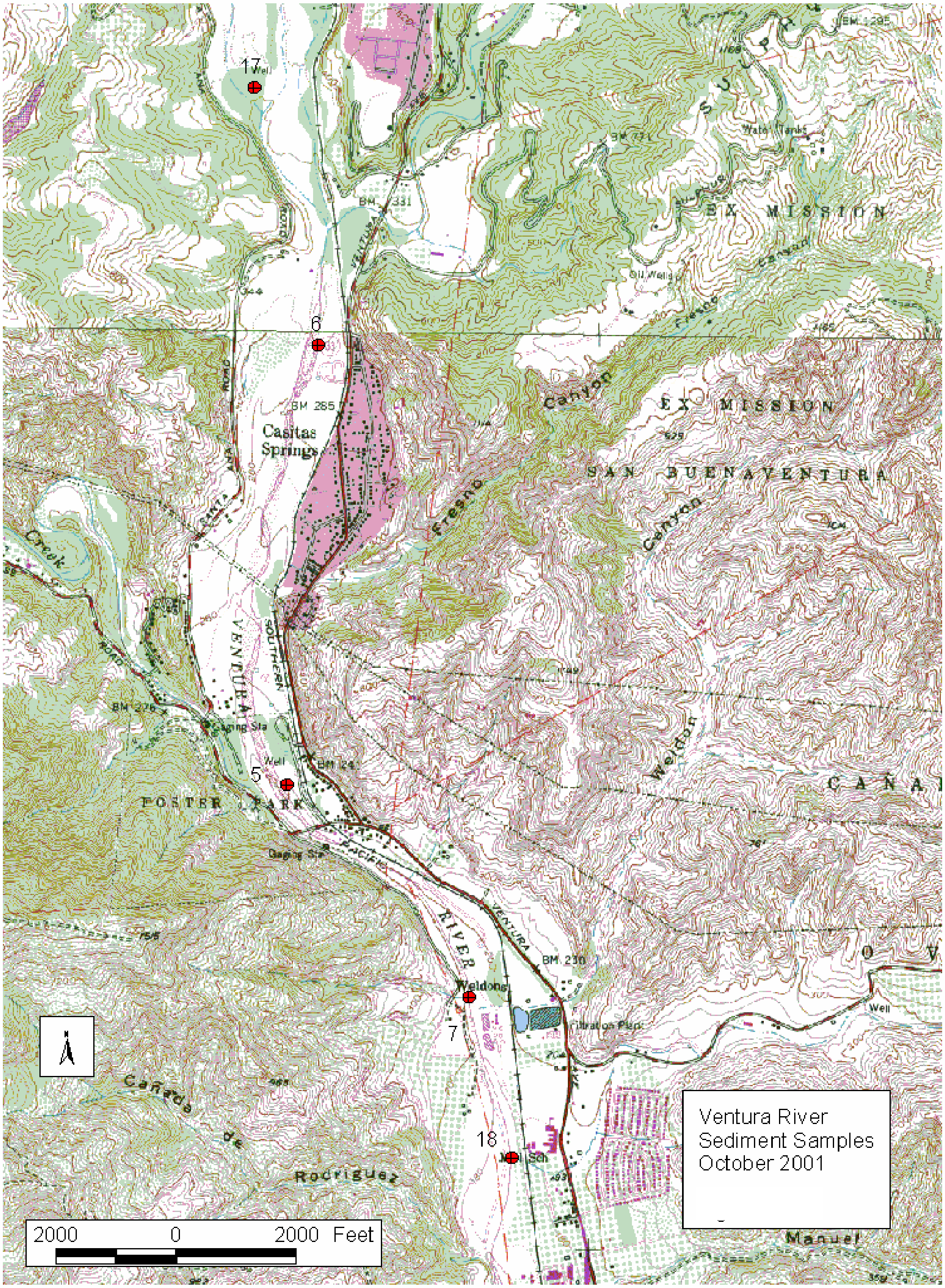
Table 17.2. Sediment gradation results. (d_{16}, d_{50}, d_{84} = diameter which 16%, 50% and 84% of the material is finer than, respectively; $d_g = \sqrt{d_{84}/d_{16}}$).

Size	Samp1	Samp2	Samp3	Samp4	Samp5	Samp6	Samp7	Samp8	Samp9	Samp10
0.0625	0	0.23	0.3	0.9	0.1	0	0	0.6	0.0	0.03
0.09	0	0.5	0.9	1.6	0.1	0	0	1.2	0.2	0.04
0.125	0	1	1.3	2.3	0.1	0	0	2.2	0.3	0.06
0.18	0	2.2	3	3.5	0.2	0	0	3.7	0.6	0.1
0.25	0	4.3	6	4.9	0.4	0	0	4.9	1.3	0.2
0.35	0	8.5	11.4	7.2	0.8	0	0	5.8	2.3	0.3
0.5	0	12.7	16.5	9.2	1.7	0	0	6.4	3.7	0.7
0.7	0	14.5	18.3	9.9	2.4	0	0	6.8	5.2	1.4
1	0	15.2	18.8	10.2	3.7	0	0	7.2	5.3	2.5
1.4	0	15.3	18.9	10.3	4.4	0	0	7.4	5.3	4
2	0	15.4	19	10.3	4.8	0	0	7.6	5.3	5.4
2.8	0	15.5	19	10.4	5.0	0	0	7.8	5.3	6.2
4	0	15.5	19	10.4	5.1	0	0	7.9	5.3	6.7
5.6	4.5	15.6	19	10.4	5.2	1.6	0	8	5.3	7
8	4.5	16.3	19	11.1	5.2	1.6	0	8.8	6	7
11	4.5	19.3	20.4	11.9	5.2	1.6	0	10.4	6	8.6
16	7.5	23	21.8	12.6	7.1	3.1	0	12	6.7	11.7
22	9	27.4	23.2	16.3	11.7	8.6	0	16	12	16.4
32	13.5	36.3	27.5	19.3	18.2	14.8	0	24.8	20	21.1
45	19.5	46.7	35.9	26.7	29.9	20.3	2.7	35.2	33.3	28.1
64	30.1	65.9	44.4	41.5	42.9	35.2	5.4	51.2	44	39.8
90	46.6	82.2	59.9	56.3	61	53.9	9.8	65.6	59.3	51.6
128	68.4	93.3	67.6	78.5	74.7	70	22.3	81.6	75.3	67.2
180	82.7	97	78.9	91.9	85	80.5	44.6	91.2	84	78.1
256	94	98.5	88.7	96.3	97.4	89.8	70.5	97.6	97.3	91.4
360	97	99.3	96.5	100	100	98.4	86.6	100	100	97.7
512	99.2	100	99.3	100	100	100	92	100	100	99.2
720	100	100	99.3	100	100	100	99.1	100	100	99.2
1024	100	100	100	100	100	100	100	100	100	100
1440	100	100	100	100	100	100	100	100	100	100
2048	100	100	100	100	100	100	100	100	100	100
2880	100	100	100	100	100	100	100	100	100	100
4096	100	100	100	100	100	100	100	100	100	100
d_{16}	39.0	6.5	0.4	16.4	25.0	41.8	107.5	16.0	26.5	16.4
d_{50}	121.2	60.2	79.6	74.0	78.7	68.7	237.9	46.2	78.7	67.0
d_{84}	245.8	120.9	213.1	156.5	132.3	224.2	270.5	165.3	128.0	219.0
d_g	2.5	4.3	24.2	3.1	2.3	2.3	1.6	3.2	2.2	3.6

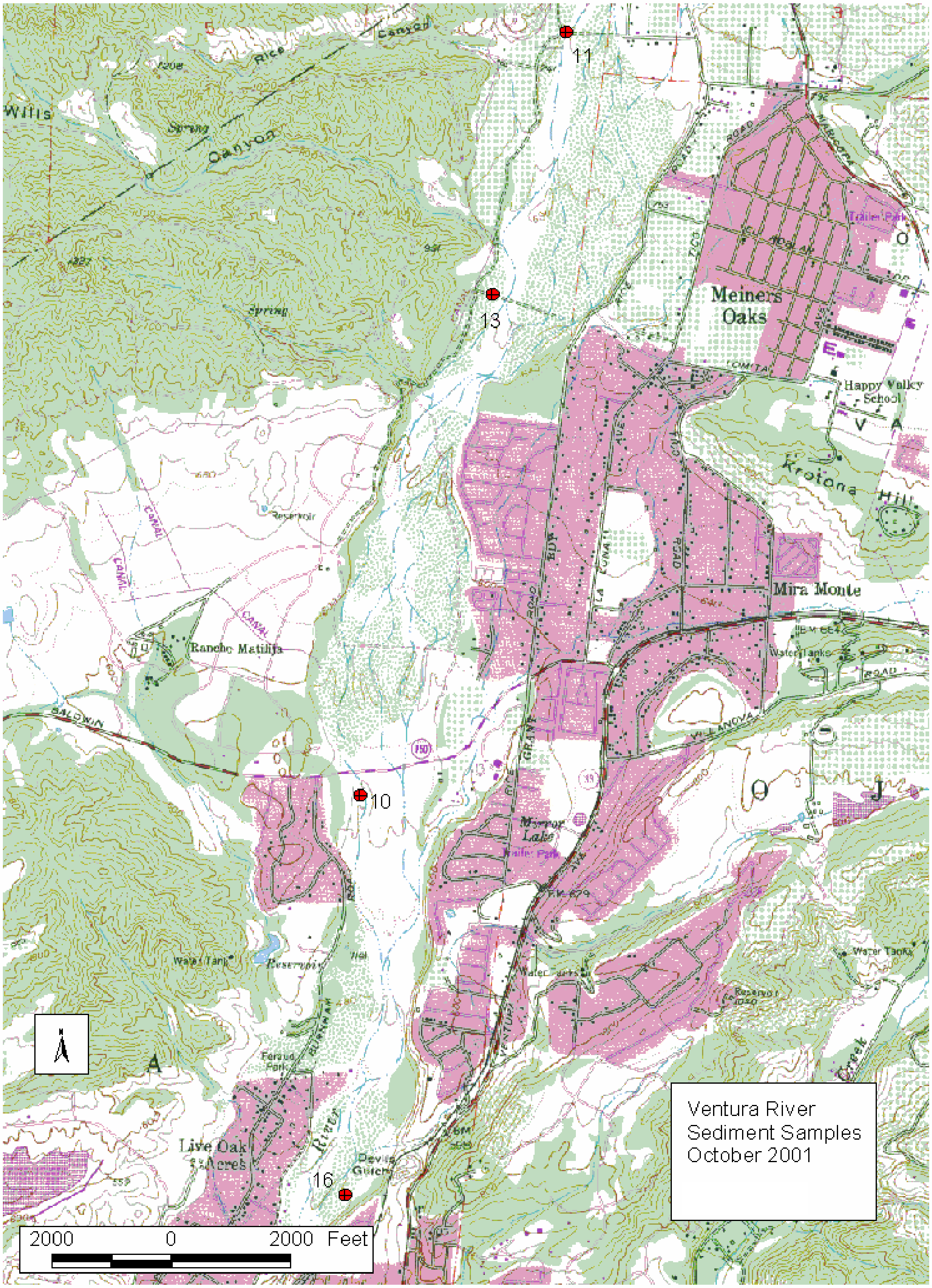
Table 17.2 (continued).

Size	Samp11	Samp12	Samp13	Samp14	Samp15	Samp16	Samp17	Samp18	Samp19	Samp20
0.0625	0	0.04	0	0	0	0	0	0.7	0	0
0.09	0	0.05	0	0	0	0	0	1.4	0.1	0.05
0.125	0	0.06	0	0	0	0	0	5.3	1.6	0.3
0.18	0	0.08	0	0	0	0	0	5.6	31.5	2.2
0.25	0	0.11	0	0	0	0	0	9.0	77	13.7
0.35	0	0.15	0	0	0	0	0	12.5	88.3	51.2
0.5	0	0.2	0	0	0	0	0	13.6	89.4	89.8
0.7	0	0.3	0	0	0	0	0	13.8	98.2	98.3
1	0	0.5	0	0	0	0	0	13.8	99.5	99.6
1.4	0	0.9	0	0	0	0	0	13.9	99.8	99.8
2	0	1.8	0	0	0	0	0	13.9	99.9	100
2.8	0	2.9	0	0	0	0	0	13.9	99.9	100
4	0	4.1	0	0	0	0	0	14.0	100	100
5.6	0	5.3	0	1.7	1.8	0	0	14.4	100	100
8	0	7.6	1.7	1.7	1.8	0	0	20.5	100	100
11	0.8	9.2	2.5	1.7	1.8	0	0	22	100	100
16	1.7	10.7	5	1.7	2.7	0	0.9	24.2	100	100
22	3.4	18.3	6.6	1.7	7.1	0.8	2.7	31	100	100
32	6.8	22.1	9.1	2.5	12.5	9.1	4.4	37.9	100	100
45	10.2	26.7	16.5	5	23.2	15.7	8	45.5	100	100
64	13.6	29	25.6	6.7	35.7	28.1	18.6	55.3	100	100
90	19.5	34.4	35.5	12.6	46.4	41.3	31	65.9	100	100
128	28.8	44.3	50.4	19.3	67.9	57	47.8	78.8	100	100
180	40.7	55	69.4	26.1	75.9	71.1	76.1	88.6	100	100
256	54.2	68.7	78.5	37.8	90.2	83.5	88.5	97	100	100
360	78.8	80.2	90	54.6	95.5	91.7	95.6	100	100	100
512	88.1	94.7	99.2	71.4	99.1	99.2	100	100	100	100
720	95.8	98.5	100	81.5	100	99.2	100	100	100	100
1024	98.3	100	100	90.8	100	100	100	100	100	100
1440	99.2	100	100	95	100	100	100	100	100	100
2048	99.2	100	100	98.3	100	100	100	100	100	100
2880	99.2	100	100	99.2	100	100	100	100	100	100
4096	100	100	100	100	100	100	100	100	100	100
d_{16}	78.3	17.6	32.7	107.0	40.3	63.5	49.1	7.3	0.15	0.34
d_{50}	200.8	150.1	90.9	281.0	120.7	105.3	175.3	54.4	0.22	0.25
d_{84}	420.5	466.9	305.8	931.5	209.7	352.6	204.5	150.2	0.28	0.37
d_g	2.3	5.1	3.1	2.9	2.3	2.4	2.0	4.5	1.4	1.0


Ventura River
Sediment Samples
October 2001
2000 0 2000 Feet
9
8
1
2
3
4
19
20



Ventura River
Sediment Samples
October 2001
2000
0
2000 Feet
17
6
5
7
18
Casitas
Springs
FOSTER PARK
VENTURA RIVER
EX MISSION
SAN BUENAVENTURA
Fresno
Canyon
Weldon
Creek
Cañada
de
Rodriguez
Manuel
Weldons
BM 285
BM 230


Ventura River
Sediment Samples
October 2001
2000 0 2000 Feet
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13
10
16
Meiners Oaks
Mira Monte
Rancho Matilija
Live Oak Acres
Happy Valley School
Krotona Hill
Mirror Lake
Canyon

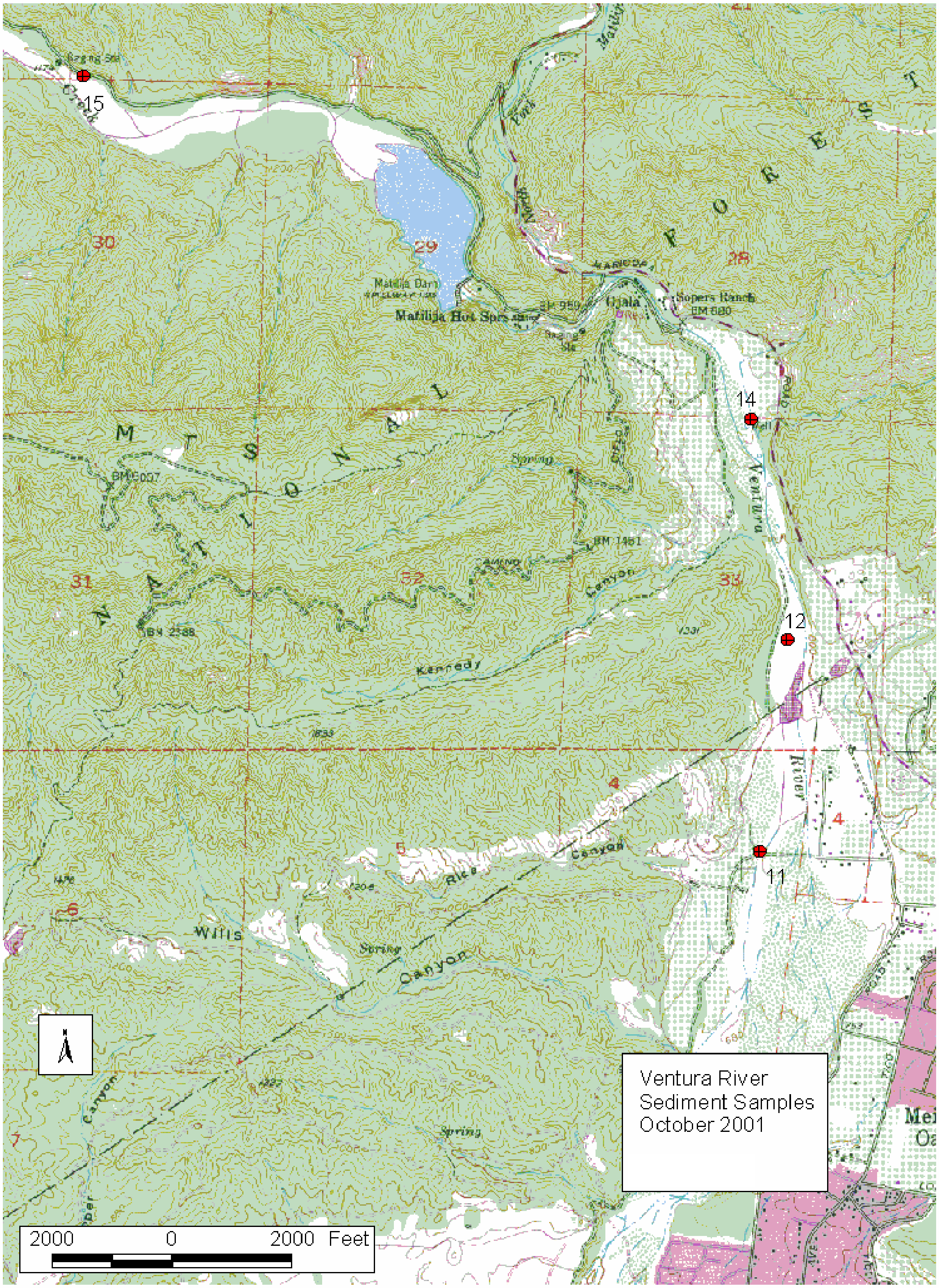


Figure 17.1. Locations of Bed Material Samples.

18. Exhibit F. Incoming sediment loads

Table 18.1. Size breakdown of incoming sediment load on Matilija Creek. Based on calibration to reservoir sediment and assumed trapping efficiency for silts and clays. Numbers in the parenthesis in the heading correspond to the size range in mm.

Flow (cfs)	Fines (<0.0625	vfs (0.062-0.125)	fs (0.125-0.25)	s (0.25-0.5)	cs (0.5-1.0)	vcs (1.0-2.0)
10	9.99E-1	1.00E-3	0.00E+0	0.00E+0	0.00E+0	0.00E+0
100	6.60E-1	2.31E-2	4.68E-2	6.32E-2	7.01E-2	6.67E-2
200	6.69E-1	2.13E-2	4.18E-2	5.48E-2	6.05E-2	6.04E-2
500	6.48E-1	2.29E-2	4.26E-2	5.27E-2	5.54E-2	5.51E-2
1000	6.11E-1	2.86E-2	5.03E-2	5.82E-2	5.69E-2	5.36E-2
2000	5.82E-1	3.68E-2	6.08E-2	6.56E-2	5.95E-2	5.18E-2
5000	5.55E-1	4.92E-2	7.54E-2	7.48E-2	6.18E-2	4.86E-2
8000	6.03E-1	4.62E-2	6.96E-2	6.78E-2	5.48E-2	4.22E-2
12500	6.33E-1	4.59E-2	6.76E-2	6.42E-2	5.04E-2	3.74E-2
15200	6.57E-1	4.41E-2	6.45E-2	6.06E-2	4.70E-2	3.45E-2
18800	6.68E-1	4.43E-2	6.39E-2	5.93E-2	4.53E-2	3.26E-2
21600	6.65E-1	4.57E-2	6.53E-2	6.01E-2	4.54E-2	3.22E-2

Flow (cfs)	vfg (2-4)	fg (4-8)	g (4-8)	cg (8-16)	vcg (16-32)
10	0.00E+0	0.00E+0	0.00E+0	0.00E+0	0.00E+0
100	4.88E-2	2.05E-2	9.58E-4	1.13E-5	1.54E-7
200	5.17E-2	3.21E-2	8.18E-3	1.75E-4	2.34E-6
500	5.16E-2	4.17E-2	2.44E-2	5.77E-3	1.40E-4
1000	4.94E-2	4.27E-2	3.12E-2	1.56E-2	2.69E-3
2000	4.51E-2	3.86E-2	3.03E-2	1.91E-2	9.09E-3
5000	3.86E-2	3.11E-2	2.46E-2	1.75E-2	1.18E-2
8000	3.27E-2	2.59E-2	2.04E-2	1.47E-2	1.04E-2
12500	2.80E-2	2.15E-2	1.66E-2	1.21E-2	9.10E-3
15200	2.54E-2	1.92E-2	1.47E-2	1.08E-2	8.25E-3
18800	2.35E-2	1.75E-2	1.33E-2	9.73E-3	7.68E-3
21600	2.29E-2	1.68E-2	1.27E-2	9.32E-3	7.53E-3

Flow (cfs)	sc (32-64)	lc (64-128)	sb (128-256)	mb (256-512)	lb (512-1024)	vlb (1024-2048)
10	0.00E+0	0.00E+0	0.00E+0	0.00E+0	0.00E+0	0.00E+0
100	5.20E-9	9.13E-11	4.60E-13	1.34E-15	9.87E-19	0.00E+0
200	7.79E-8	1.35E-9	6.79E-12	1.97E-14	1.45E-17	0.00E+0
500	4.48E-6	7.48E-8	3.77E-10	1.09E-12	8.06E-16	0.00E+0
1000	1.16E-4	1.84E-6	9.27E-9	2.69E-11	1.98E-14	0.00E+0
2000	1.64E-3	2.75E-5	1.38E-7	4.01E-10	2.95E-13	0.00E+0
5000	1.12E-2	6.05E-4	3.14E-6	9.11E-9	6.70E-12	0.00E+0
8000	1.14E-2	1.07E-3	6.01E-6	1.74E-8	1.28E-11	0.00E+0
12500	1.17E-2	2.29E-3	1.63E-5	4.72E-8	3.47E-11	0.00E+0
15200	1.12E-2	2.88E-3	2.40E-5	6.95E-8	5.11E-11	0.00E+0
18800	1.12E-2	4.21E-3	5.03E-5	1.46E-7	1.08E-10	0.00E+0
21600	1.17E-2	5.48E-3	9.89E-5	2.95E-7	2.17E-10	0.00E+0

19. Exhibit G. Model Results for All Simulations

19.1. Model Results for 1998 Flood

19.1.1. No Action Alternative

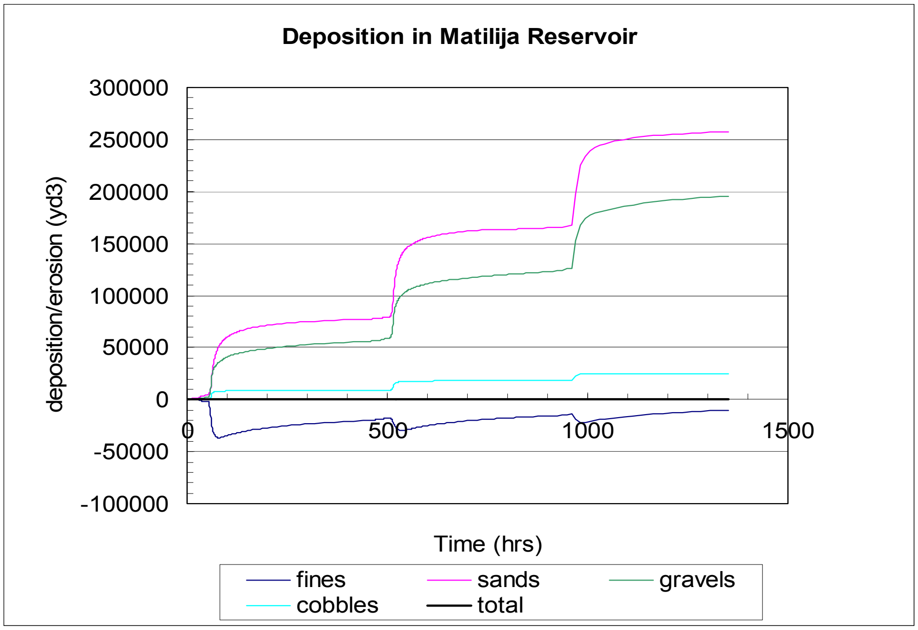


Figure 19.1. Cumulative erosion from the reservoir in No Action alternative for 1998 flood.

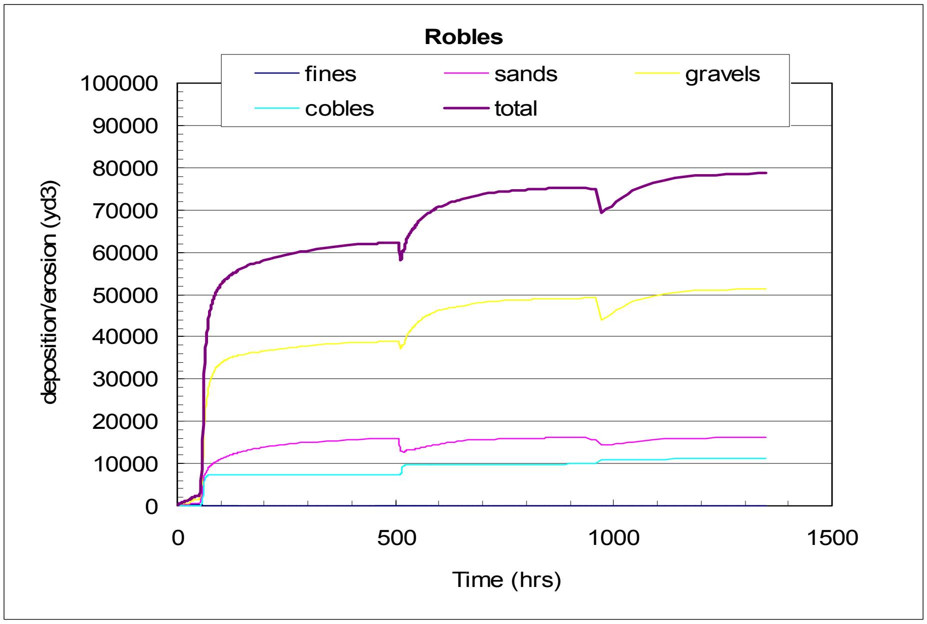


Figure 19.2. Cumulative deposition from RM 14.15 to RM 14.5 in No Action alternative for 1998 flood. Robles Diversion Dam is at RM 14.15.

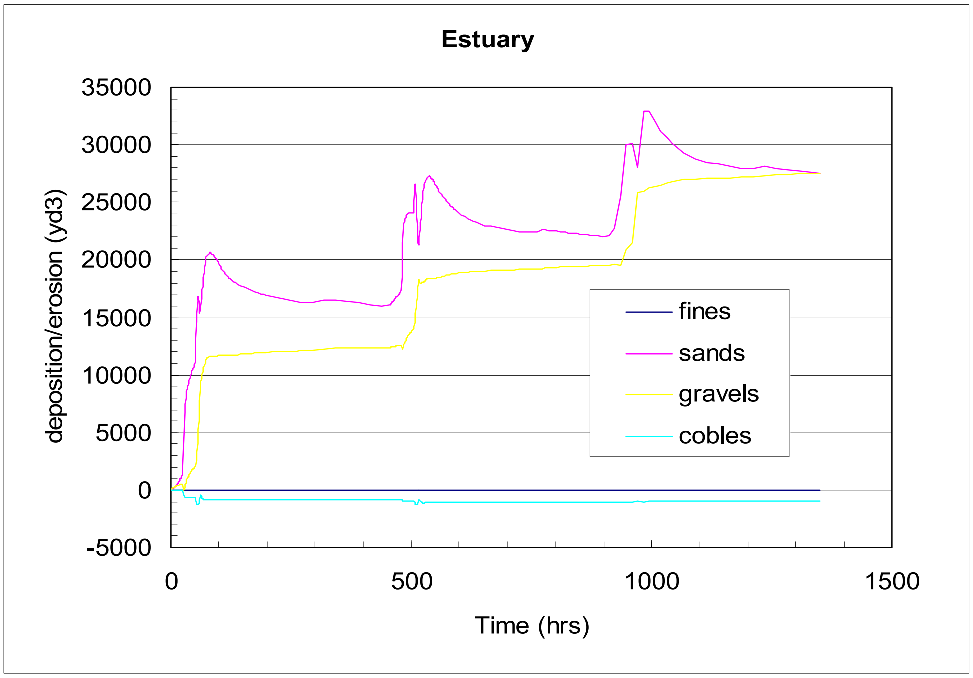


Figure 19.3. Cumulative deposition in Estuary (RM 0.6 - 0.2) in No Action alternative for 1998 flood.

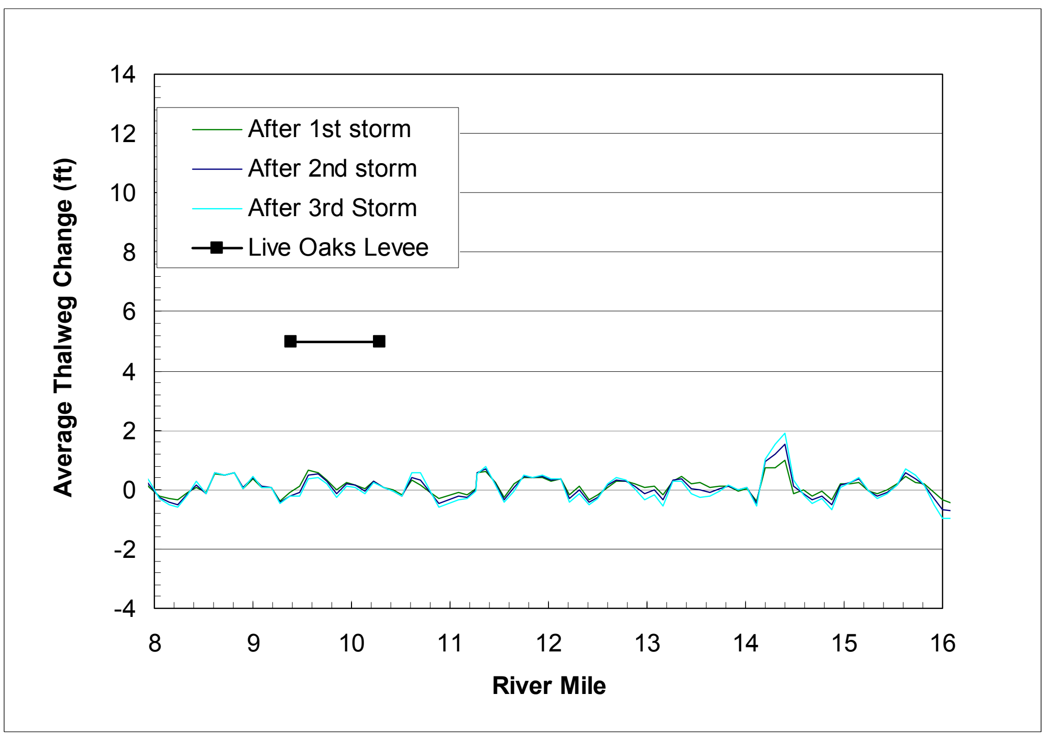


Figure 19.4. Change in thalweg elevation for 1998 flood in reach 4 at various times from start of simulation of No Action alternative.

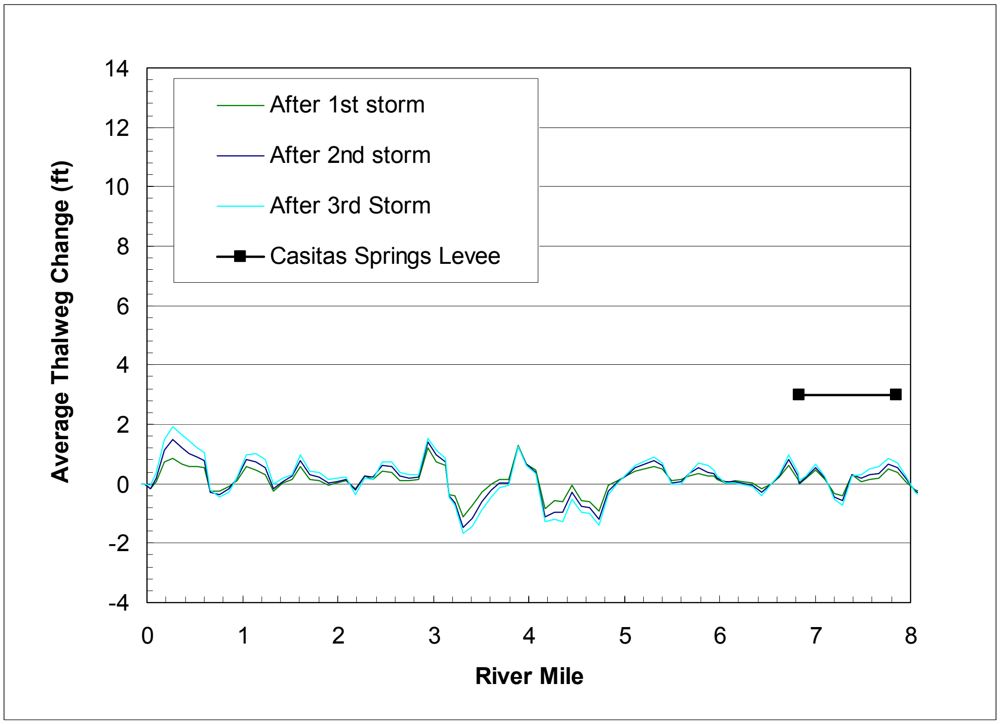


Figure 19.5. Change in thalweg elevation for 1998 flood in reach 3 at various times from start of simulation of No Action alternative.

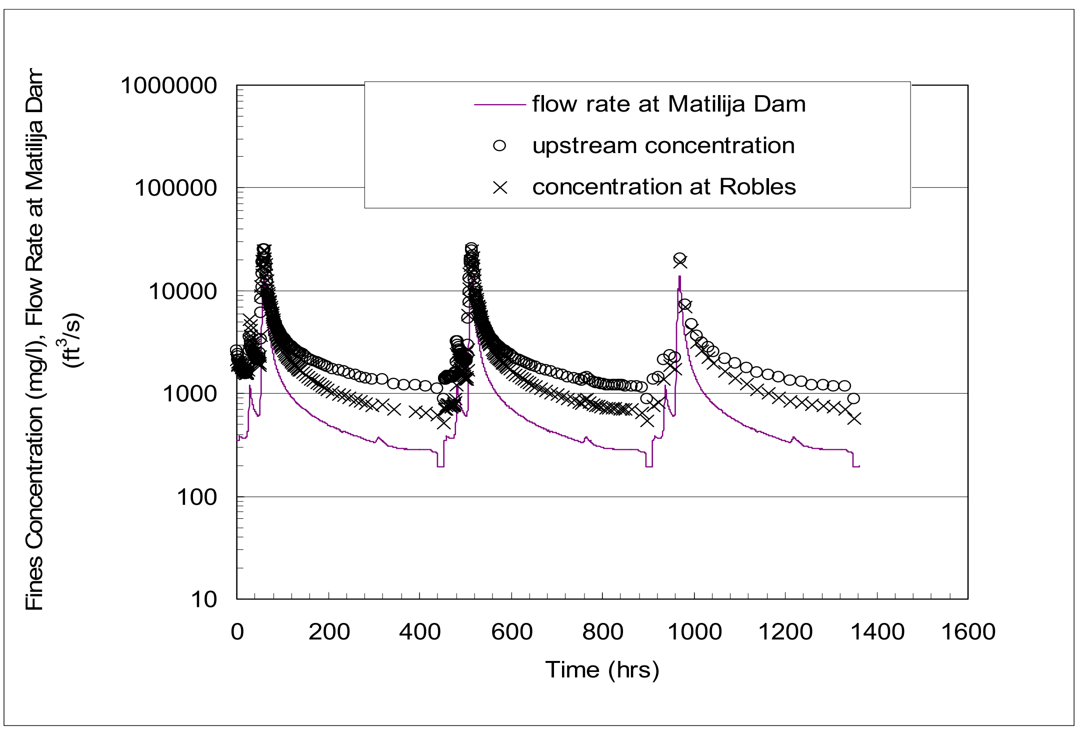


Figure 19.6. Concentrations downstream of dam following removal for 1998 flood for No Action alternative.

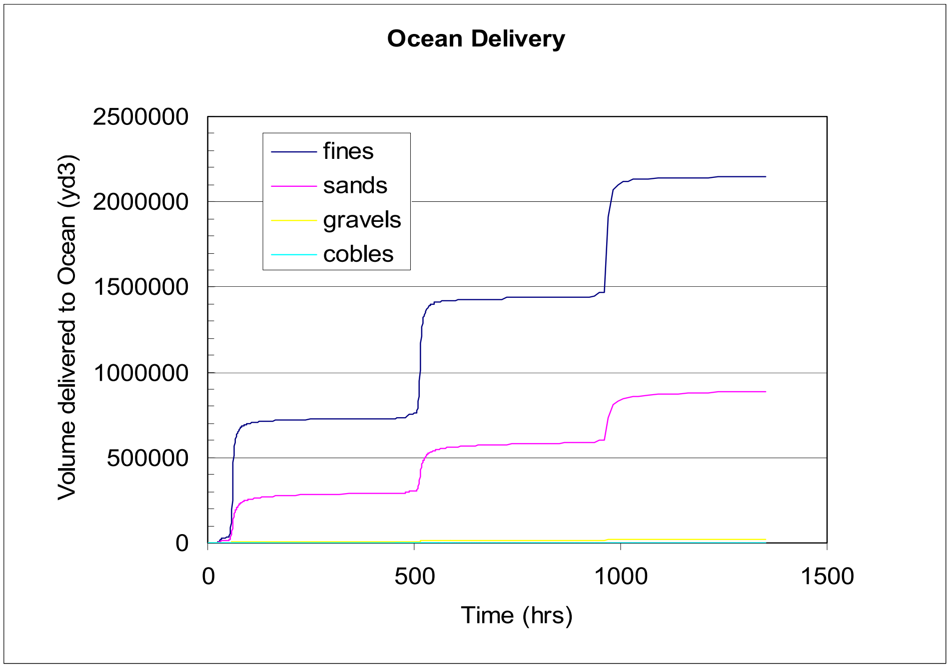


Figure 19.7. Sediment Delivery to ocean for 1998 flood for No Action alternative.

19.1.2. ALTERNATIVE 2A - FULL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH REMOVAL OF RESERVOIR AREA FINES OFFSITE

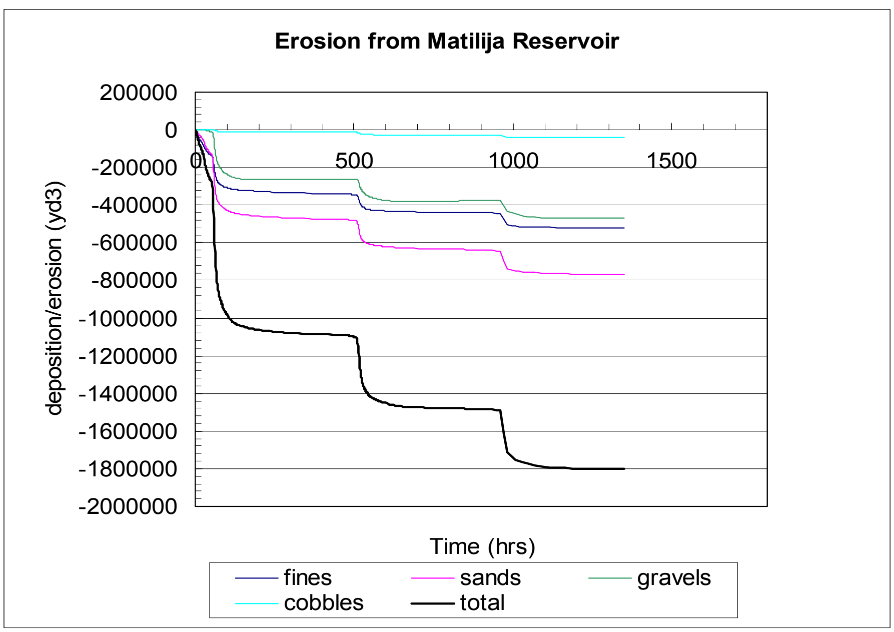


Figure 19.8. Cumulative erosion from the reservoir in alternative 2a for 1998 flood.

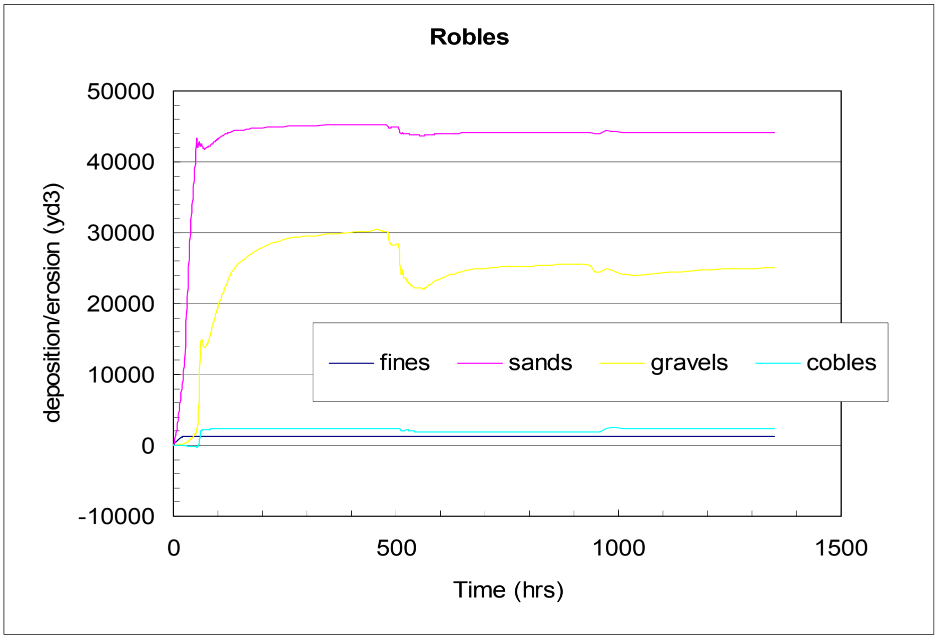


Figure 19.9. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 2a for 1998 flood. Robles Diversion Dam is at RM 14.15. With Sediment Bypass.

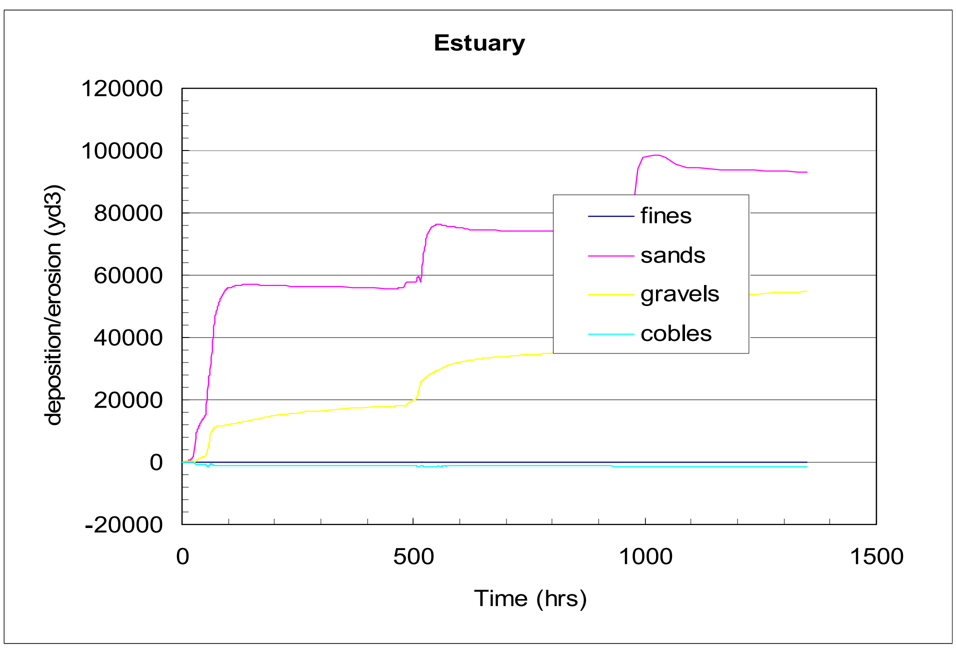


Figure 19.10. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 2a for 1998 flood.

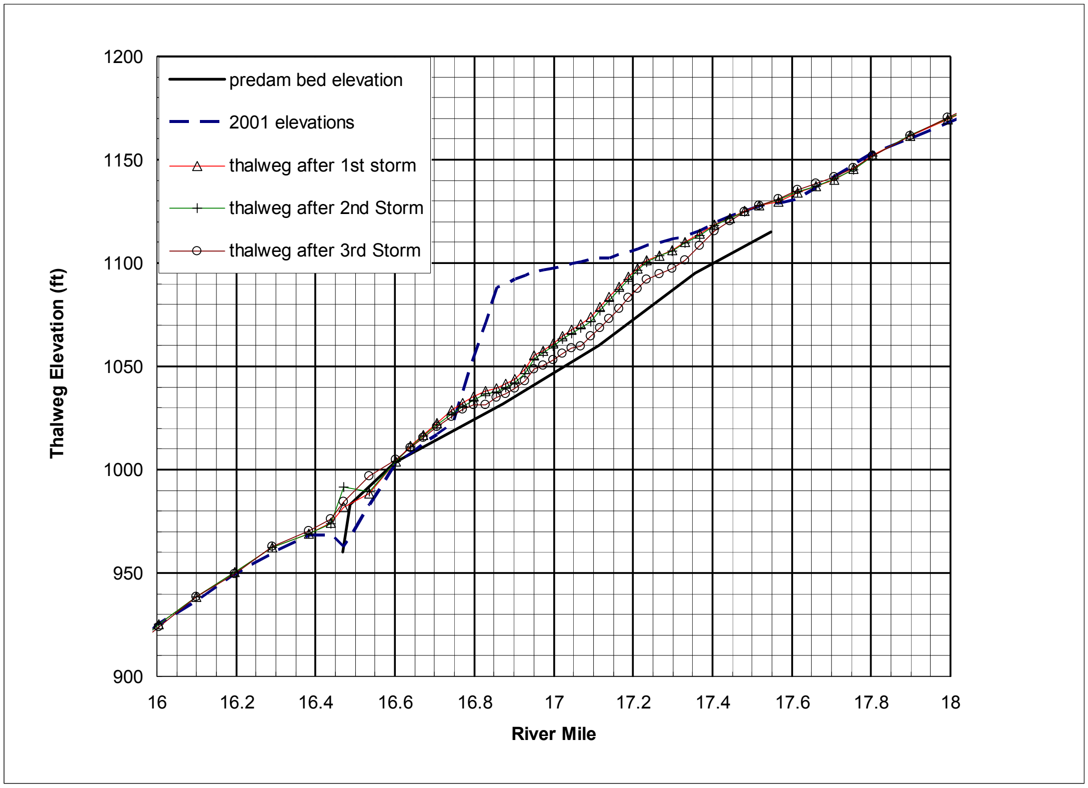


Figure 19.11. Change in thalweg elevation for 1998 flood in reach 7 of Alternative 2a.

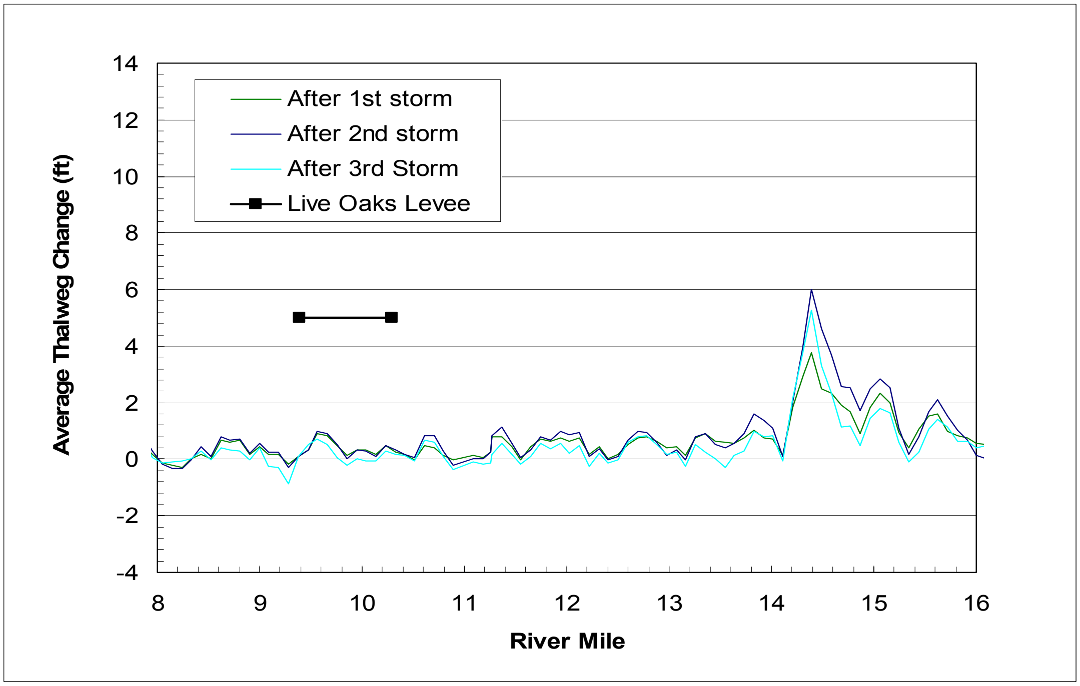


Figure 19.12. Change in thalweg elevation for 1998 flood in reach 4 at various times from start of simulation of alternative 2a.

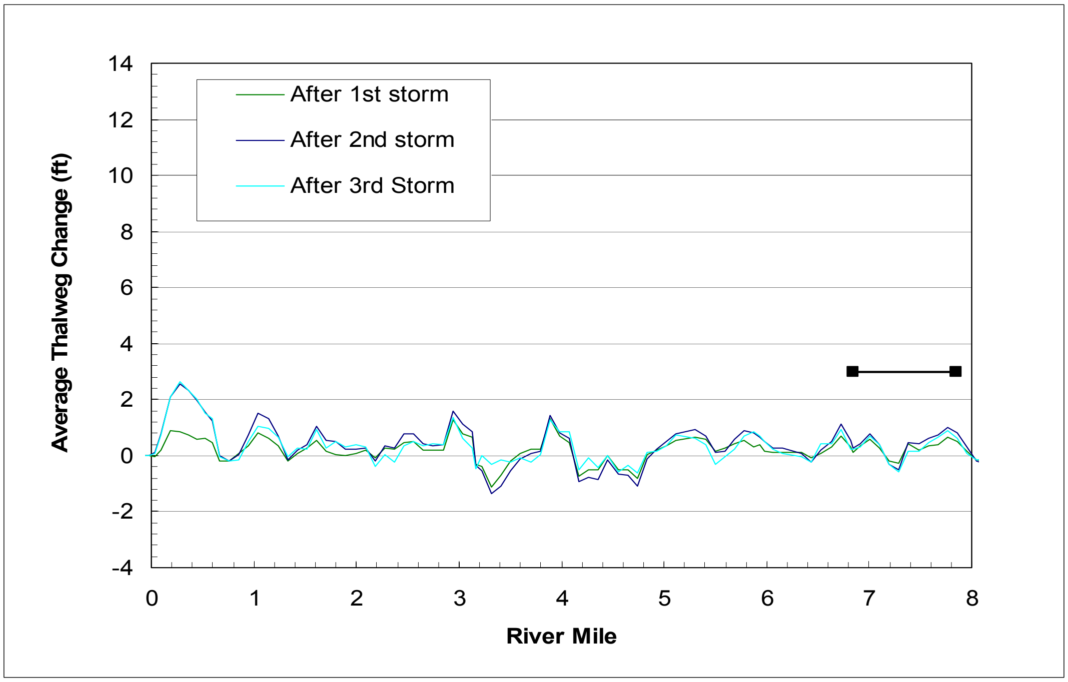


Figure 19.13. Change in thalweg elevation for 1998 flood in reach 3 at various times from start of simulation of alternative 2a.

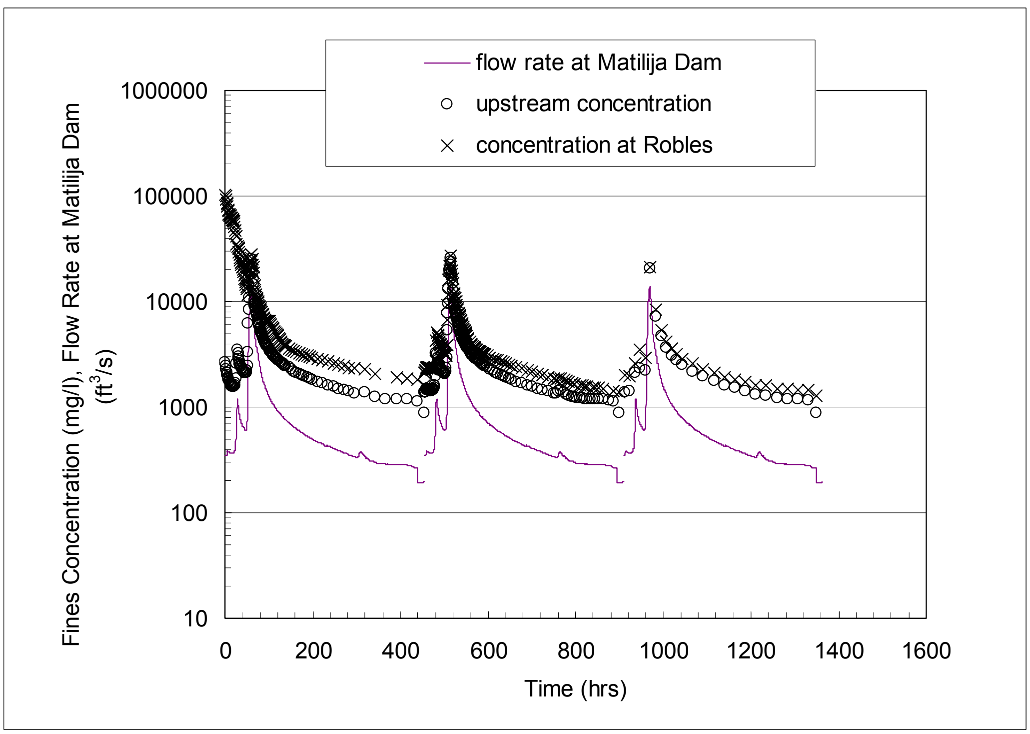


Figure 19.14. Concentrations downstream of dam following removal for 1998 flood for alternative 2a.

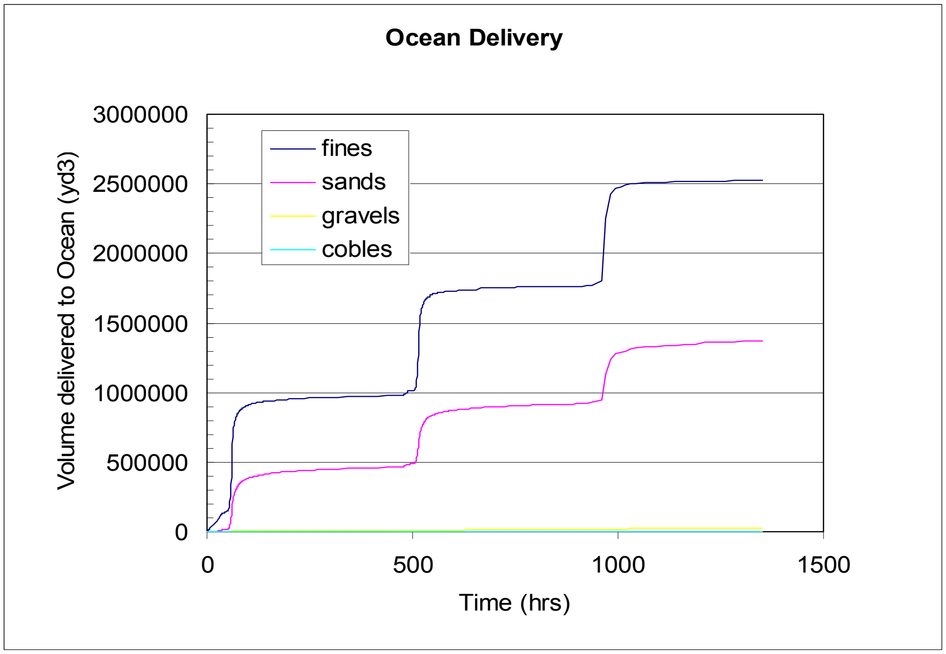


Figure 19.15. Sediment Delivery to ocean for 1998 flood for alternative 2a.

19.1.3. ALTERNATIVE 2B - FULL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH NATURAL TRANSPORT OF RESERVOIR AREA FINES

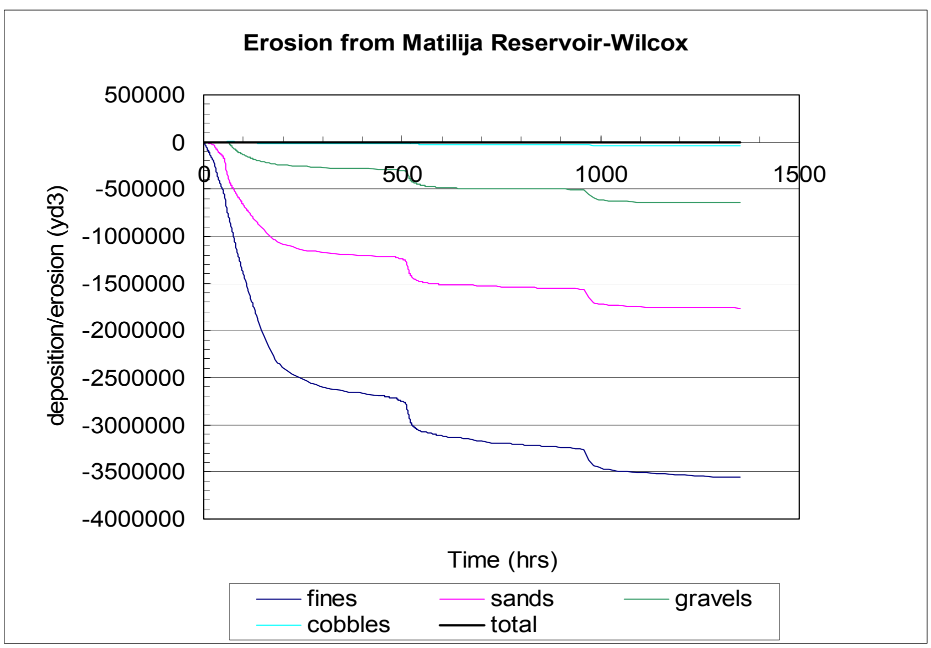


Figure 19.16. Cumulative erosion from the reservoir in alternative 2b for 1998 flood.

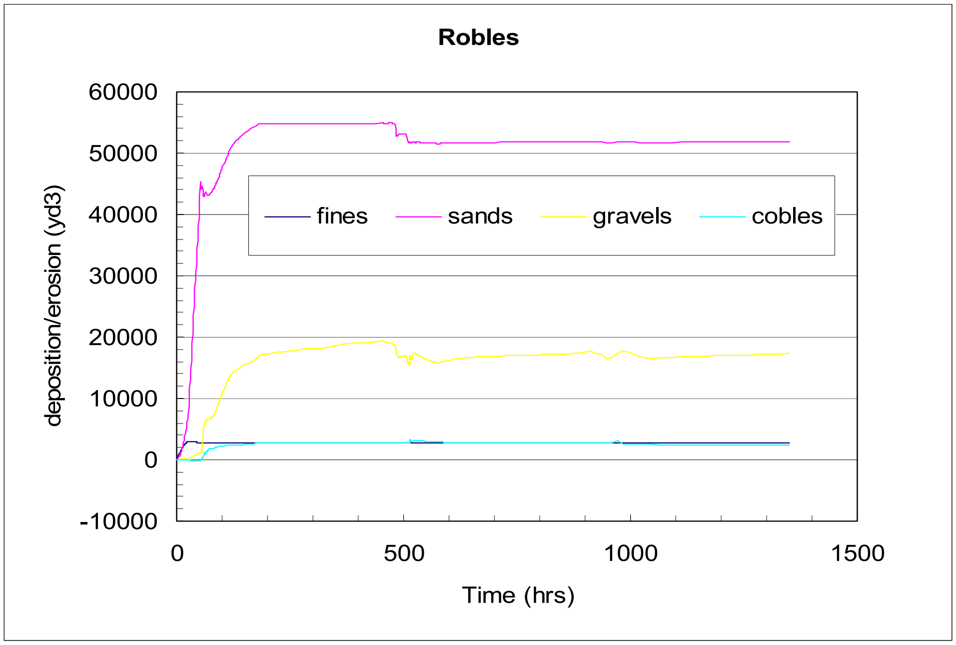


Figure 19.17. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 2b for 1998 flood. Robles Diversion Dam is at RM 14.15.

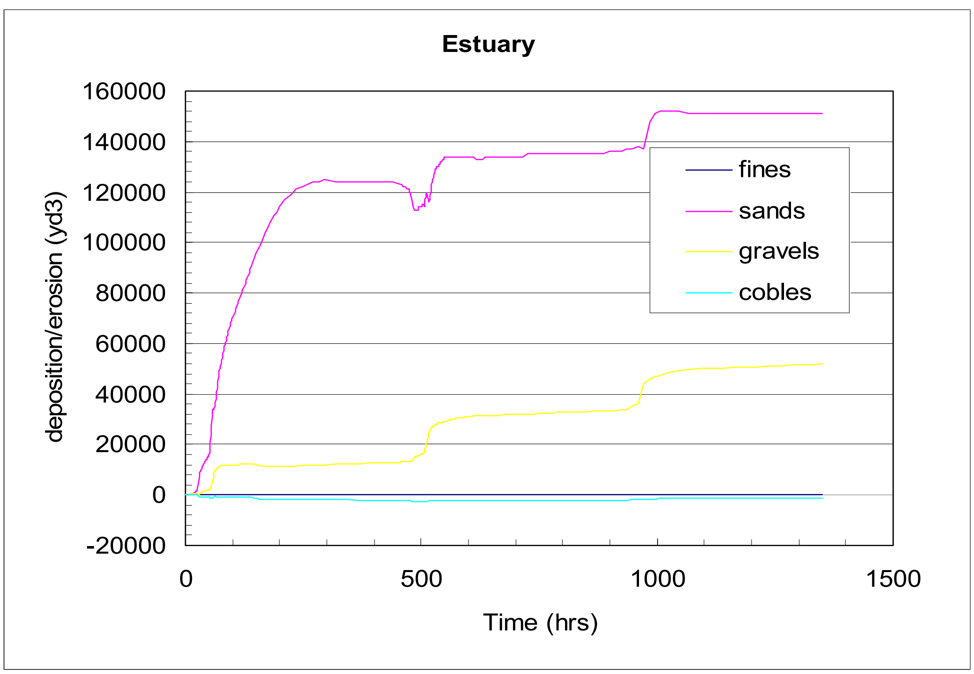


Figure 19.18. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 2b for 1998 flood.

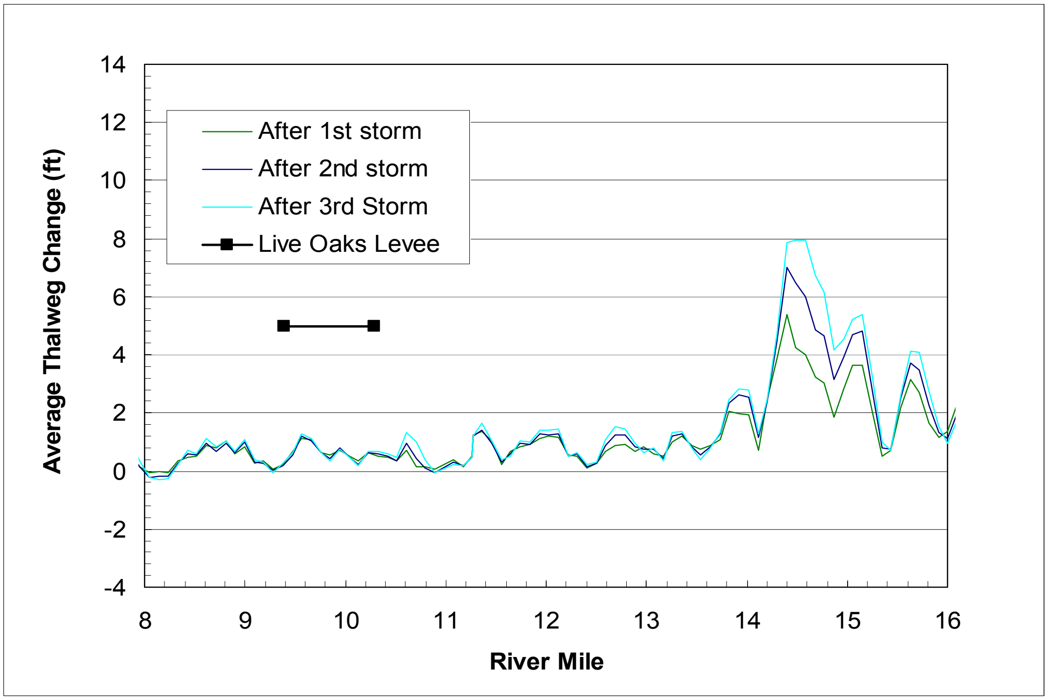


Figure 19.19. Change in thalweg elevation for 1998 flood in reach 4 at various times from start of simulation of alternative 2b.

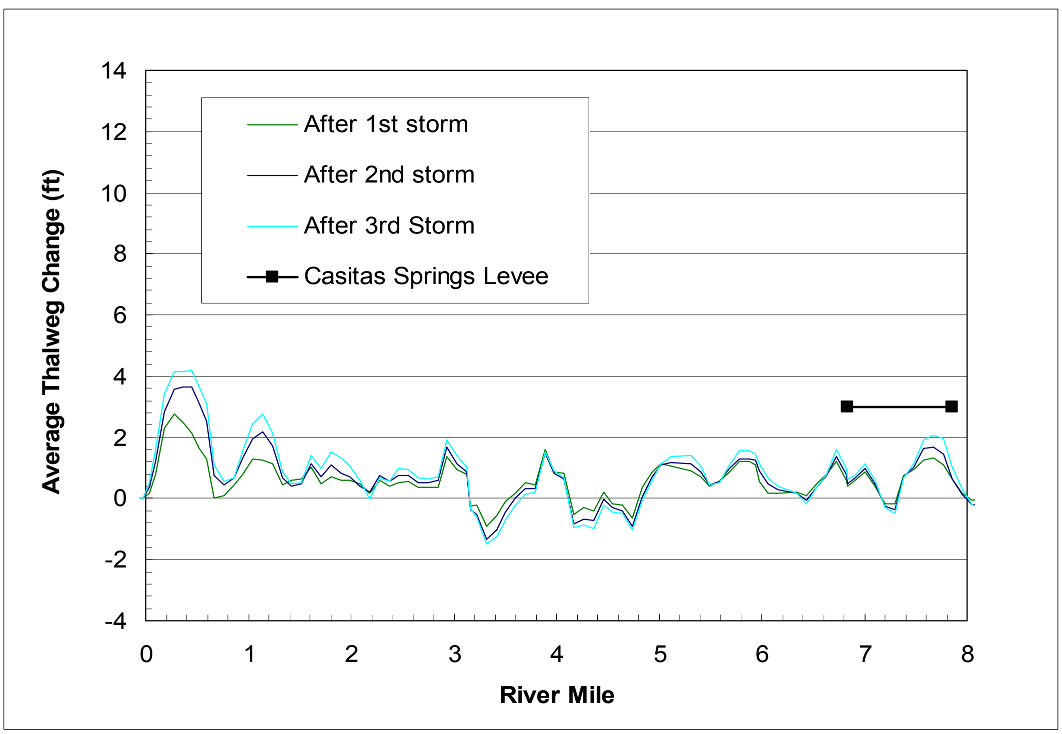


Figure 19.20. Change in thalweg elevation for 1998 flood in reach 3 at various times from start of simulation of alternative 2b.

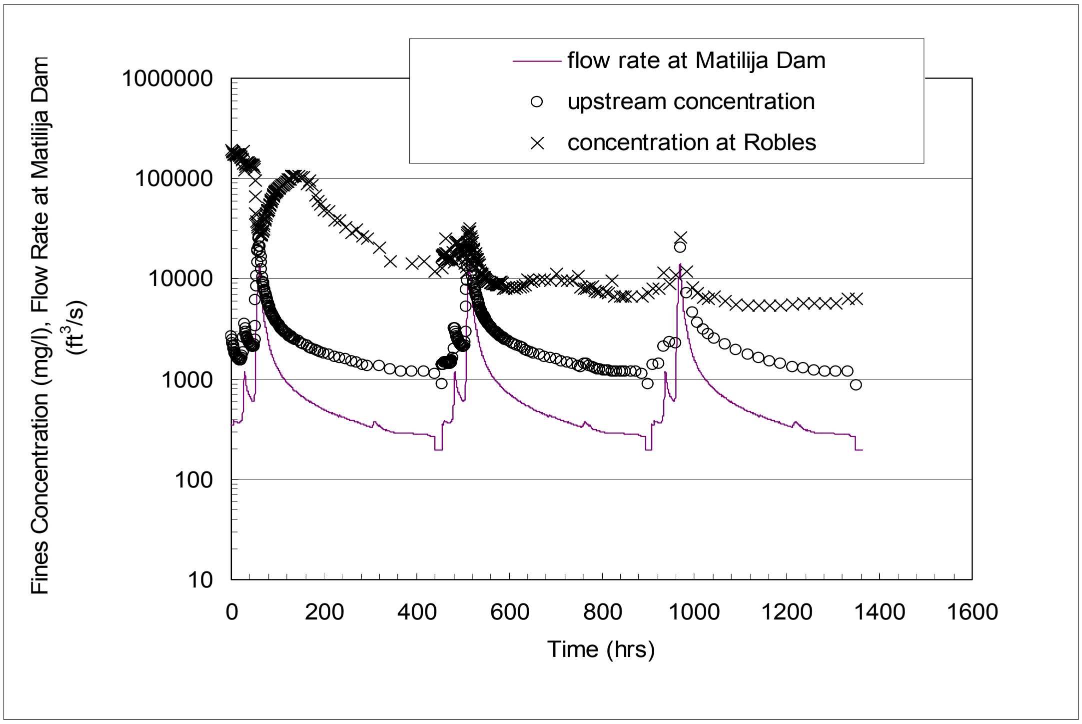


Figure 19.21. Concentrations downstream of dam following removal for 1998 flood for alternative 2b.

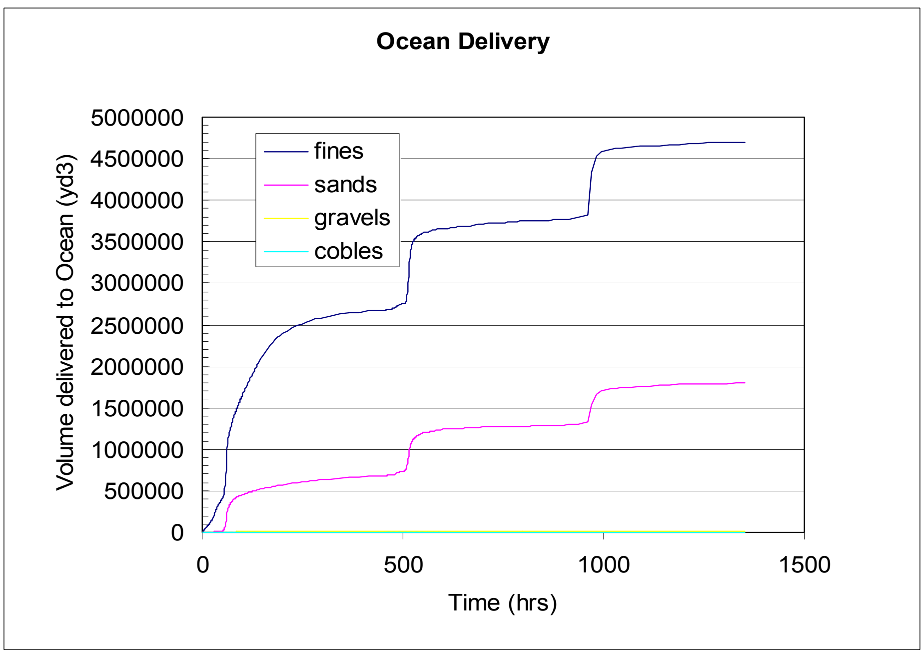


Figure 19.22. Sediment Delivery to ocean for 1998 flood for alternative 2b.

19.1.4. ALTERNATIVE 3A – INCREMENTAL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH REMOVAL OF RESERVOIR AREA FINES OFFSITE

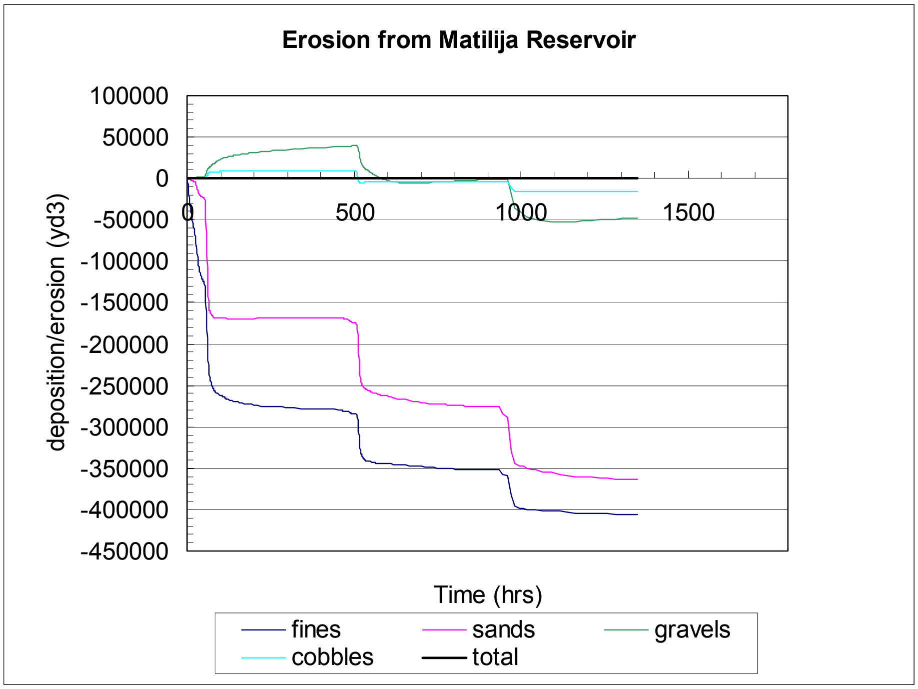


Figure 19.23. Cumulative erosion from the reservoir in alternative 3a for 1998 flood.

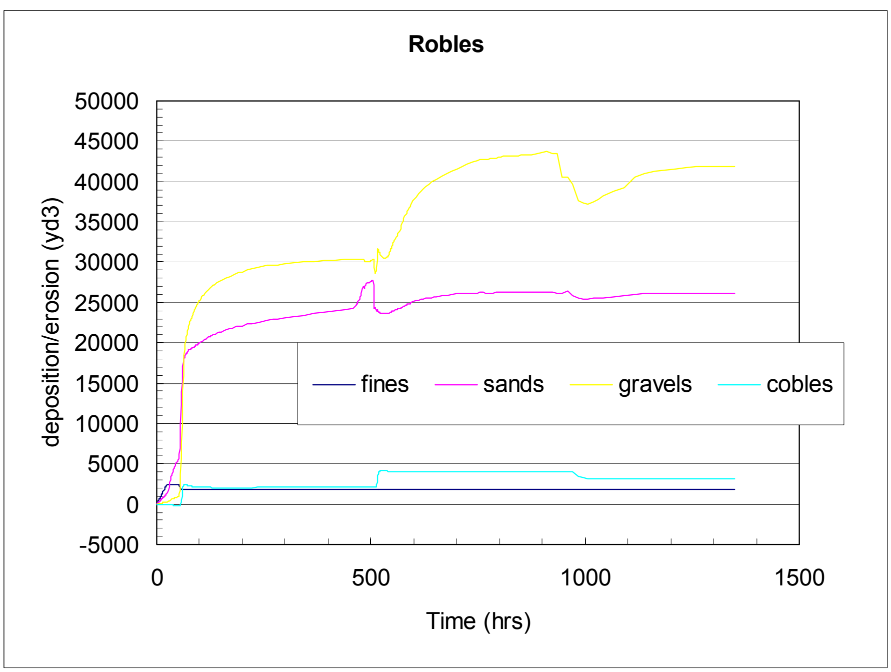


Figure 19.24. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 3a for 1998 flood. Robles Diversion Dam is at RM 14.15.

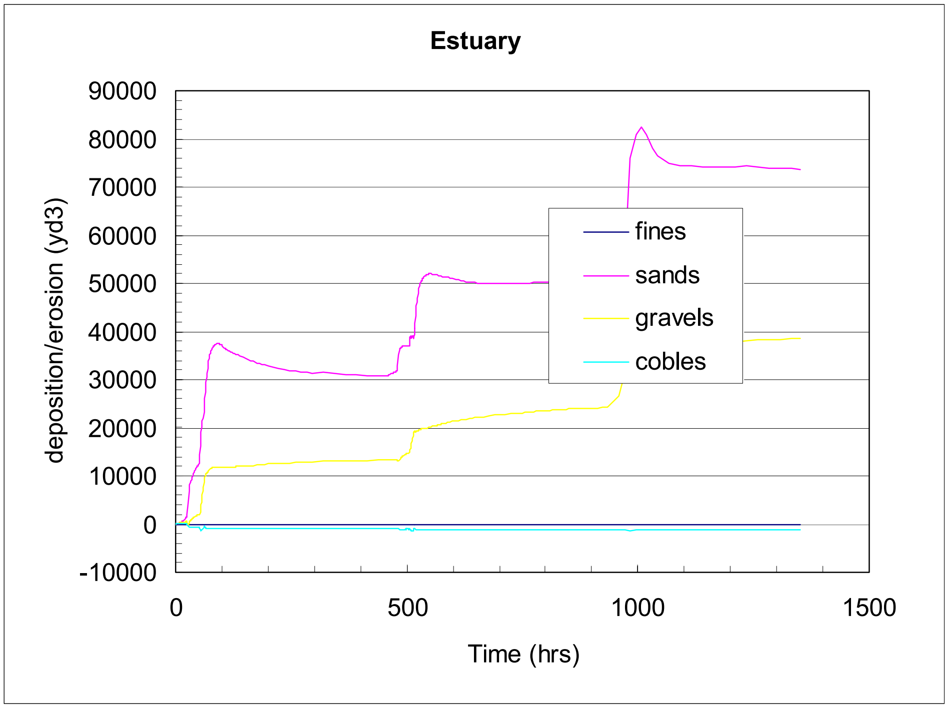


Figure 19.25. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 3a for 1998 flood.

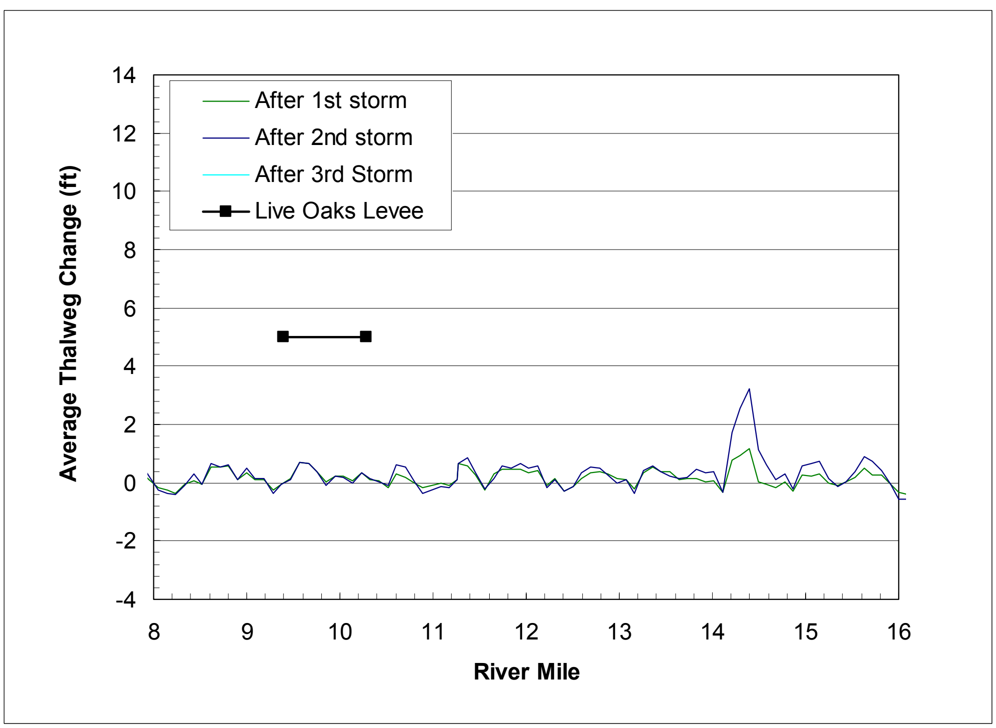


Figure 19.26. Change in thalweg elevation for 1998 flood in reach 4 at various times from start of simulation of alternative 3a.

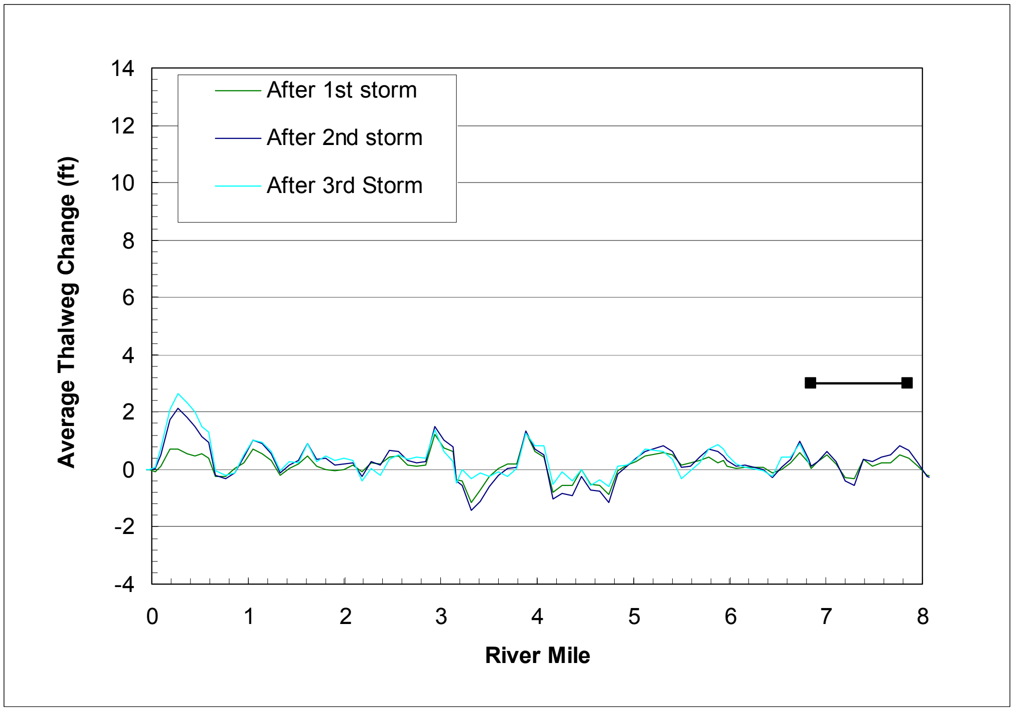


Figure 19.27. Change in thalweg elevation for 1998 flood in reach 3 at various times from start of simulation of alternative 3a.

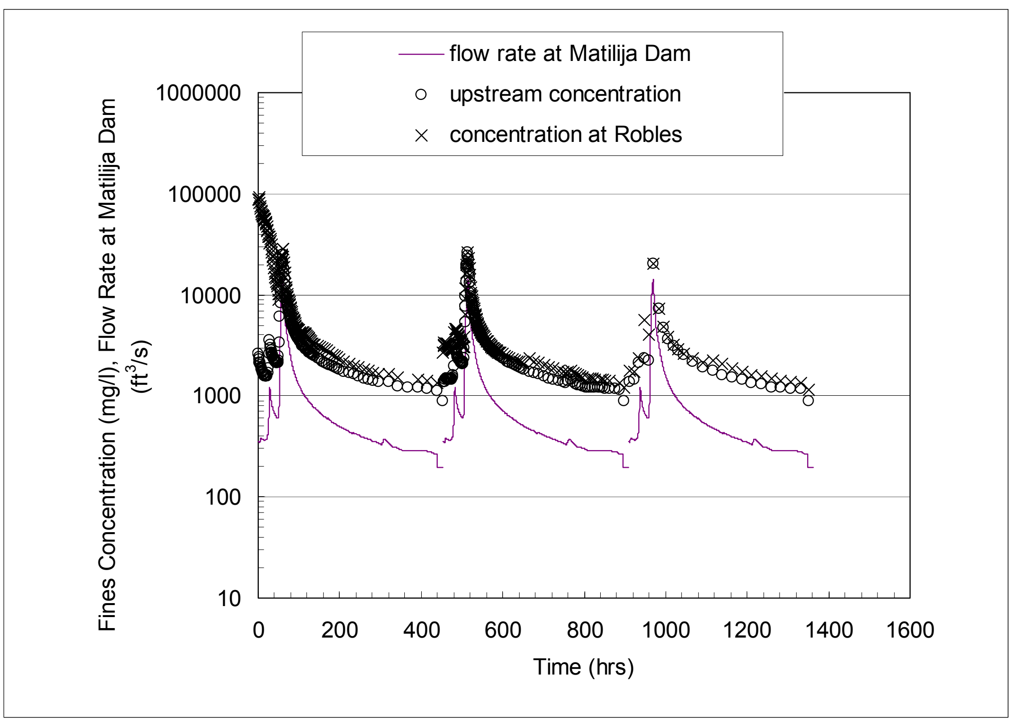


Figure 19.28. Concentrations downstream of dam following removal for 1998 flood for alternative 3a.

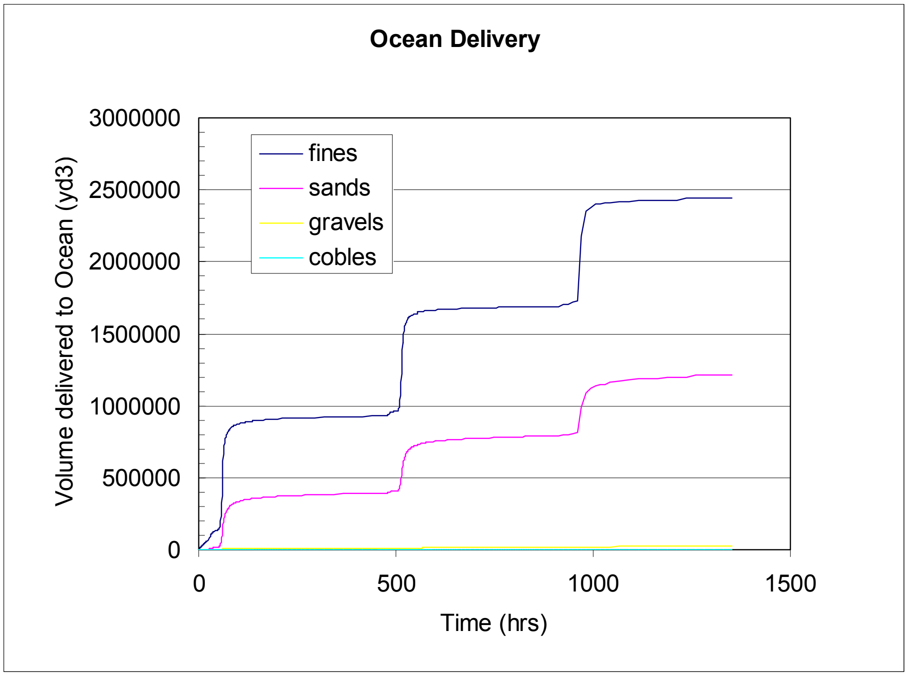


Figure 19.29. Sediment Delivery to ocean for 1998 flood for alternative 3a.

19.1.5. ALTERNATIVE 3B – INCREMENTAL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH NO REMOVAL OF RESERVOIR AREA FINES OFFSITE

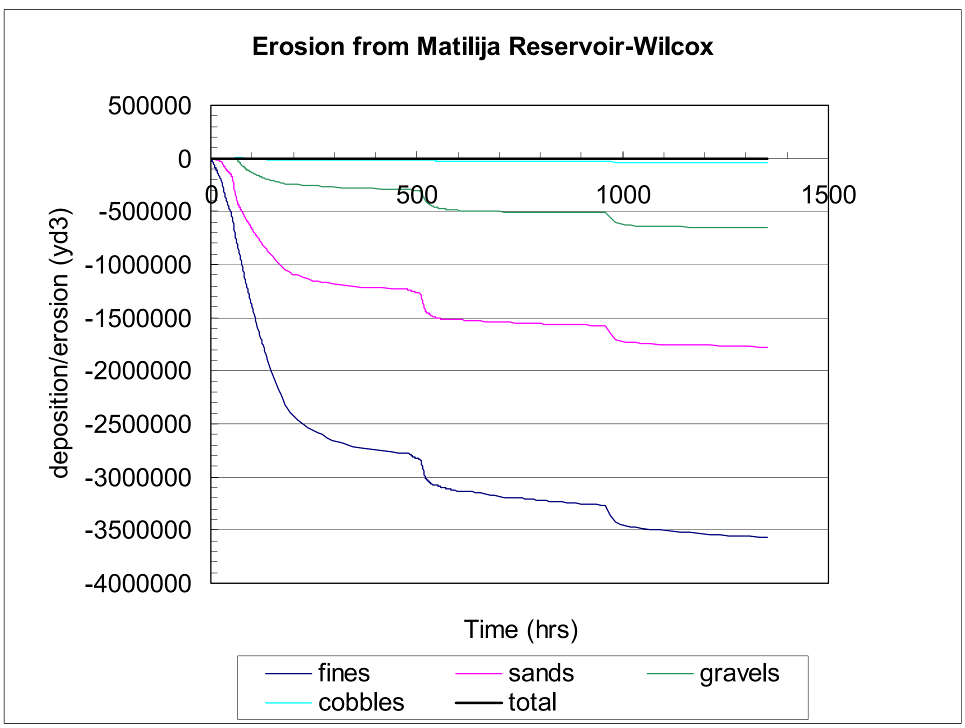


Figure 19.30. Cumulative erosion from the reservoir in alternative 3b for 1998 flood.

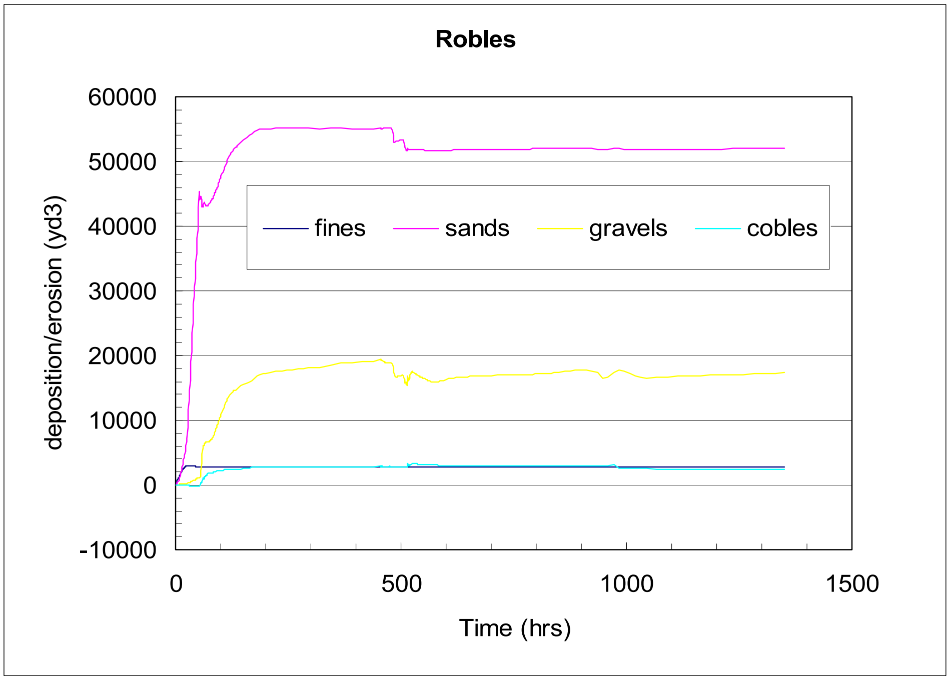


Figure 19.31. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 3b for 1998 flood. Robles Diversion Dam is at RM 14.15.

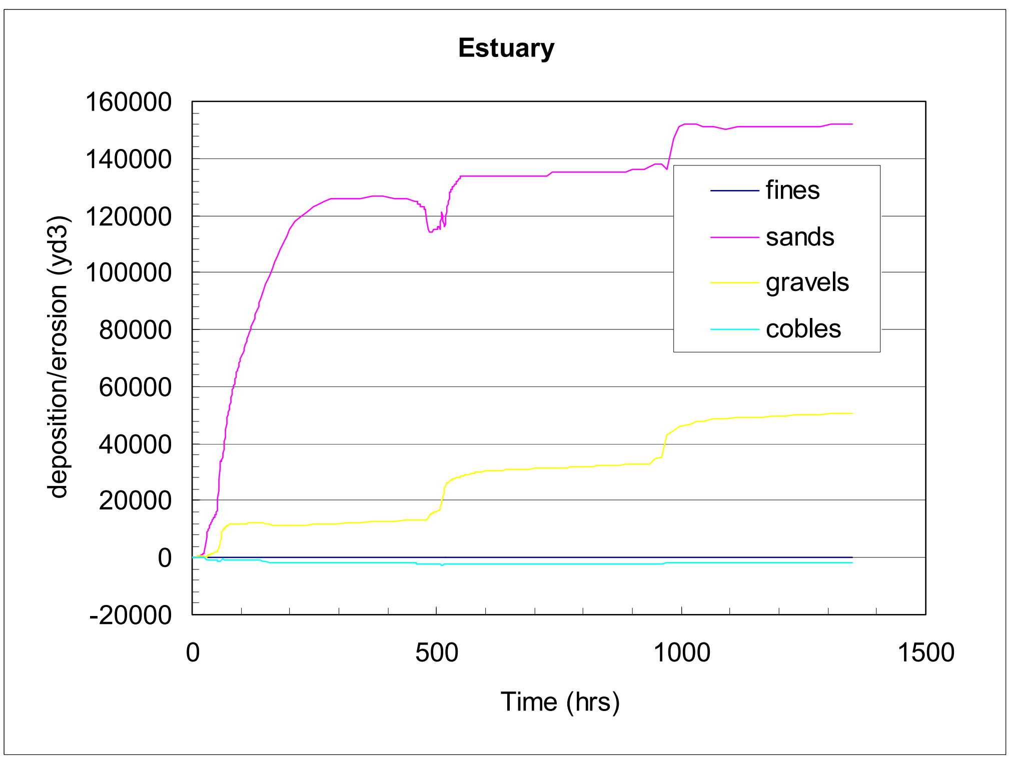


Figure 19.32. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 3b for 1998 flood.

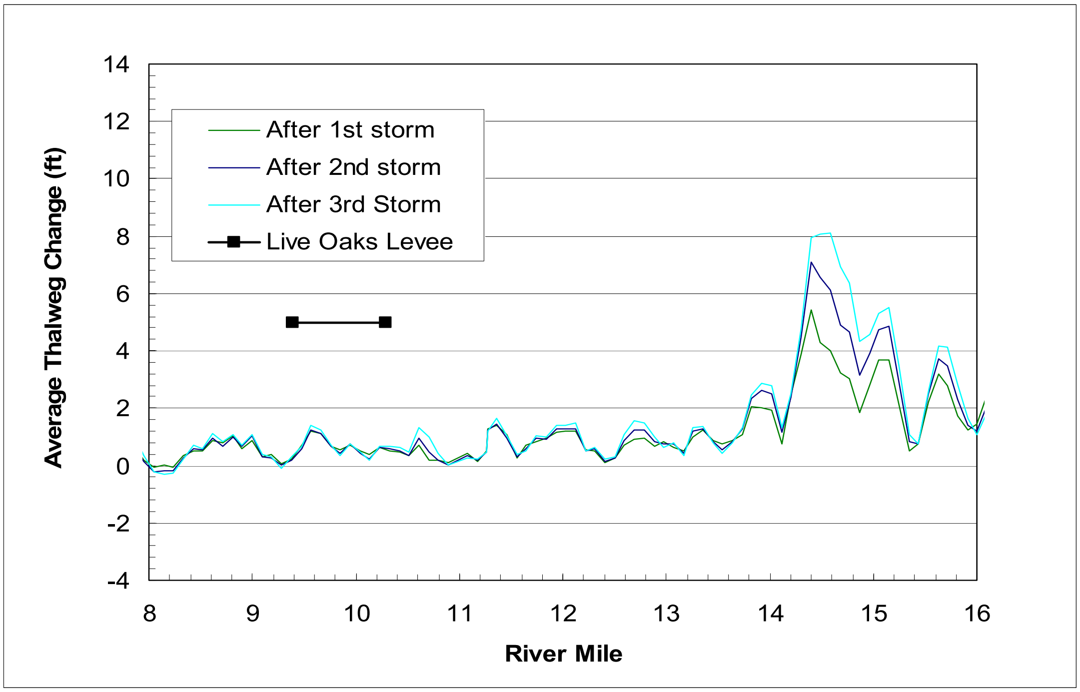


Figure 19.33. Change in thalweg elevation for 1998 flood in reach 4 at various times from start of simulation of alternative 3b.

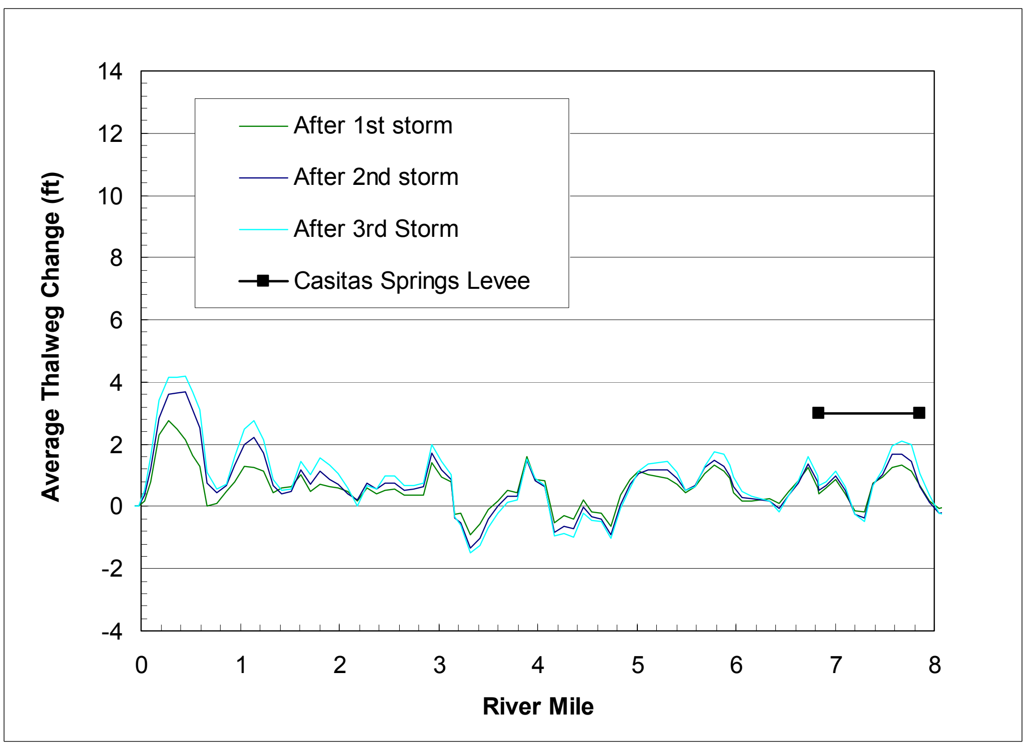


Figure 19.34. Change in thalweg elevation for 1998 flood in reach 3 at various times from start of simulation of alternative 3b.

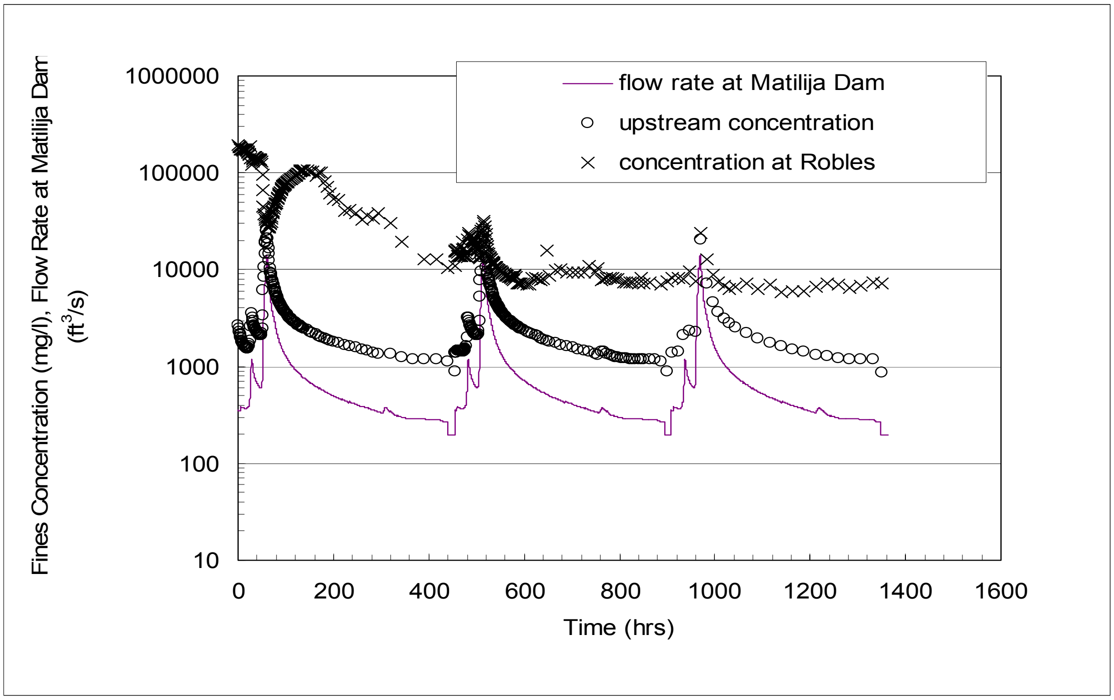


Figure 19.35. Concentrations downstream of dam following removal for 1998 flood for alternative 3b.

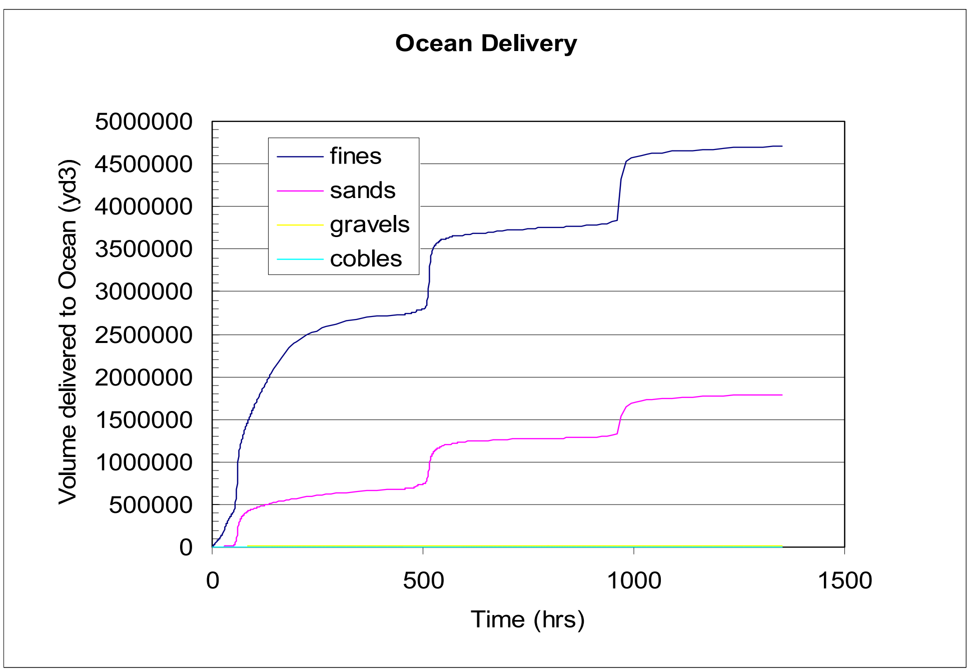


Figure 19.36. Sediment Delivery to ocean for 1998 flood for alternative 3b.

19.1.6. ALTERNATIVE 4B – FULL DAM REMOVAL/TEMPORARILY STABILIZE SEDIMENTS ON SITE

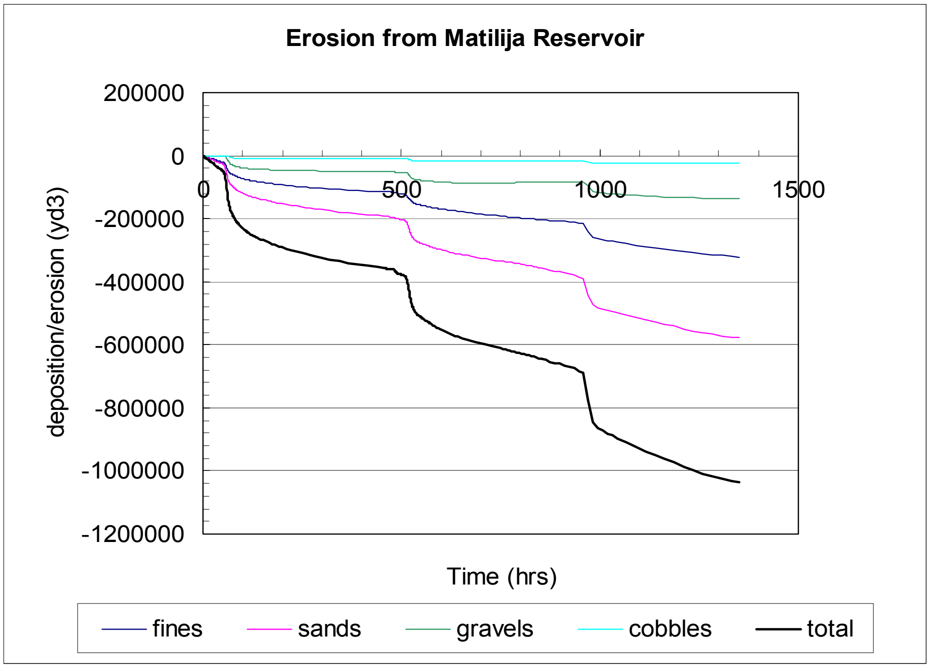


Figure 19.37. Cumulative erosion from the reservoir in alternative 4b for 1998 flood.

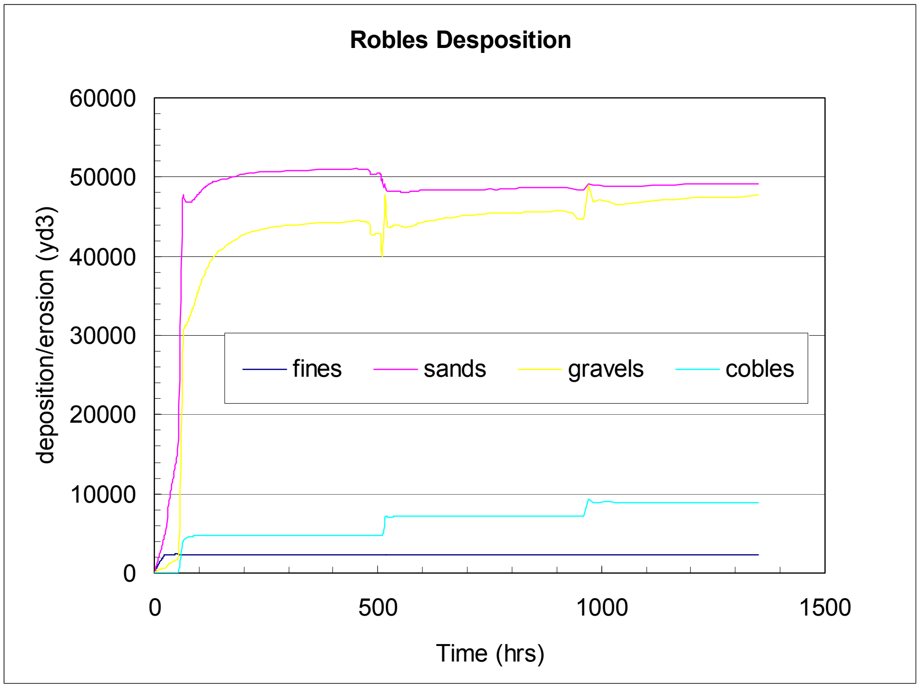


Figure 19.38. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 4b for 1998 flood. Robles Diversion Dam is at RM 14.15.

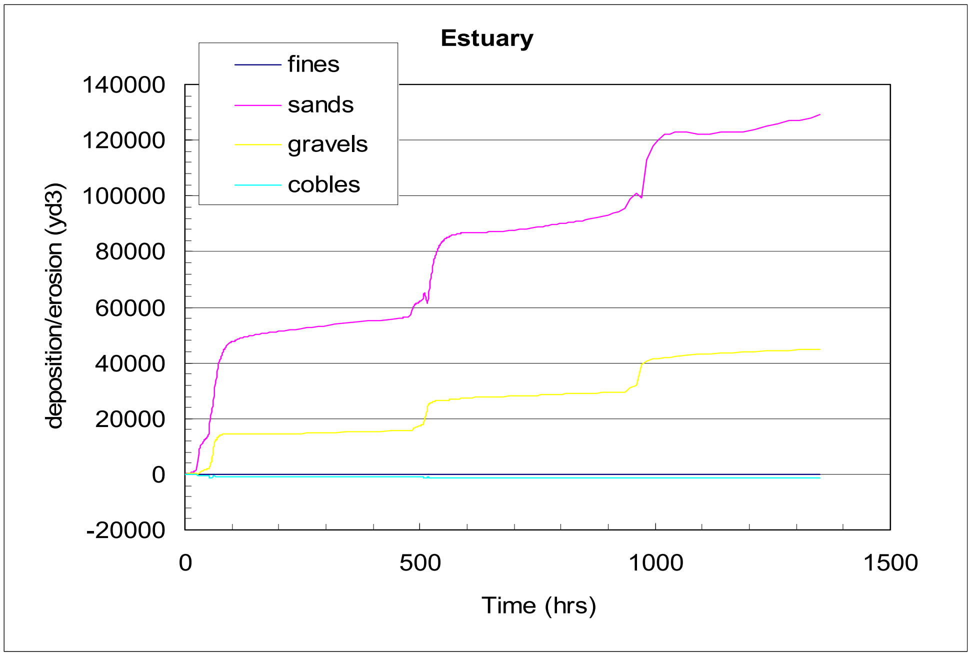


Figure 19.39. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 4b for 1998 flood.

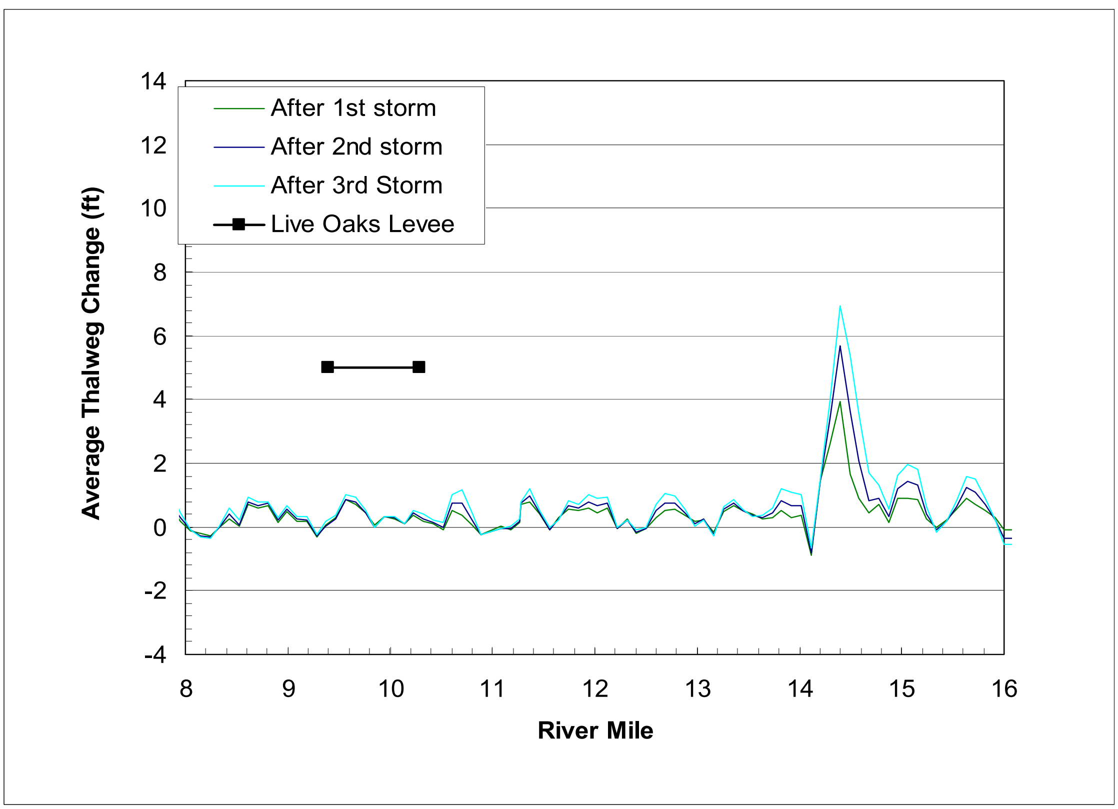


Figure 19.40. Change in thalweg elevation for 1998 flood in reach 4 at various times from start of simulation of alternative 4b.

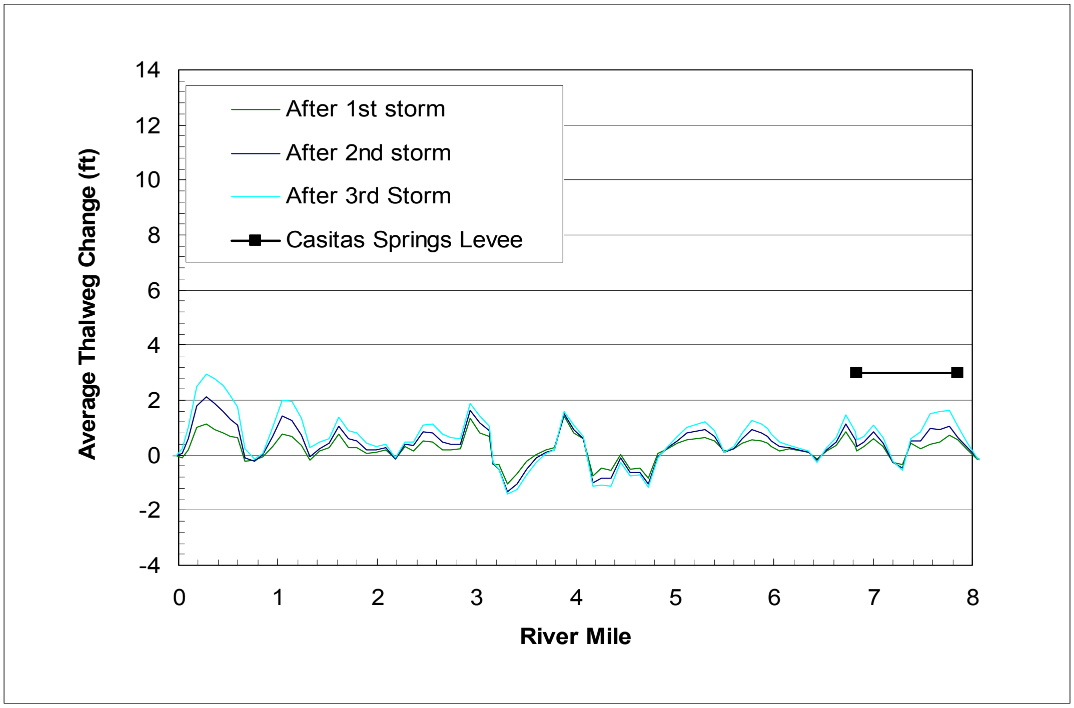


Figure 19.41. Change in thalweg elevation for 1998 flood in reach 3 at various times from start of simulation of alternative 4b.

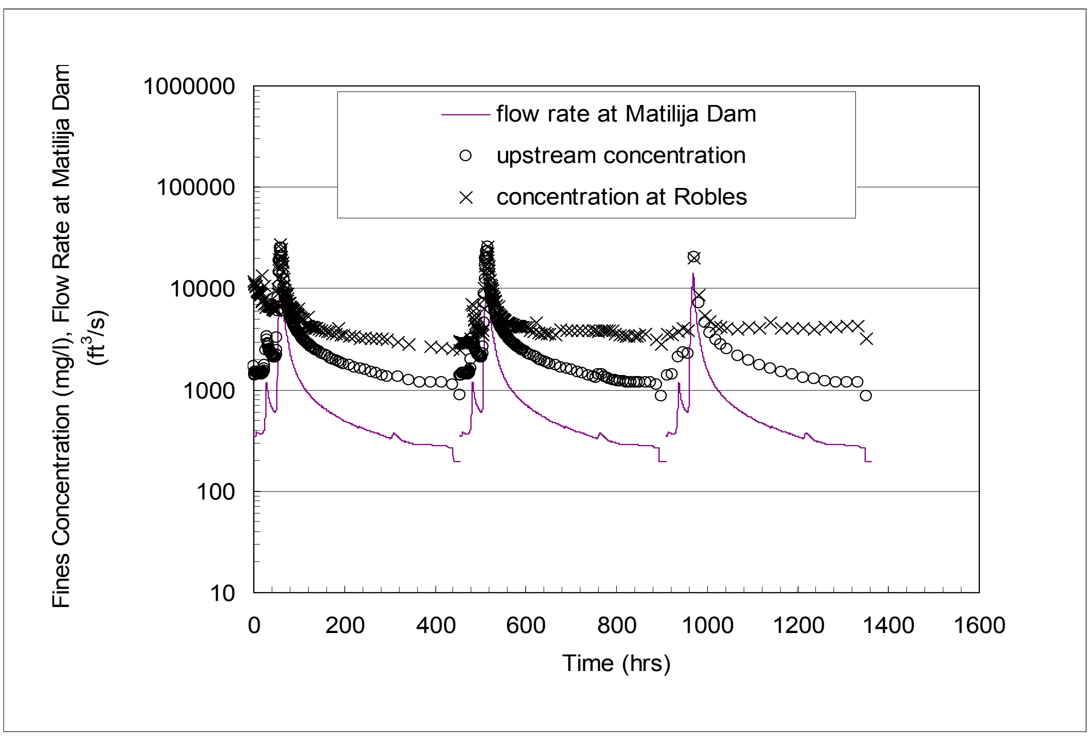


Figure 19.42. Concentrations downstream of dam following removal for 1998 flood for alternative 4b.

Figure 19.43. Sediment Delivery to ocean for 1998 flood for alternative 4b.

19.1.7. Alternatives 1 and 4a – Complete Removal of Dam and Reservoir Sediments from River System

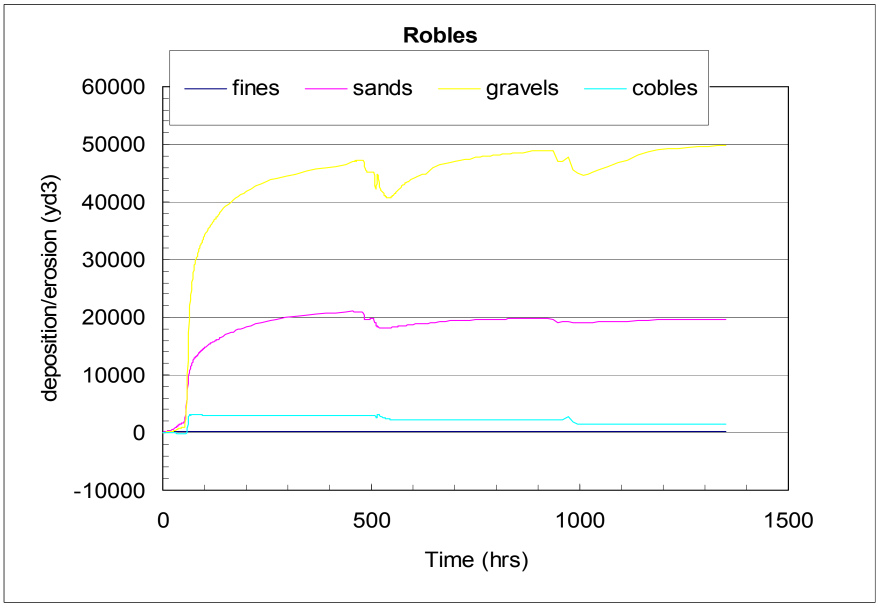


Figure 19.44. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 1 and 4a for 1998 flood. Robles Diversion Dam is at RM 14.15.

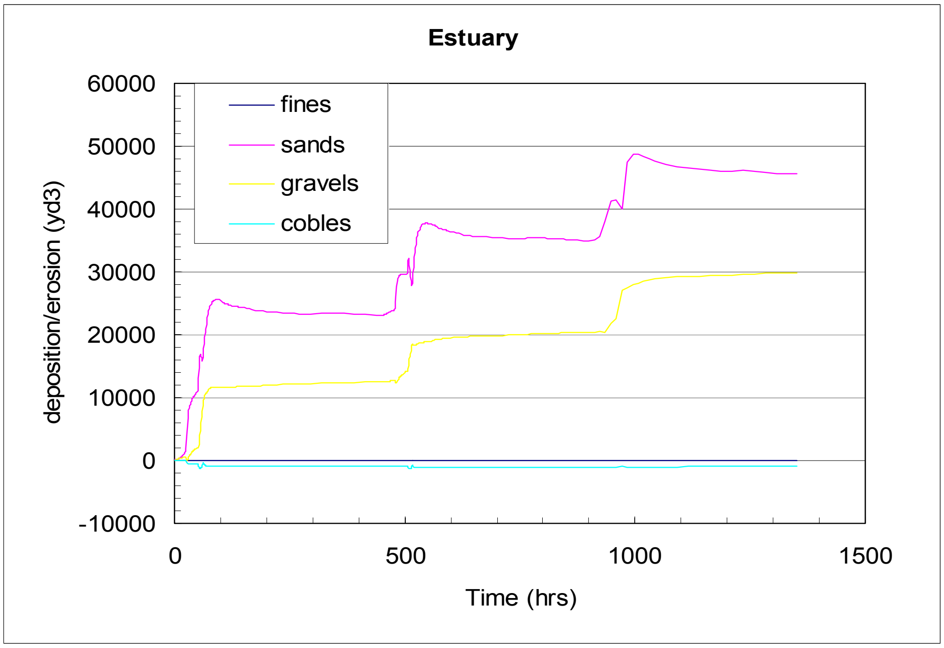


Figure 19.45. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 1 and 4a for 1998 flood.

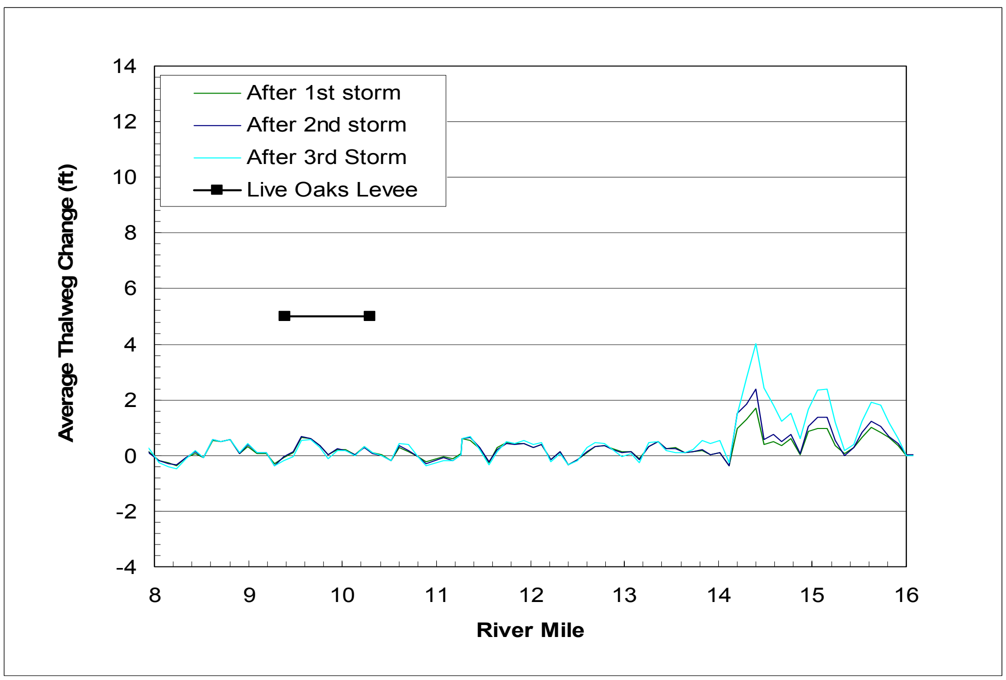


Figure 19.46. Change in thalweg elevation for 1998 flood in reach 4 at various times from start of simulation of alternative 1 and 4a.

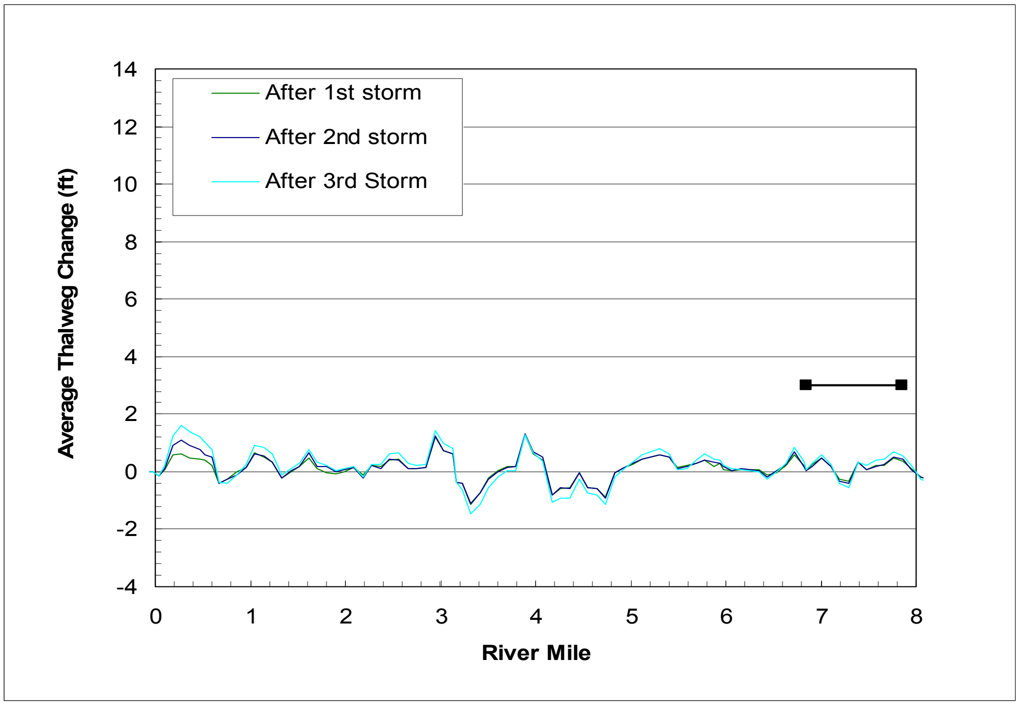


Figure 19.47. Change in thalweg elevation for 1998 flood in reach 3 at various times from start of simulation of alternative 1 and 4a.

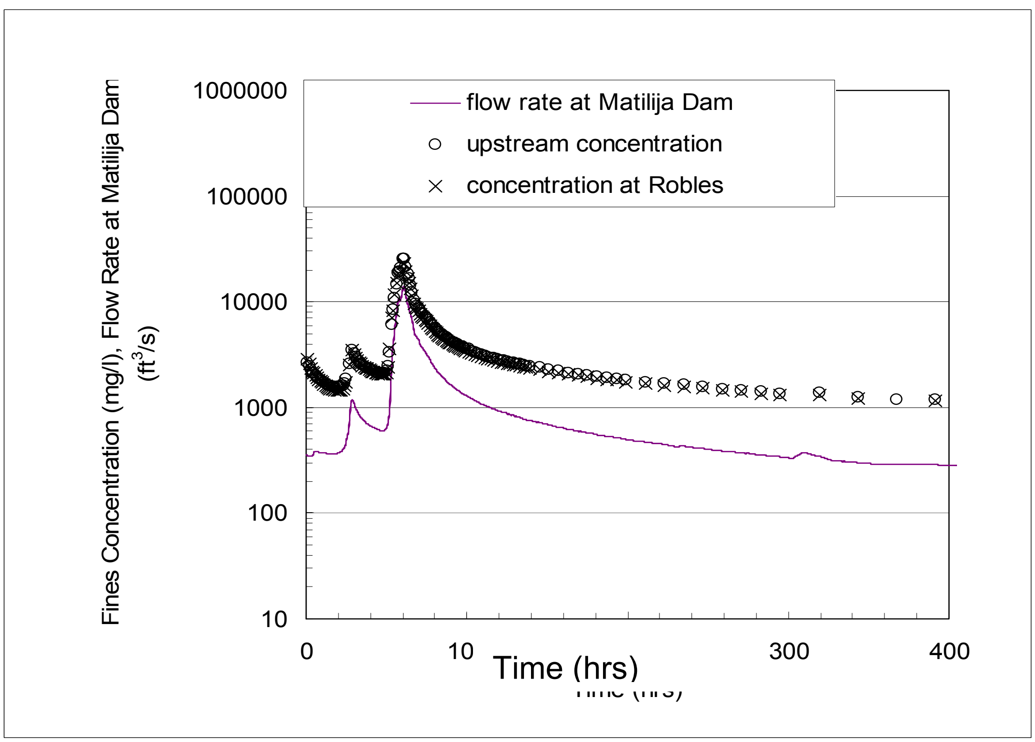


Figure 19.48. Concentrations following removal for 1998 flood for alternative 1 and 4a.

Figure 19.49. Sediment Delivery to ocean for 1998 flood for alternative 1 and 4a.

19.2. Model Results for 1991 Flood

19.2.1. No Action Alternative

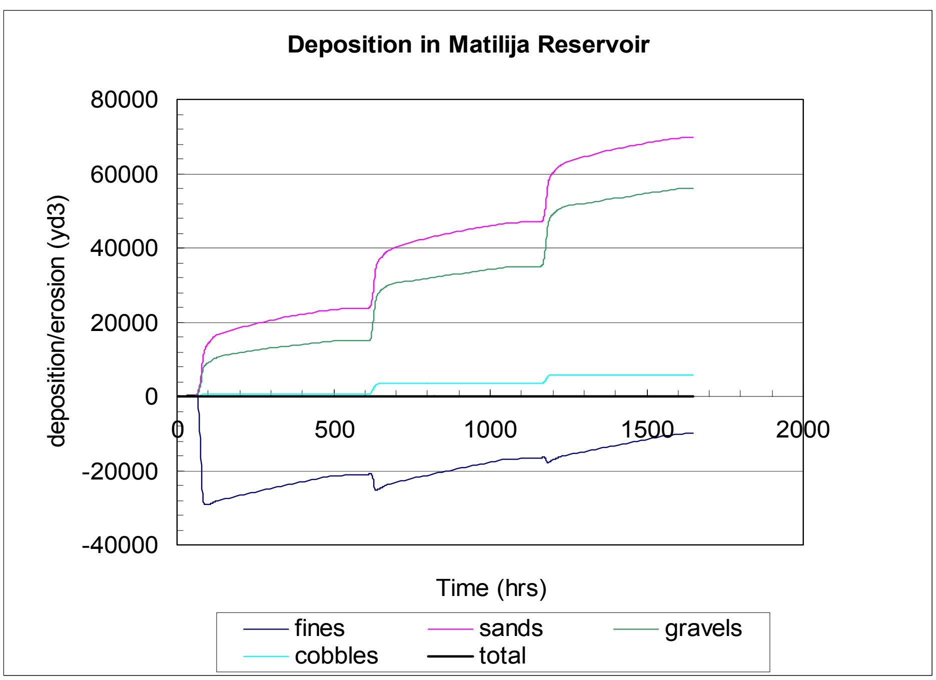


Figure 19.50. Cumulative erosion from the reservoir in No Action alternative for 1991 flood.

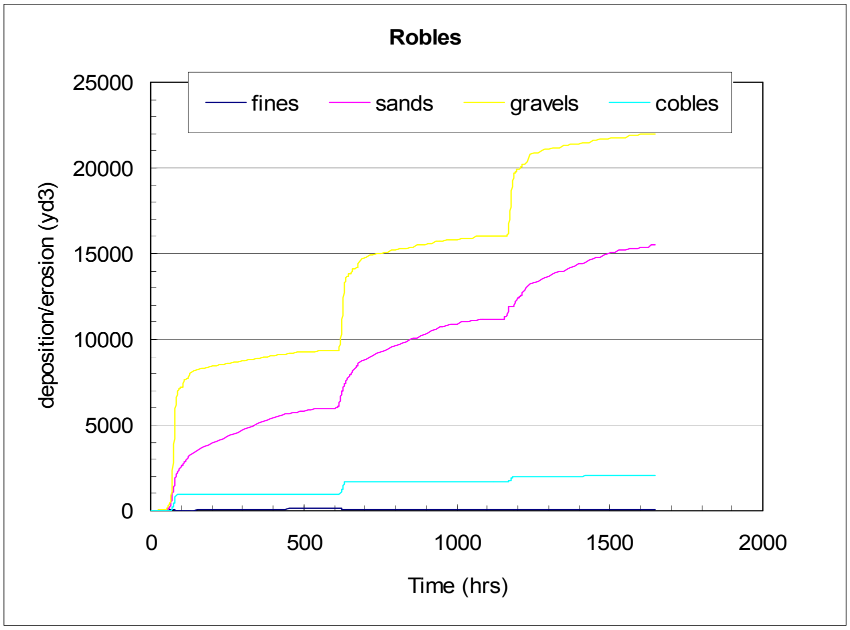


Figure 19.51. Cumulative deposition from RM 14.15 to RM 14.5 in No Action alternative for 1991 flood. Robles Diversion Dam is at RM 14.15.

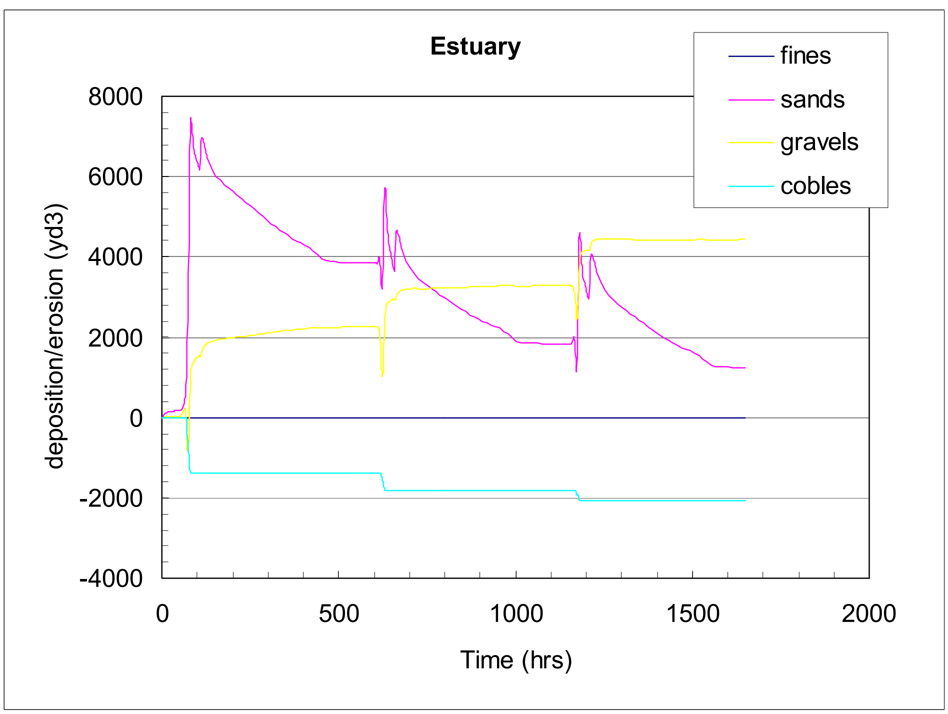


Figure 19.52. Cumulative deposition in Estuary (RM 0.6 - 0.2) in No Action alternative for 1991 flood.

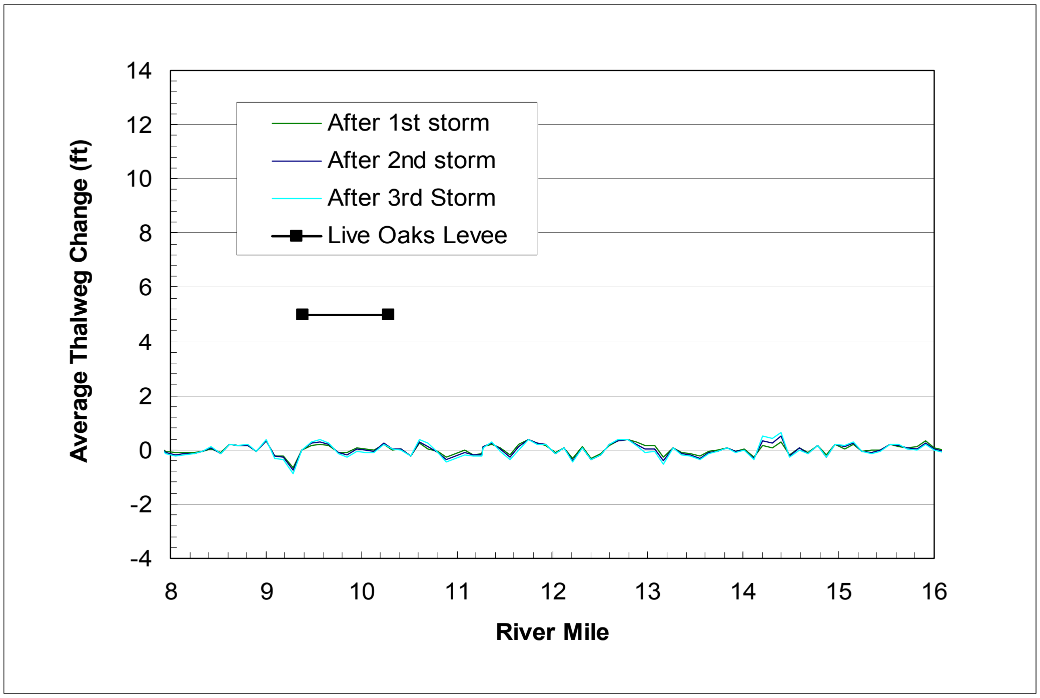


Figure 19.53. Change in thalweg elevation for 1991 flood in reach 4 at various times from start of simulation of No Action alternative.

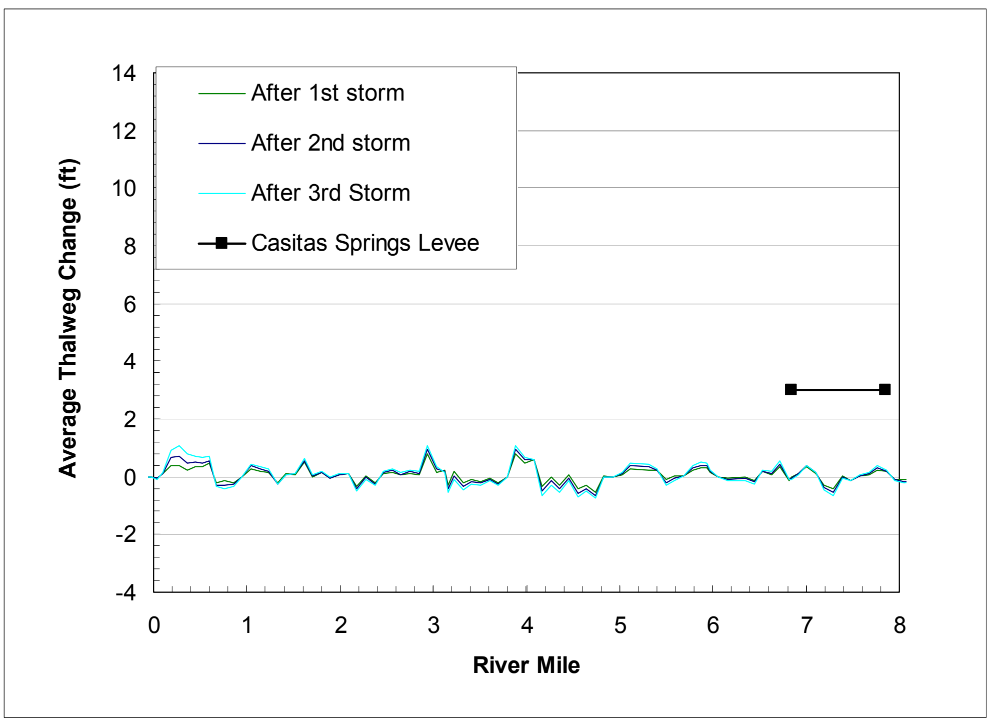


Figure 19.54. Change in thalweg elevation for 1991 flood in reach 3 at various times from start of simulation of No Action alternative.

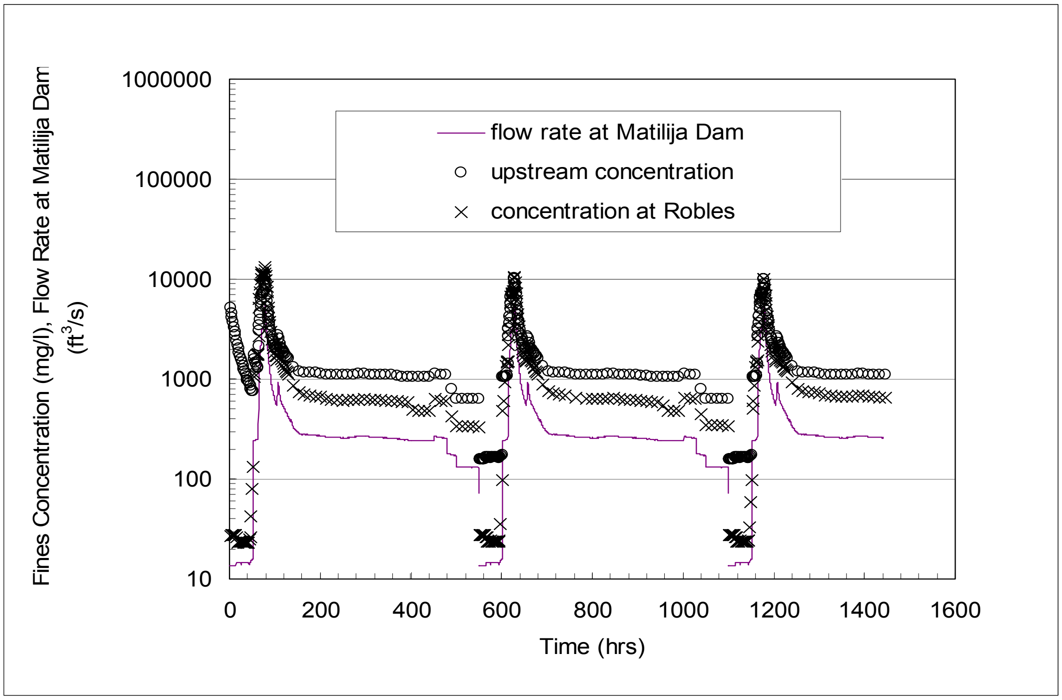


Figure 19.55. Concentrations downstream of dam following removal for 1991 flood for No Action alternative.

19.2.2. ALTERNATIVE 2A - FULL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH REMOVAL OF RESERVOIR AREA FINES OFFSITE

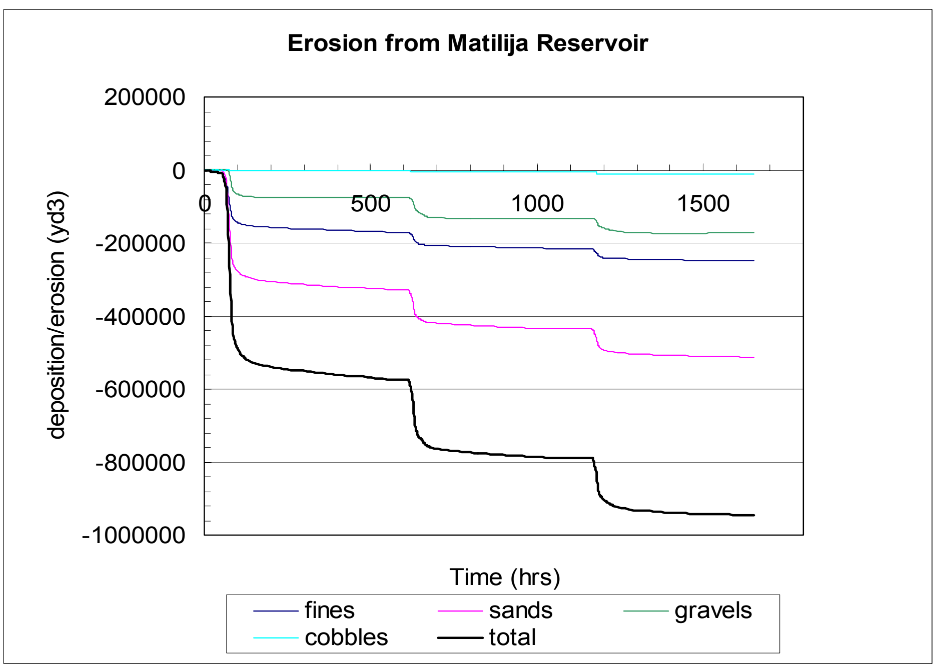


Figure 19.56. Cumulative erosion from the reservoir in alternative 2a for 1991 flood (simulated 3 times in succession).

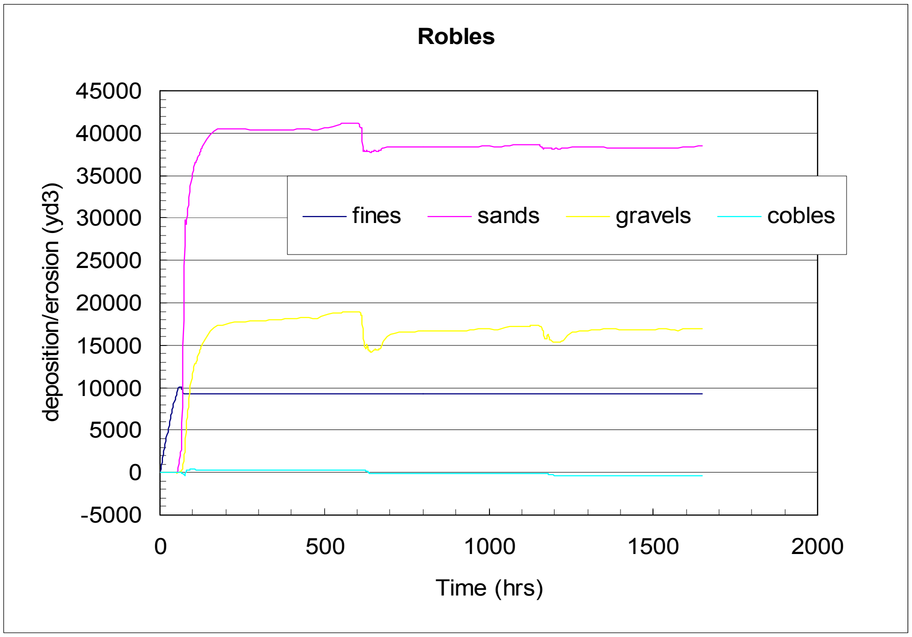


Figure 19.57. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 2a for 1991 flood. Robles Diversion Dam is at RM 14.15.

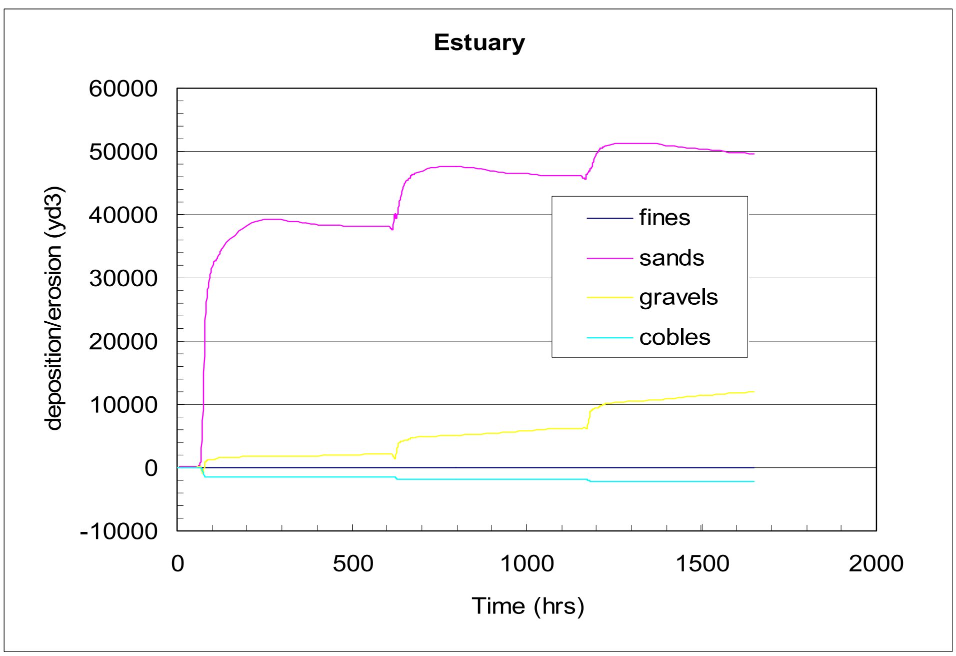


Figure 19.58. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 2a for 1991 flood.

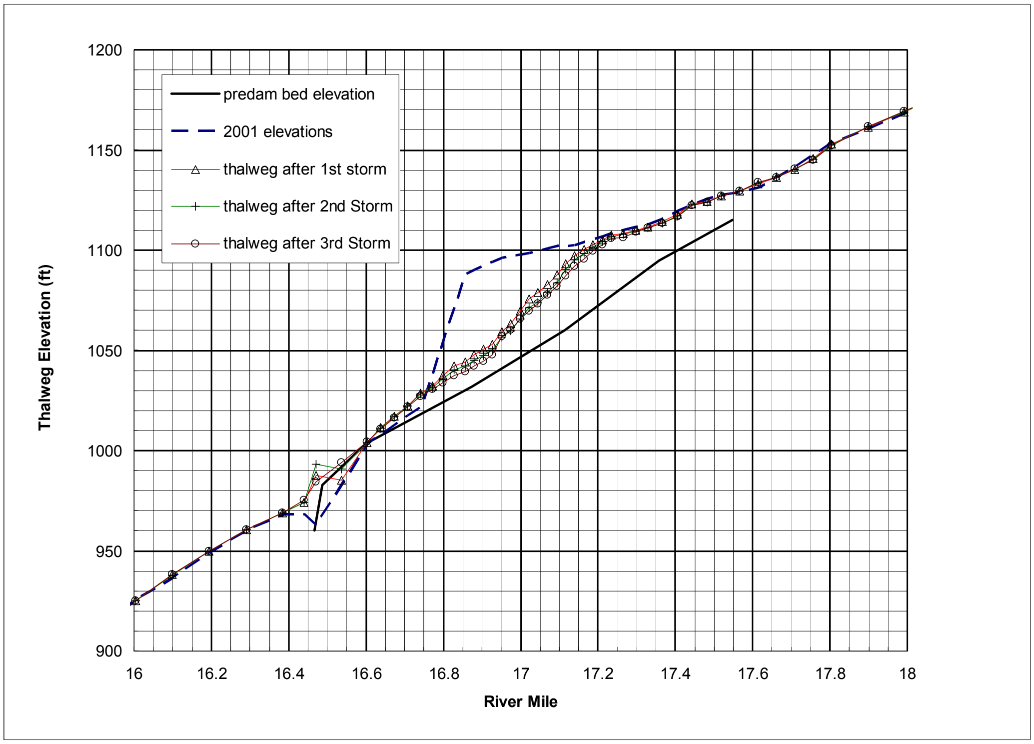


Figure 19.59. Change in thalweg elevation for 1991 flood in reach 7 at various times from start of simulation of alternative 2a.

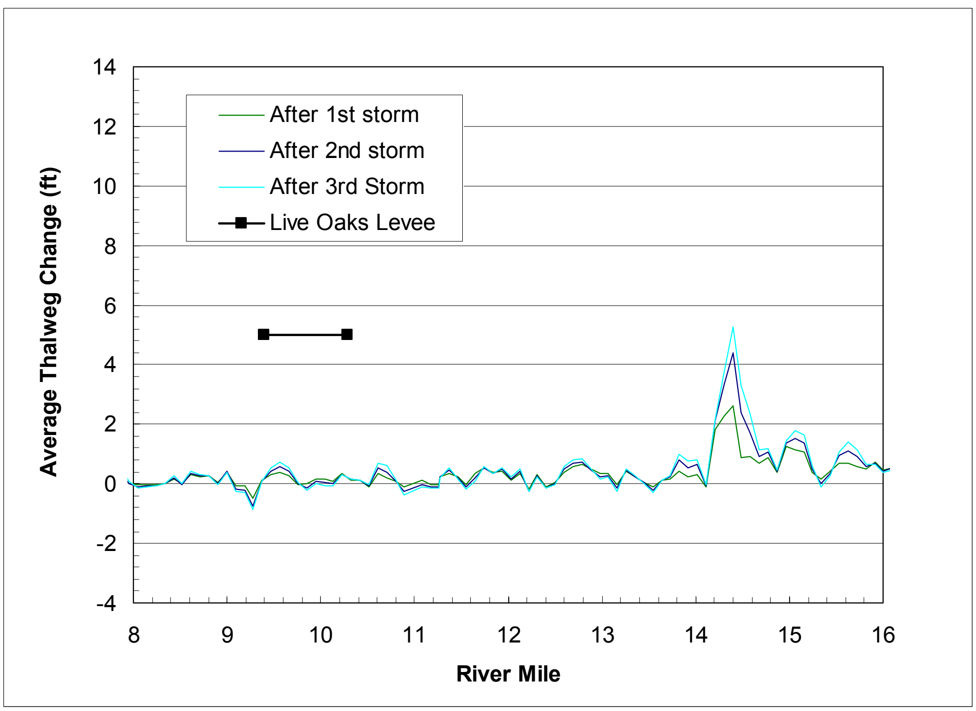


Figure 19.60. Change in thalweg elevation for 1991 flood in reach 4 at various times from start of simulation of alternative 2a.

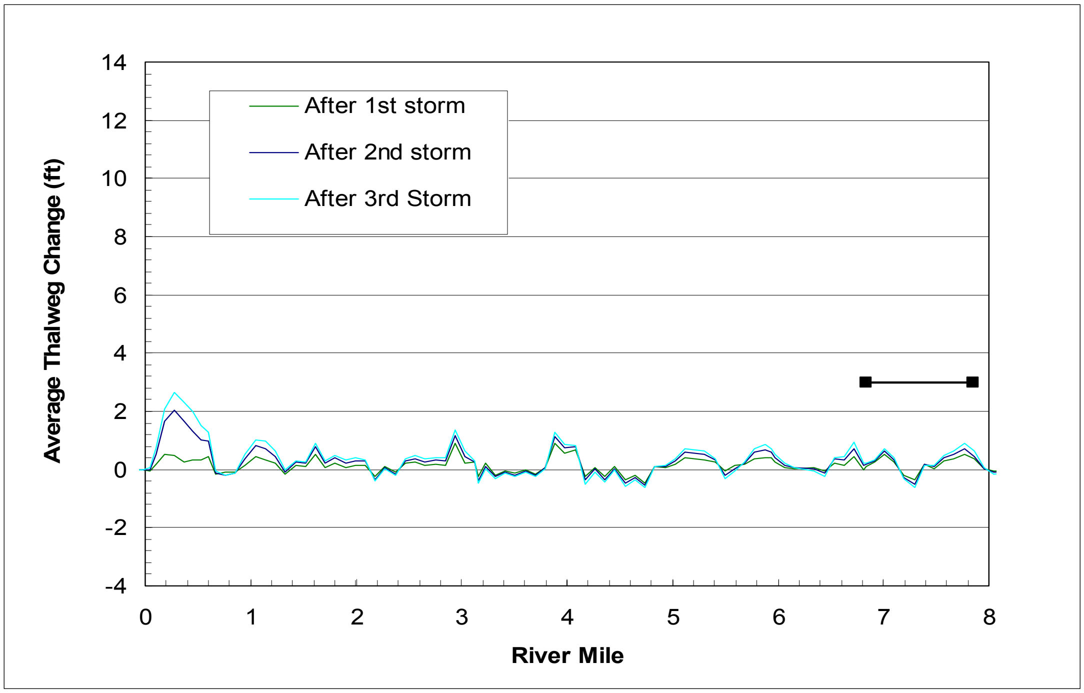


Figure 19.61. Change in thalweg elevation for 1991 flood in reach 3 at various times from start of simulation of alternative 2a.

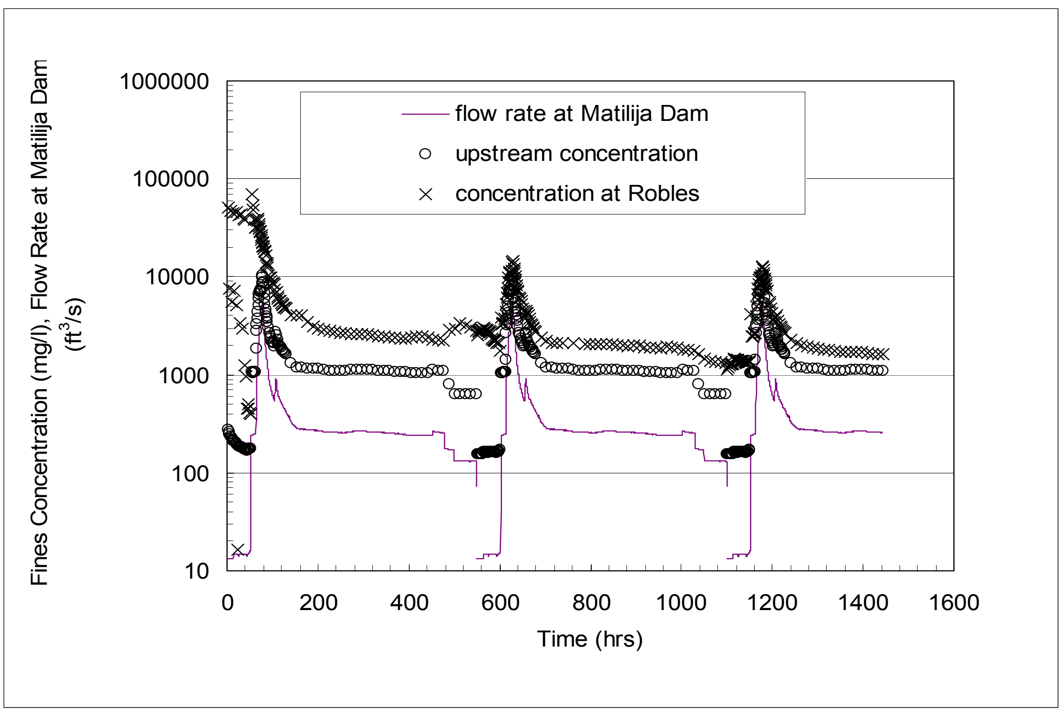


Figure 19.62. Concentrations downstream of dam following removal for 1991 flood for alternative 2a.

19.2.3. Alternative 2b - Full Dam Removal/Natural Sediment Transport with Natural Transport of Reservoir Area Fines

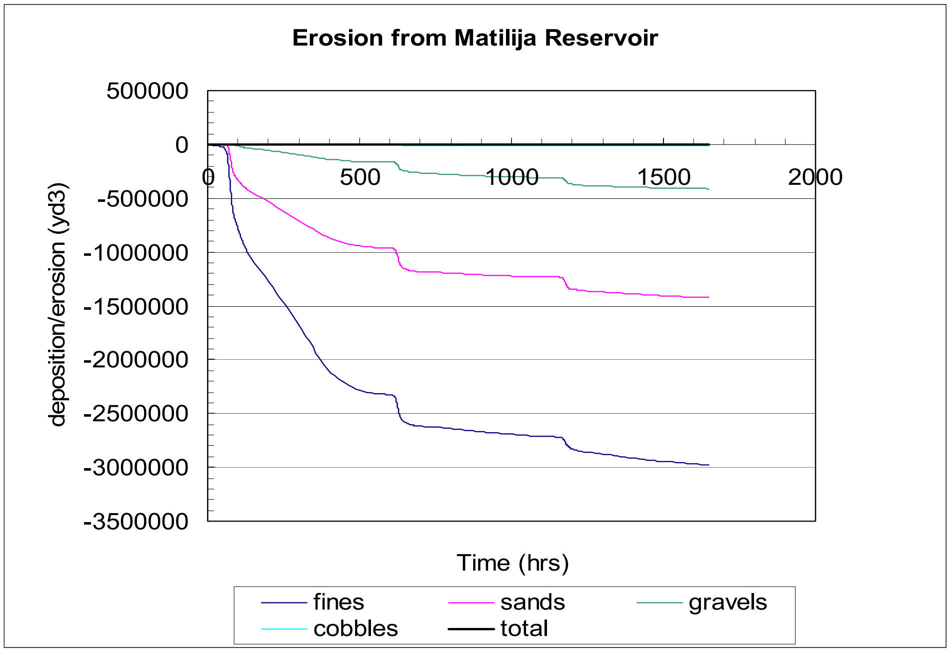


Figure 19.63. Cumulative erosion from the reservoir in alternative 2b for 1991 flood.

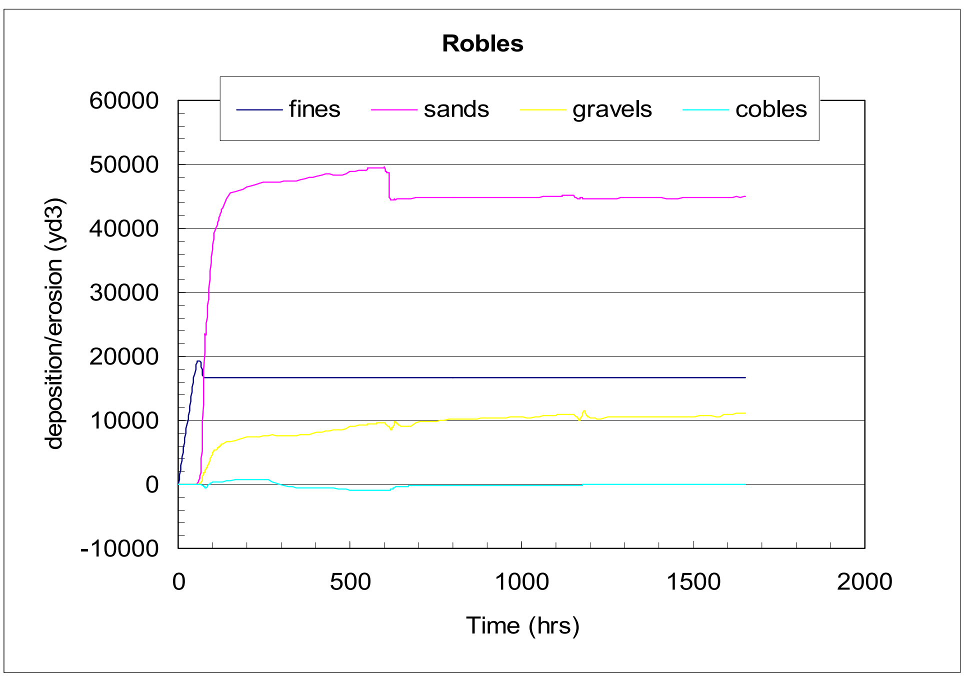


Figure 19.64. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 2b for 1991 flood. Robles Diversion Dam is at RM 14.15.

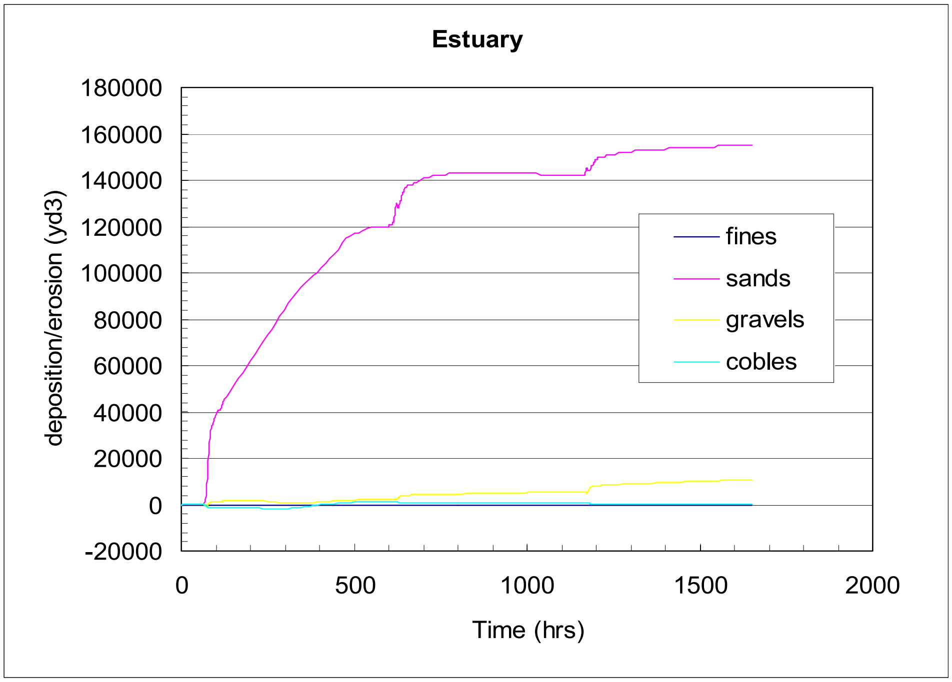


Figure 19.65. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 2b for 1991 flood.

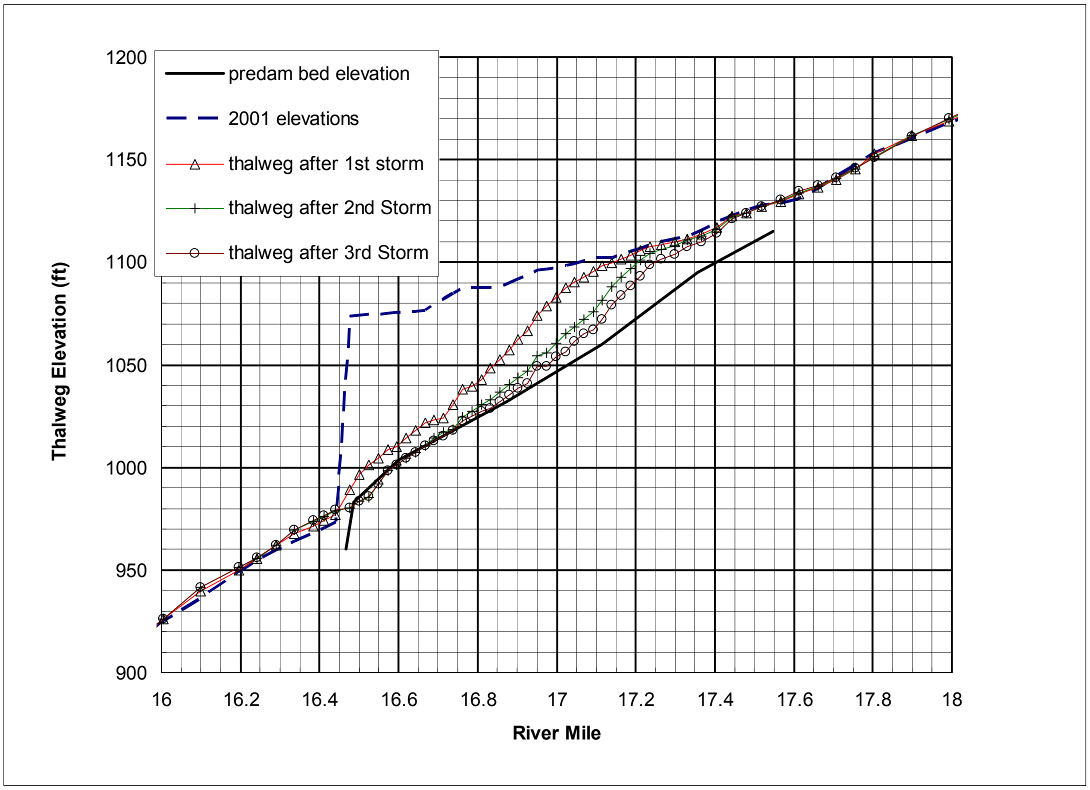


Figure 19.66. Change in thalweg elevation for 1991 flood in reach 7 at various times from start of simulation of alternative 2b.

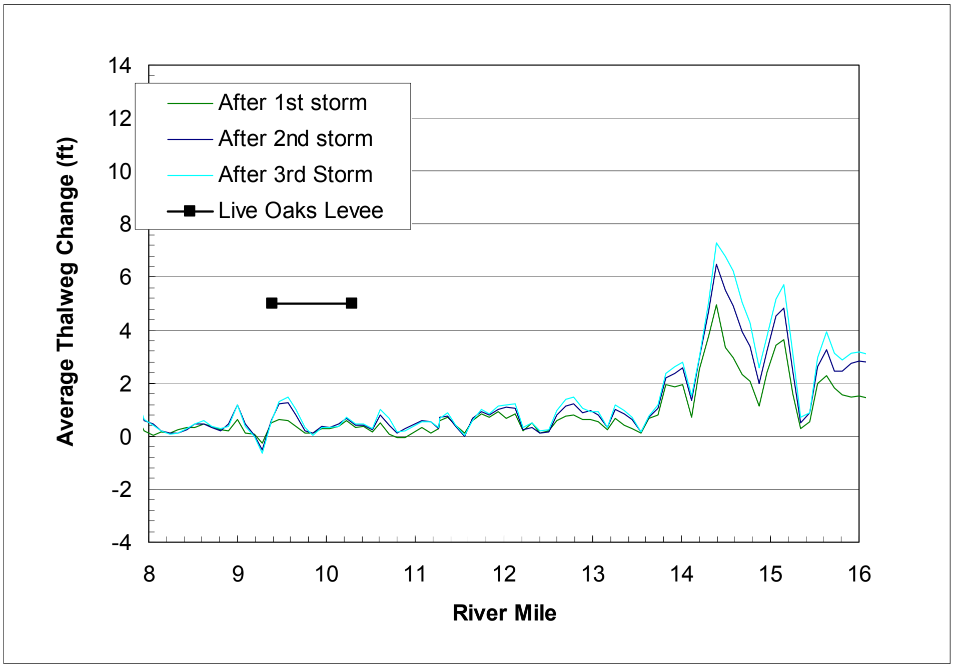


Figure 19.67. Change in thalweg elevation for 1991 flood in reach 4 at various times from start of simulation of alternative 2b.

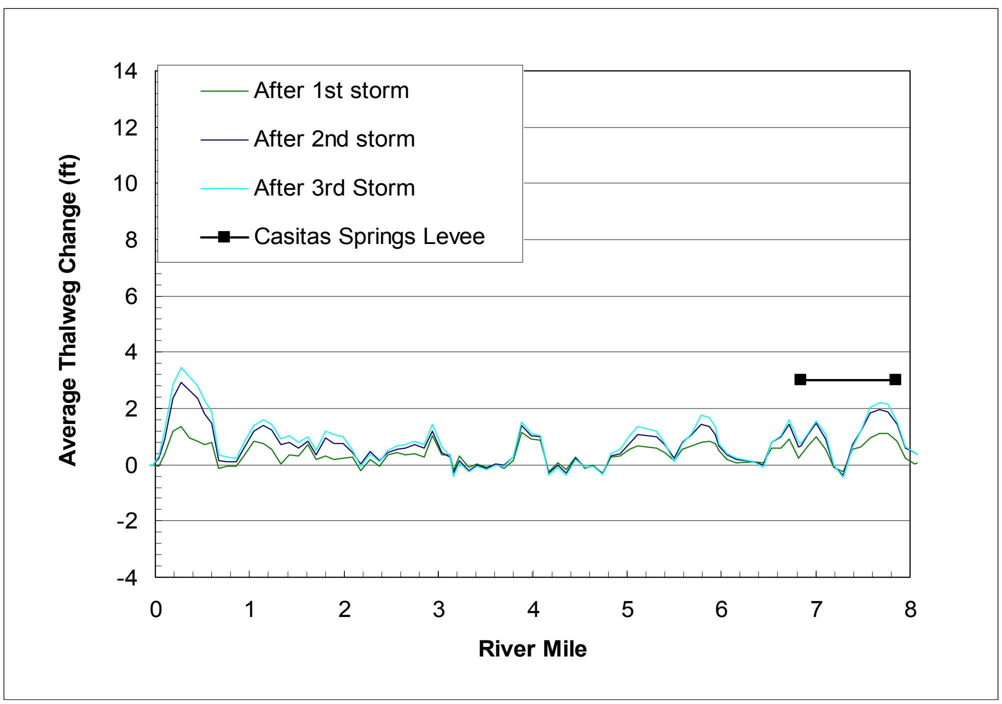


Figure 19.68. Change in thalweg elevation for 1991 flood in reach 3 at various times from start of simulation of alternative 2b.

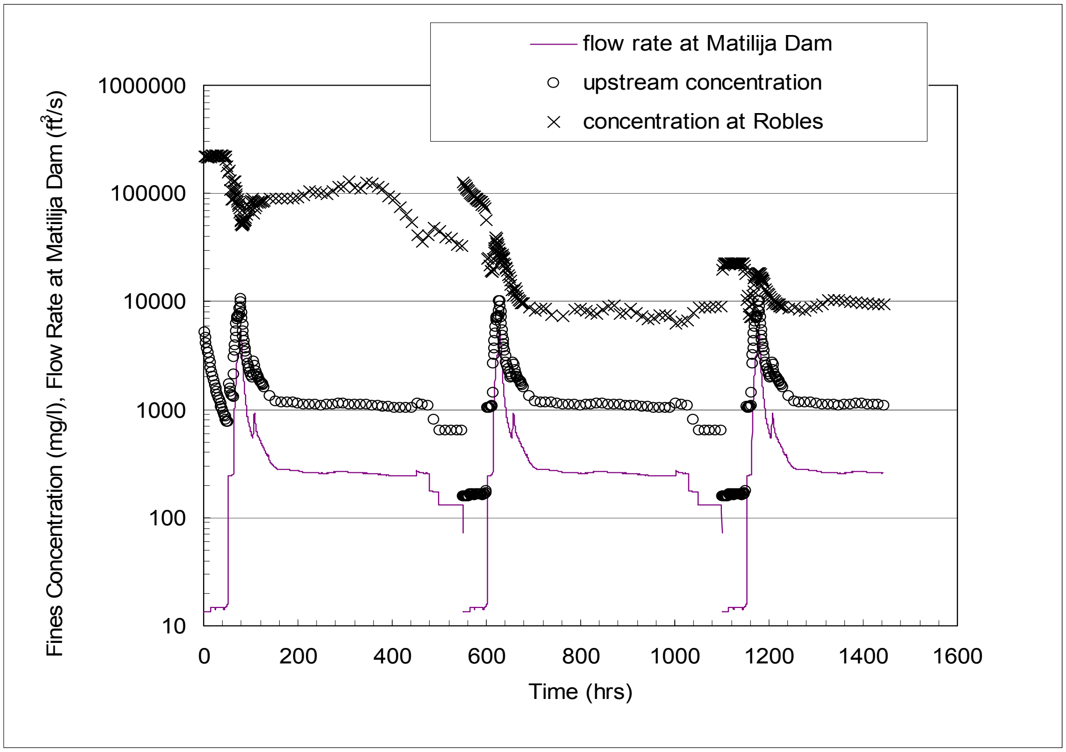


Figure 19.69. Concentrations downstream of dam following removal for 1991 flood for alternative 2b.

19.2.4. ALTERNATIVE 3A – INCREMENTAL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH REMOVAL OF RESERVOIR AREA FINES OFFSITE

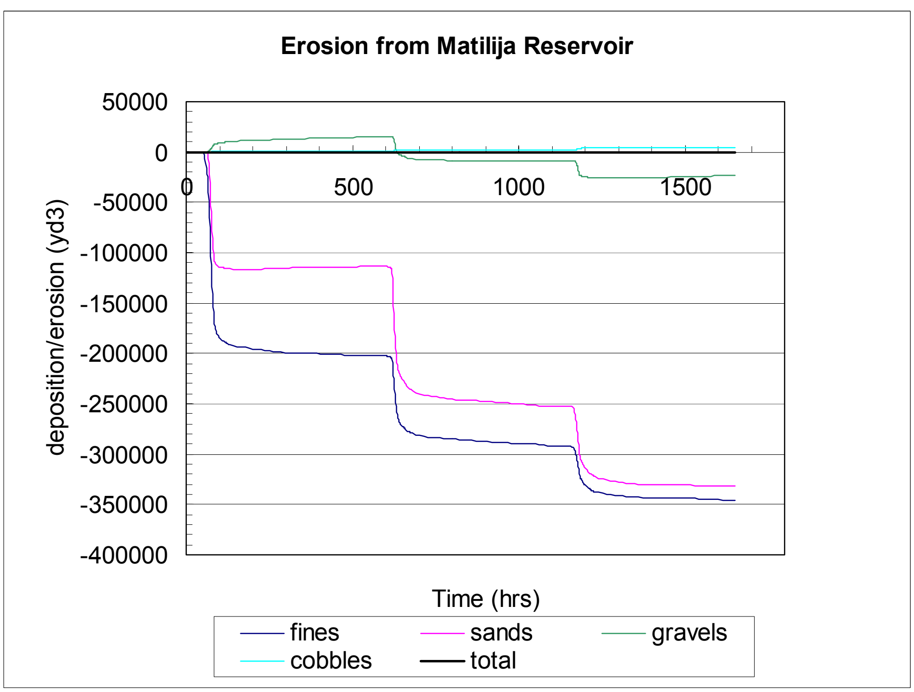


Figure 19.70. Cumulative erosion from the reservoir in alternative 3a for 1991 flood.

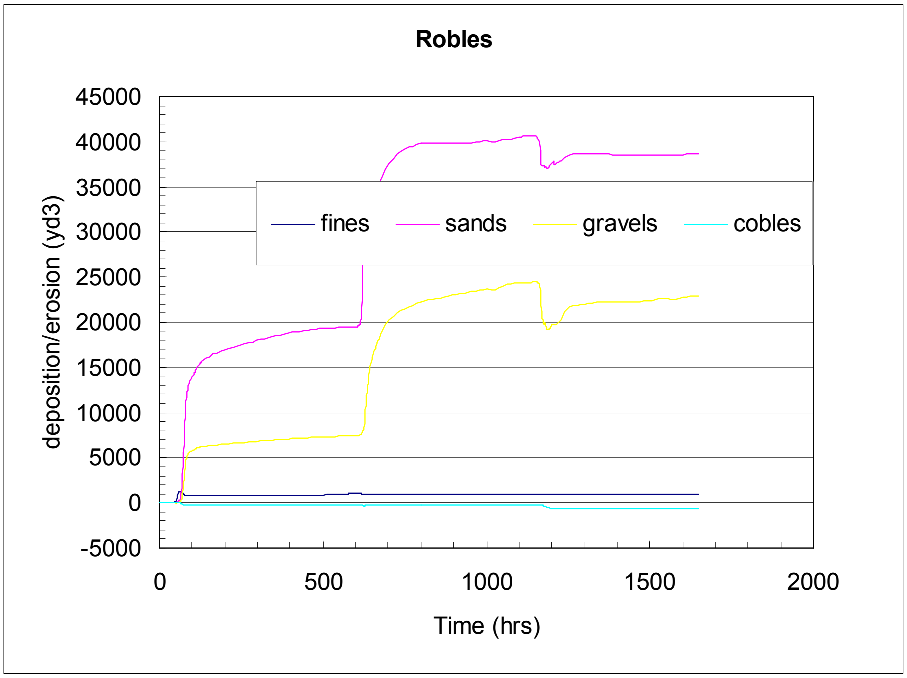


Figure 19.71. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 3a for 1991 flood. Robles Diversion Dam is at RM 14.15.

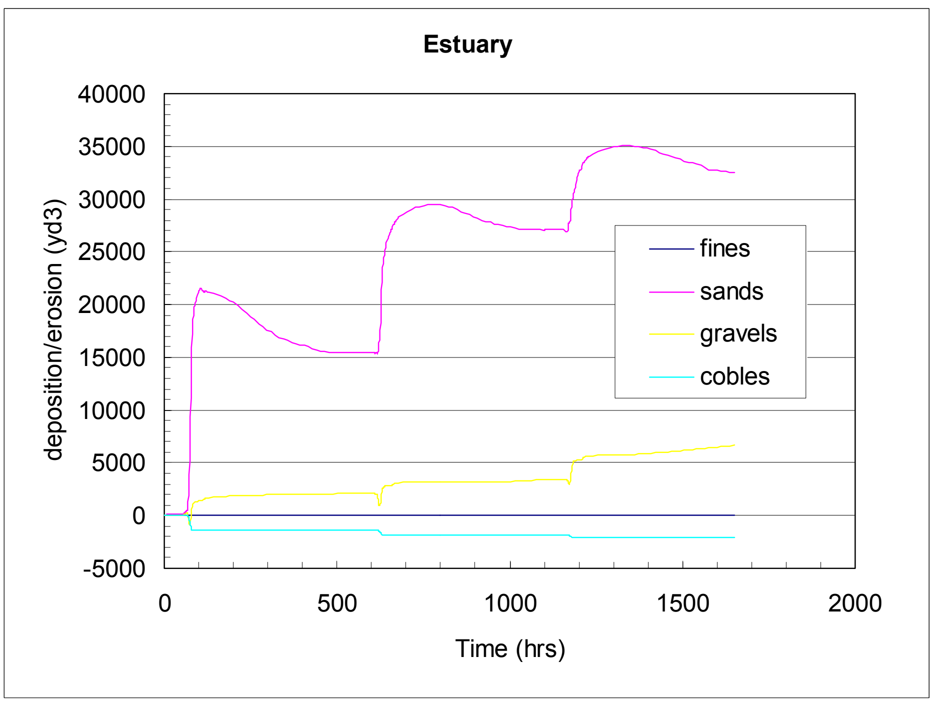


Figure 19.72. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 3a for 1991 flood.

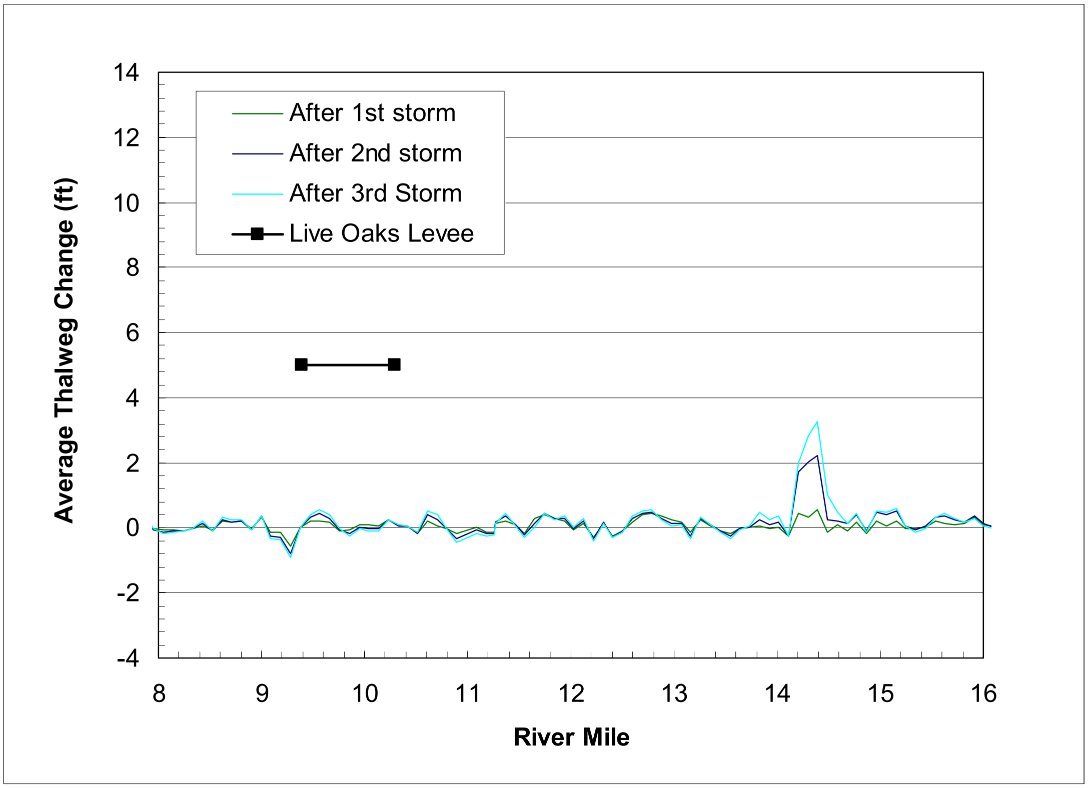


Figure 19.73. Change in thalweg elevation for 1991 flood in reach 4 at various times from start of simulation of alternative 3a.

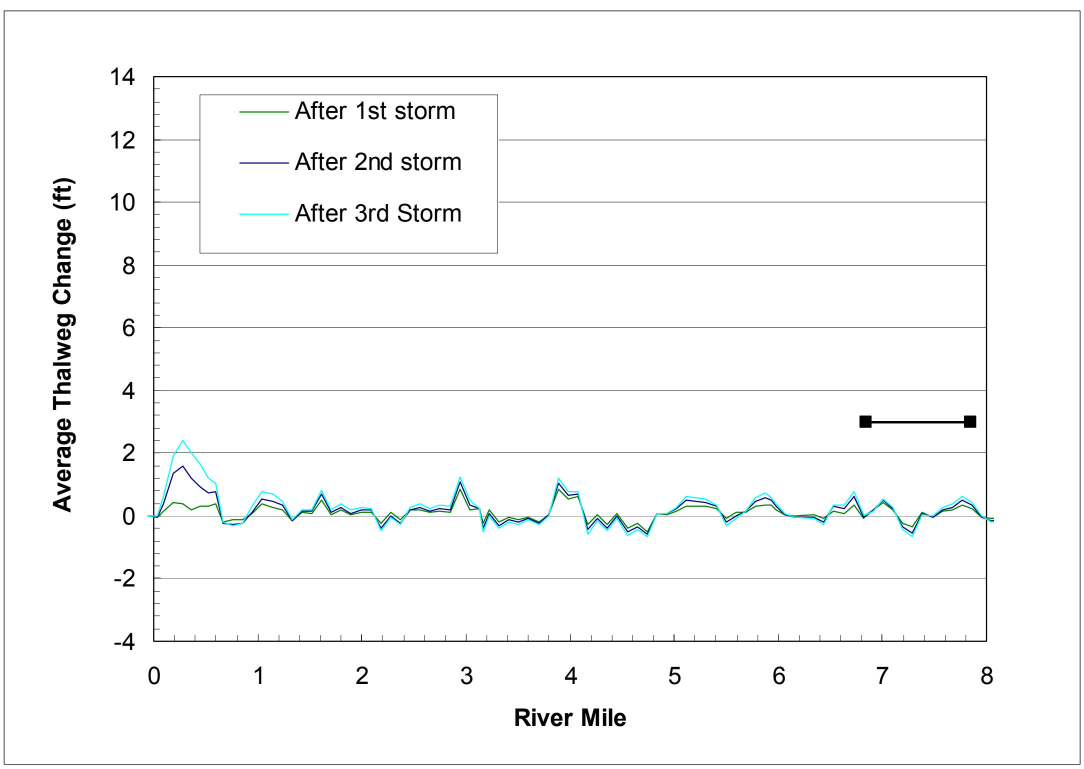


Figure 19.74. Change in thalweg elevation for 1991 flood in reach 3 at various times from start of simulation of alternative 3a.

19.2.5. Alternative 3b – Incremental Dam Removal/Natural Sediment Transport with No Removal of Reservoir Area Fines Offsite

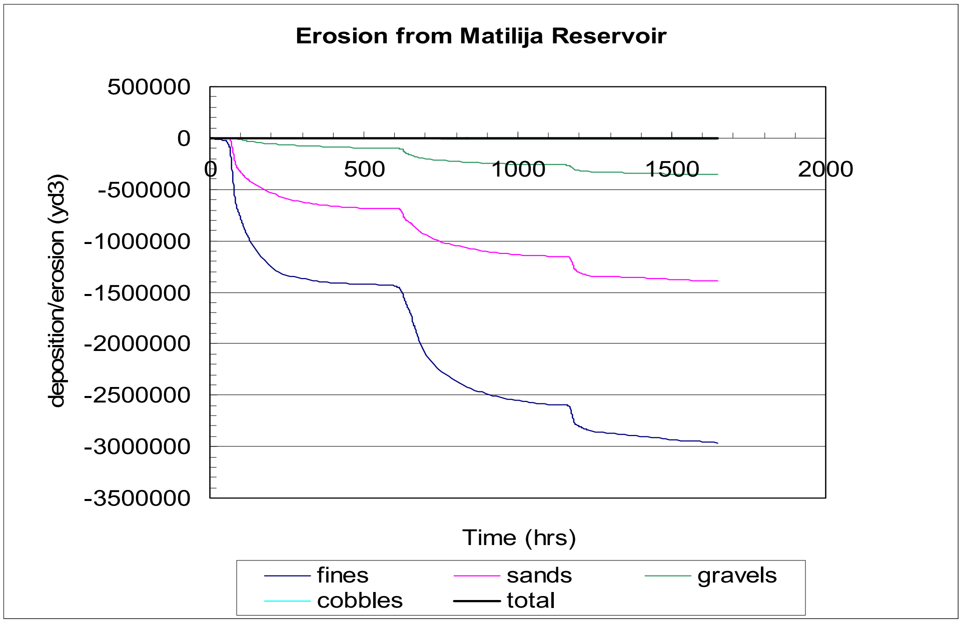


Figure 19.75. Cumulative erosion from the reservoir in alternative 3b for 1991 flood.

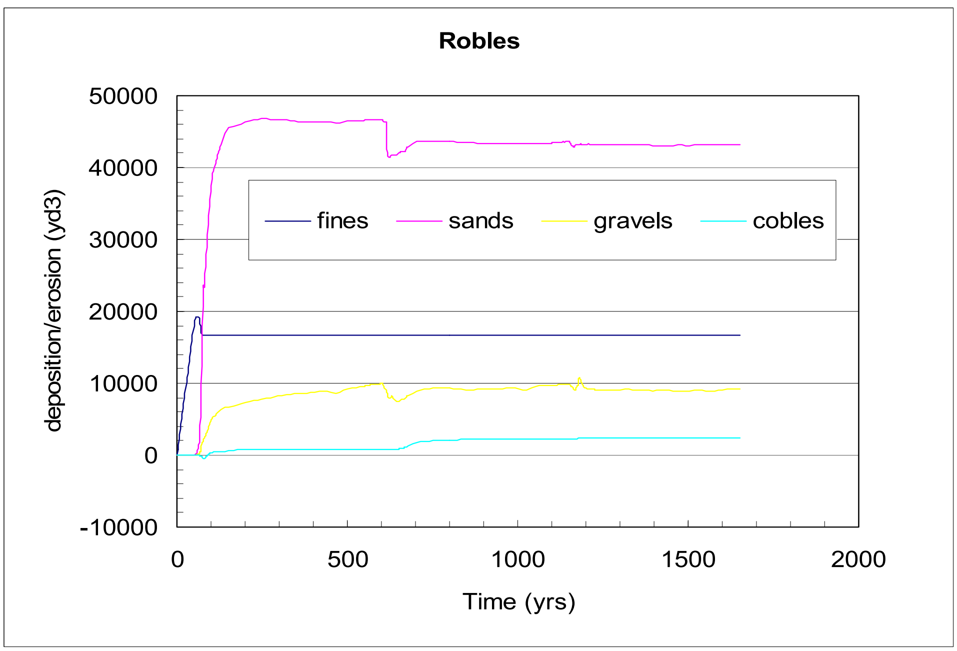


Figure 19.76. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 3b for 1991 flood. Robles Diversion Dam is at RM 14.15.

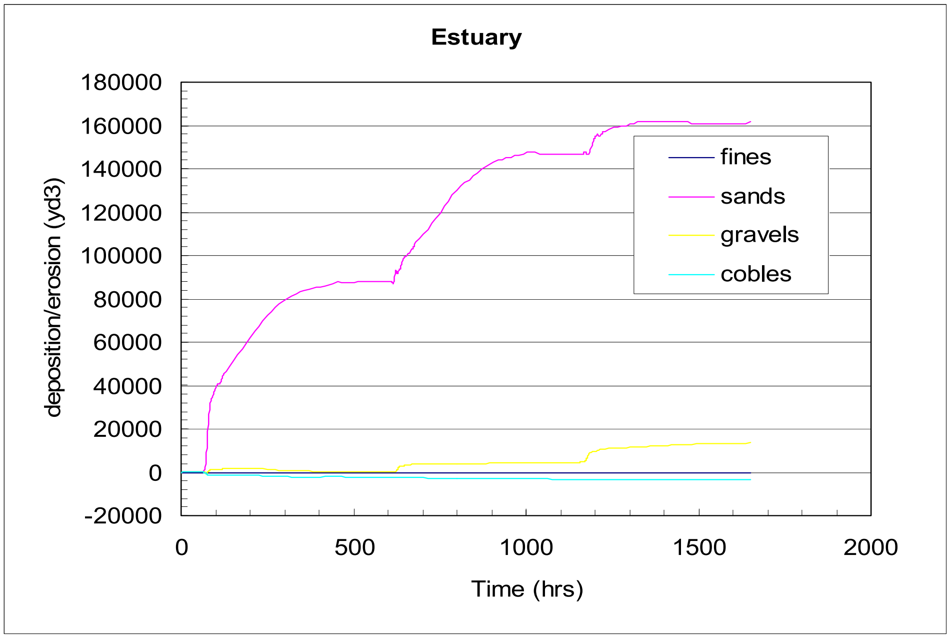


Figure 19.77. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 3b for 1991 flood.

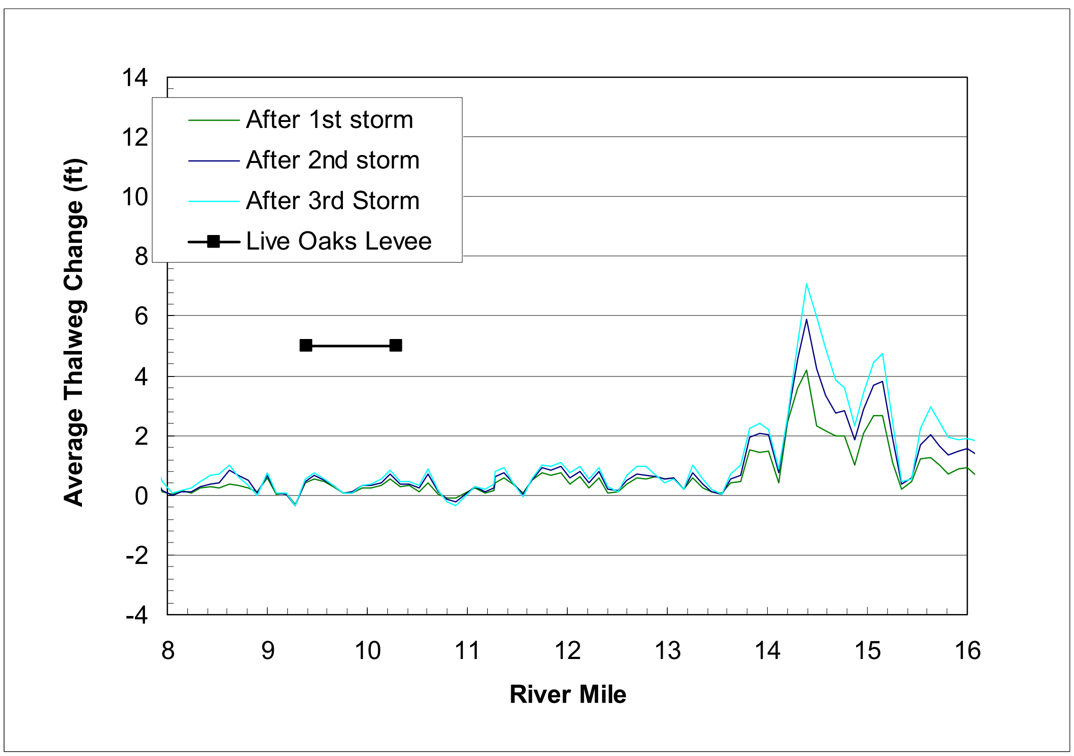


Figure 19.78. Change in thalweg elevation for 1991 flood in reach 4 at various times from start of simulation of alternative 3b.

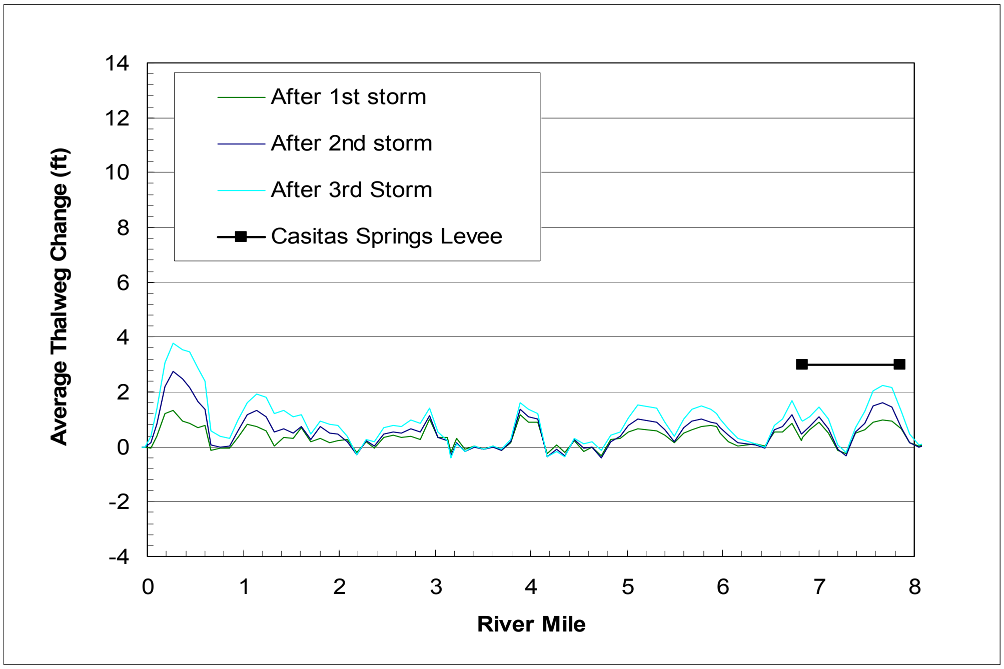


Figure 19.79. Change in thalweg elevation for 1991 flood in reach 3 at various times from start of simulation of alternative 3b.

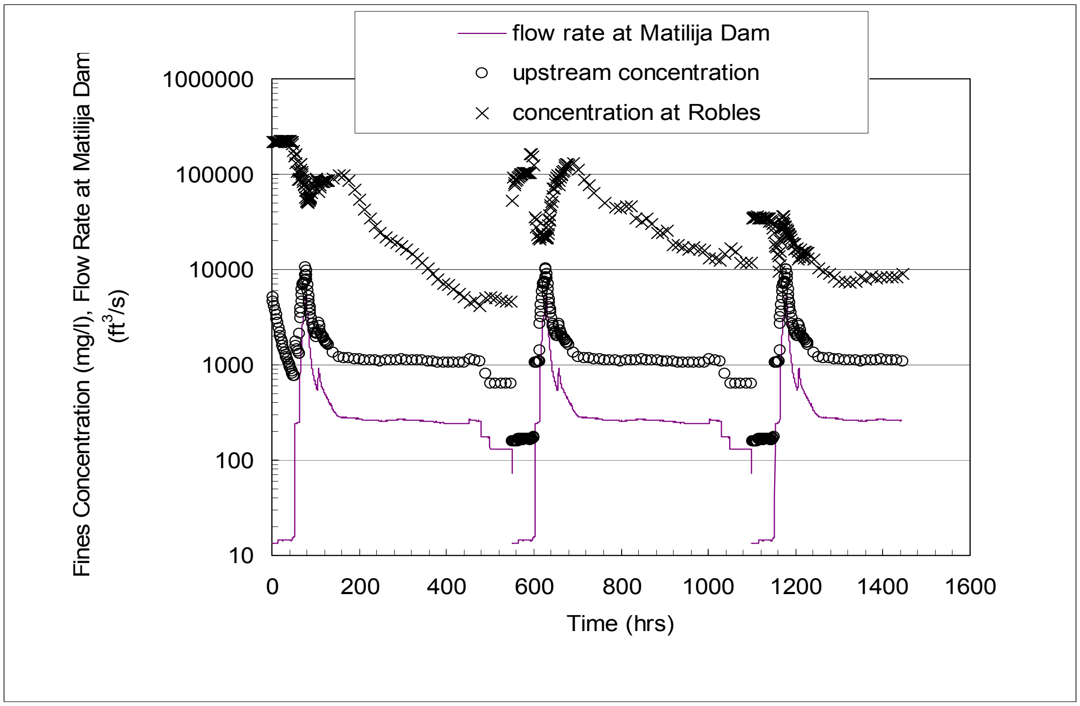


Figure 19.80. Concentrations downstream of dam following removal for 1991 flood for alternative 3b.

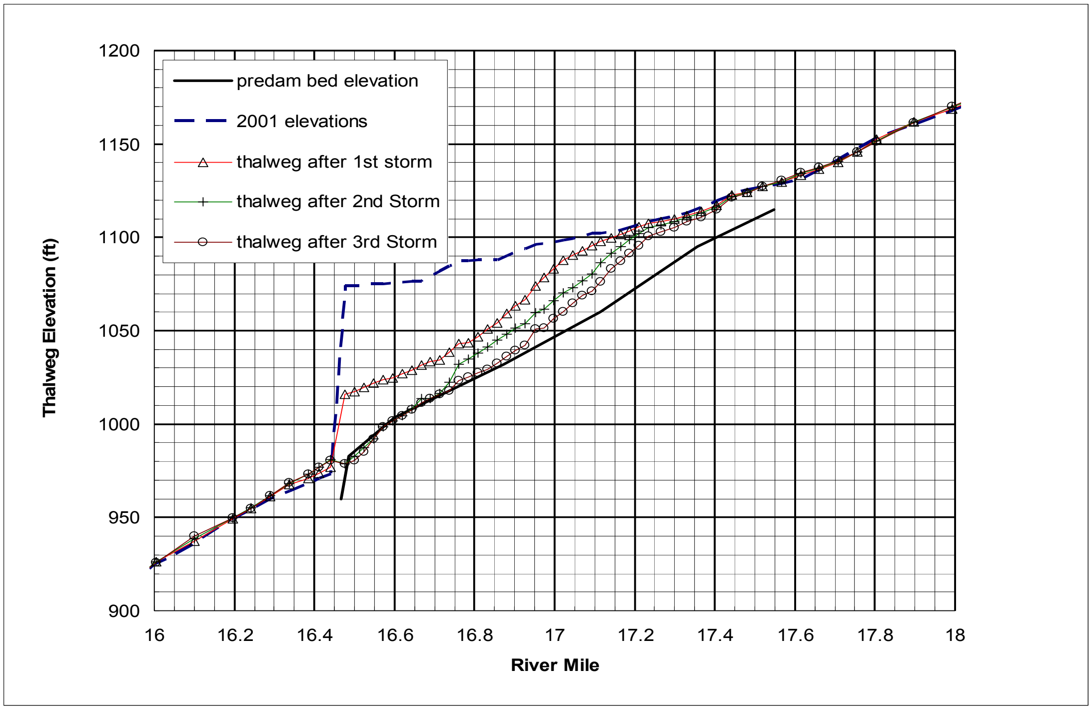


Figure 19.81. Thalweg elevations upstream of Matilija Dam for 1991 flood for alternative 3b.

19.2.6. ALTERNATIVE 4B – FULL DAM REMOVAL/TEMPORARILY STABILIZE SEDIMENTS ON SITE

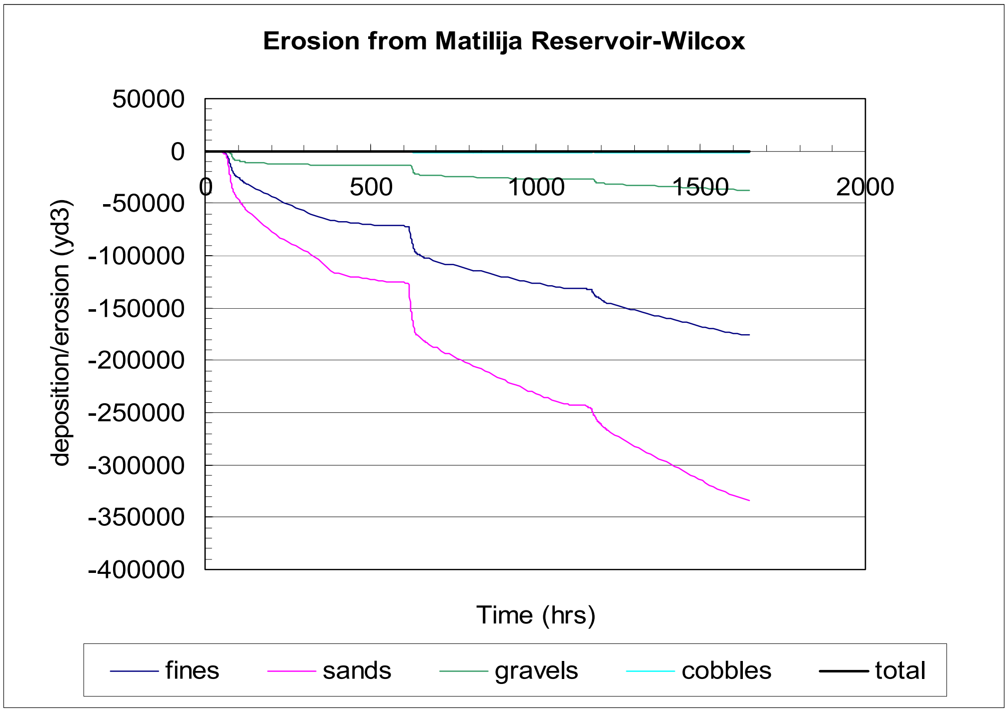


Figure 19.82. Cumulative erosion from the reservoir in alternative 4b for 1991 flood.

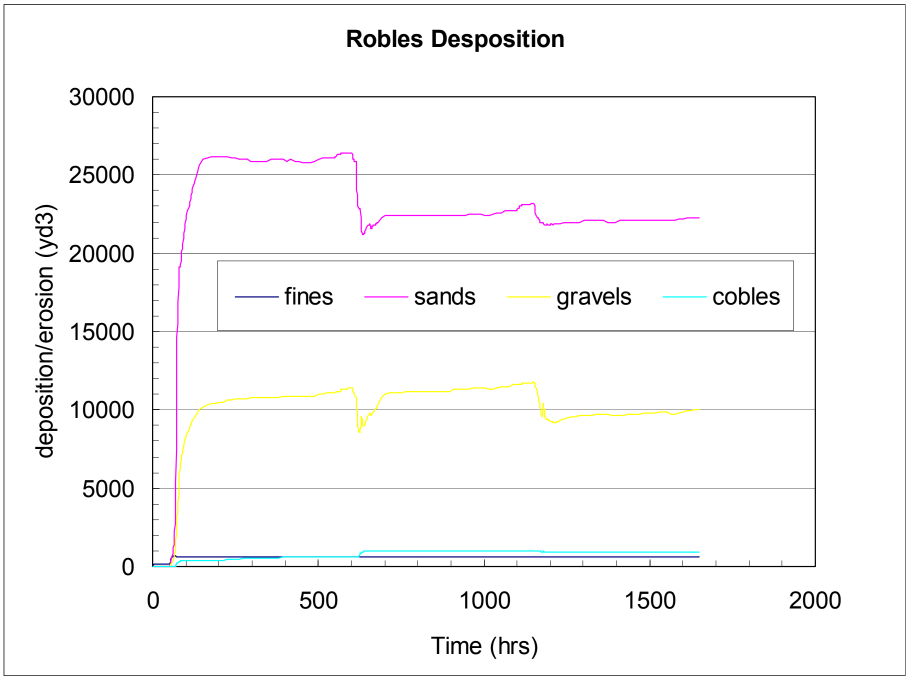


Figure 19.83. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 4b for 1991 flood. Robles Diversion Dam is at RM 14.15.

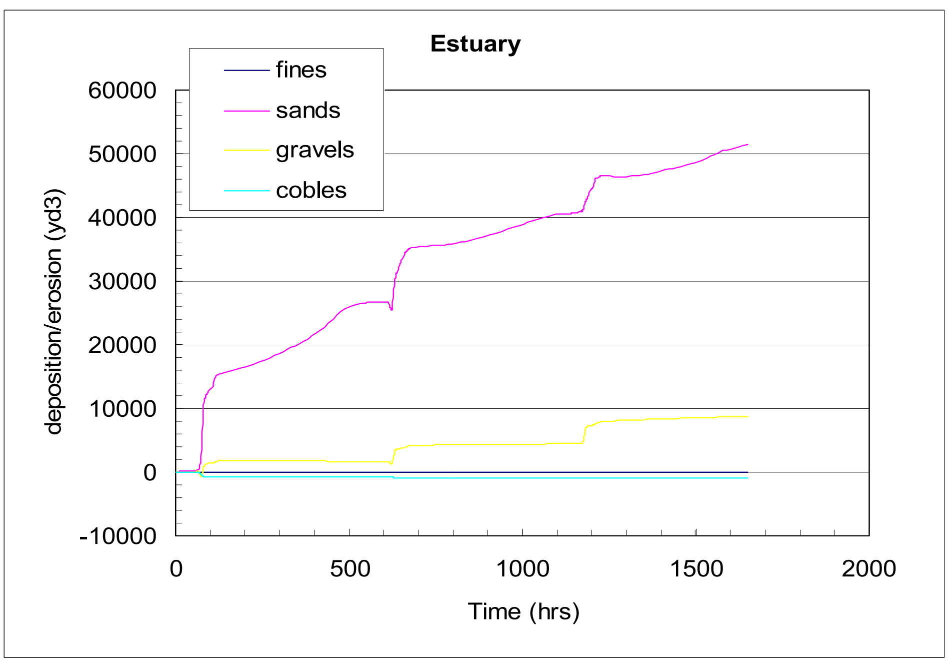


Figure 19.84. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 4b for 1991 flood.

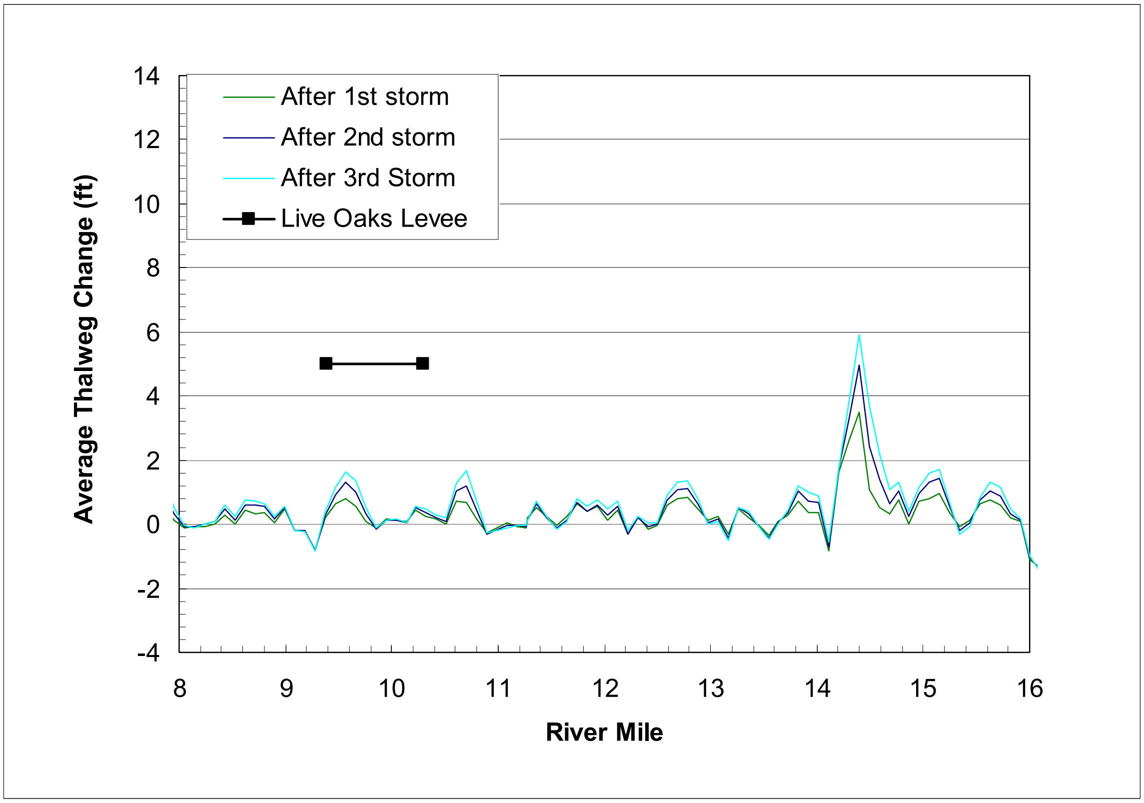


Figure 19.85. Change in thalweg elevation for 1991 flood in reach 4 at various times from start of simulation of alternative 4b.

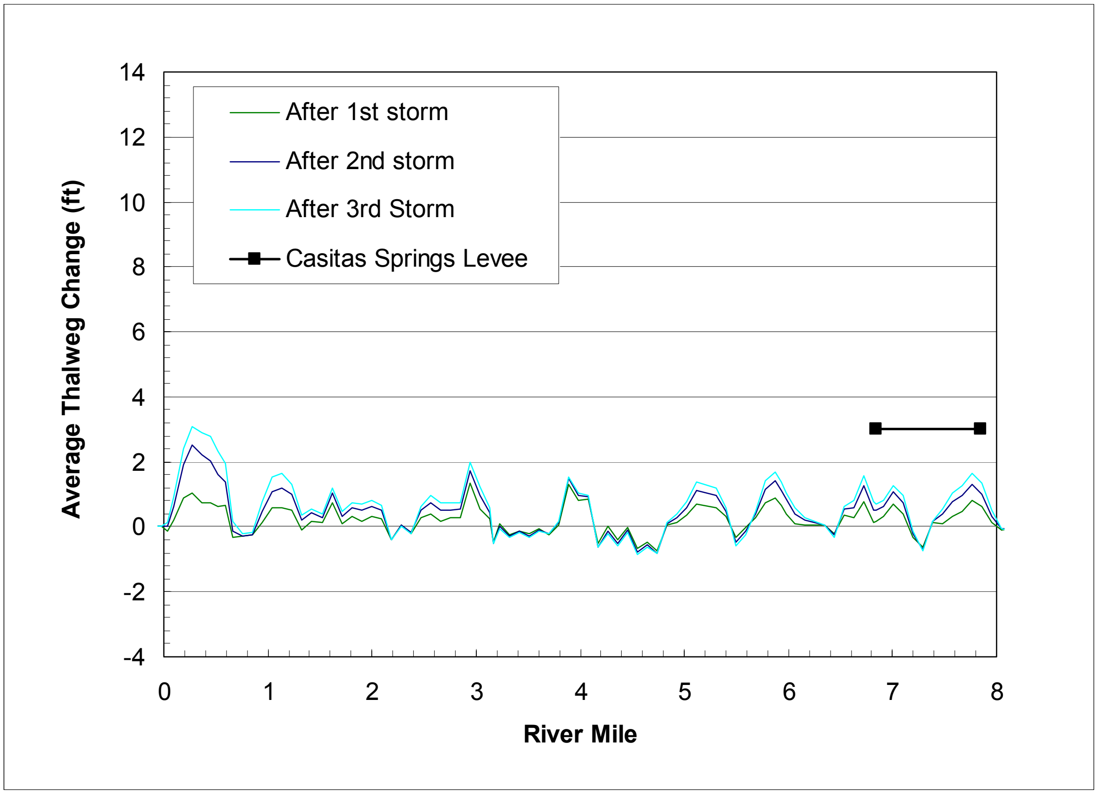


Figure 19.86. Change in thalweg elevation for 1991 flood in reach 3 at various times from start of simulation of alternative 4b.

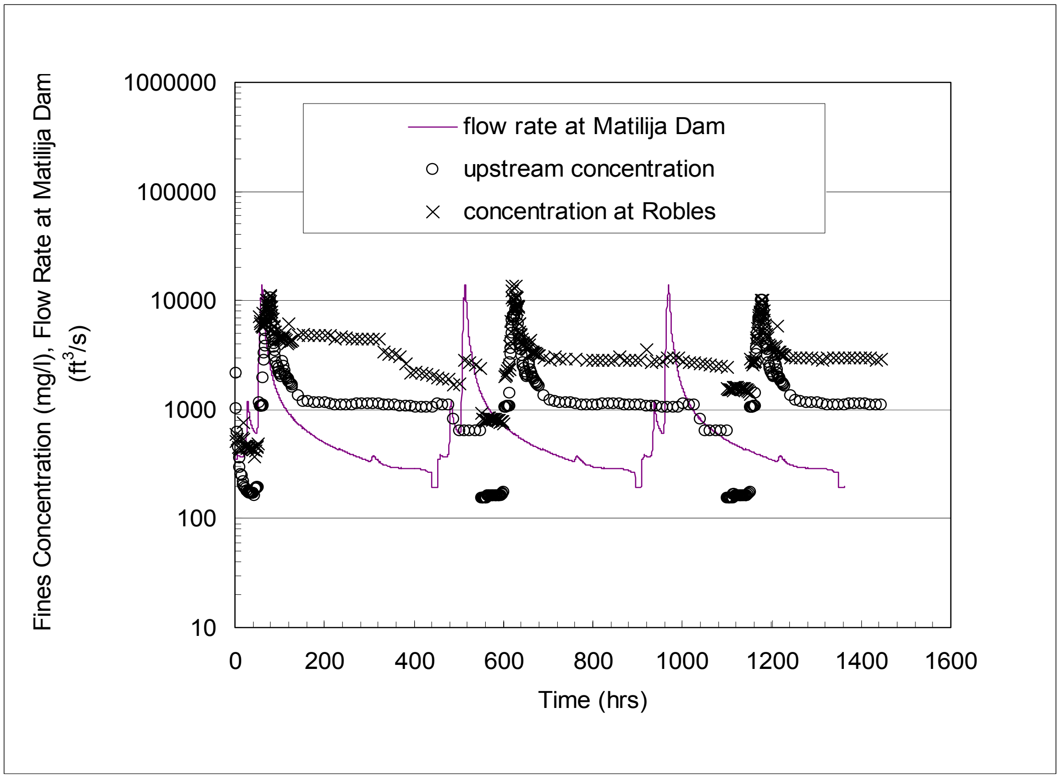


Figure 19.87. Concentrations downstream of dam following removal for 1991 flood for alternative 4b.

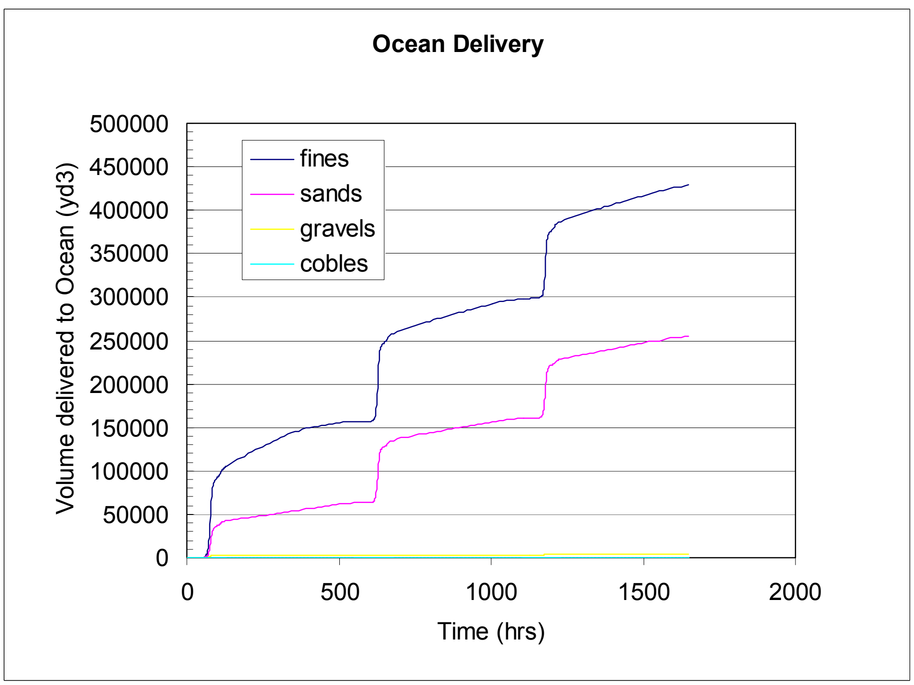


Figure 19.88. Sediment Delivery to ocean for 1991 flood for alternative 4b.

19.2.7. ALTERNATIVES 1 AND 4A – COMPLETE REMOVAL OF DAM AND RESERVOIR SEDIMENTS FROM RIVER SYSTEM

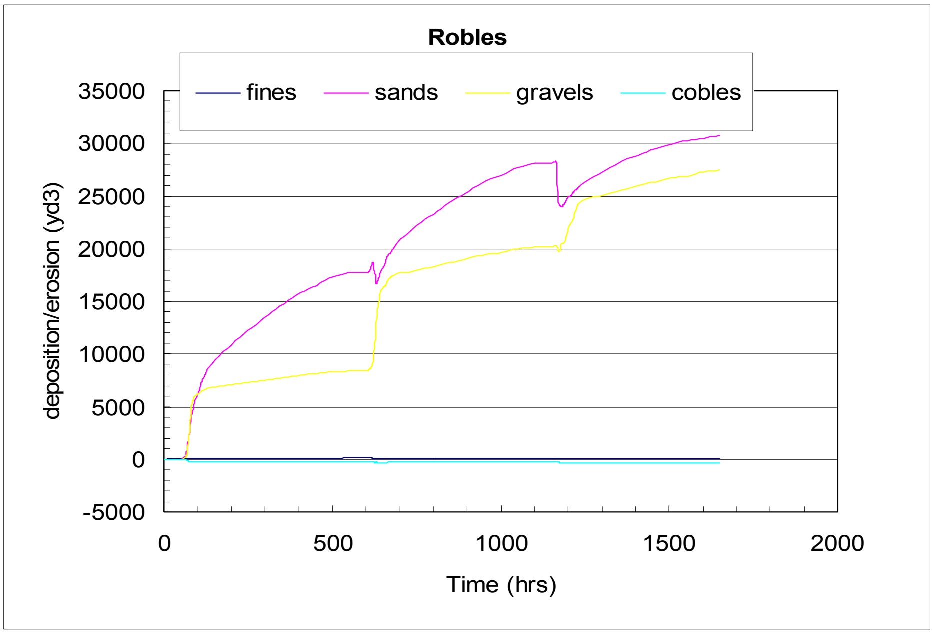


Figure 19.89. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 1 and 4a for 1991 flood (simulated 3 times in succession). Robles Diversion Dam is at RM 14.15.

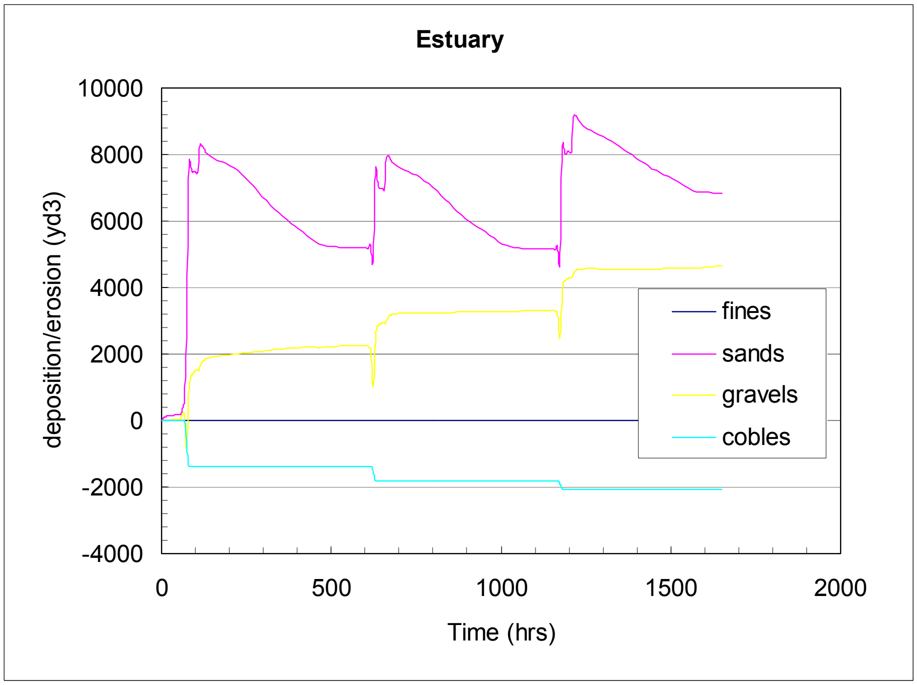


Figure 19.90. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 1 and 4a for 1991 flood (simulated 3 times in succession).

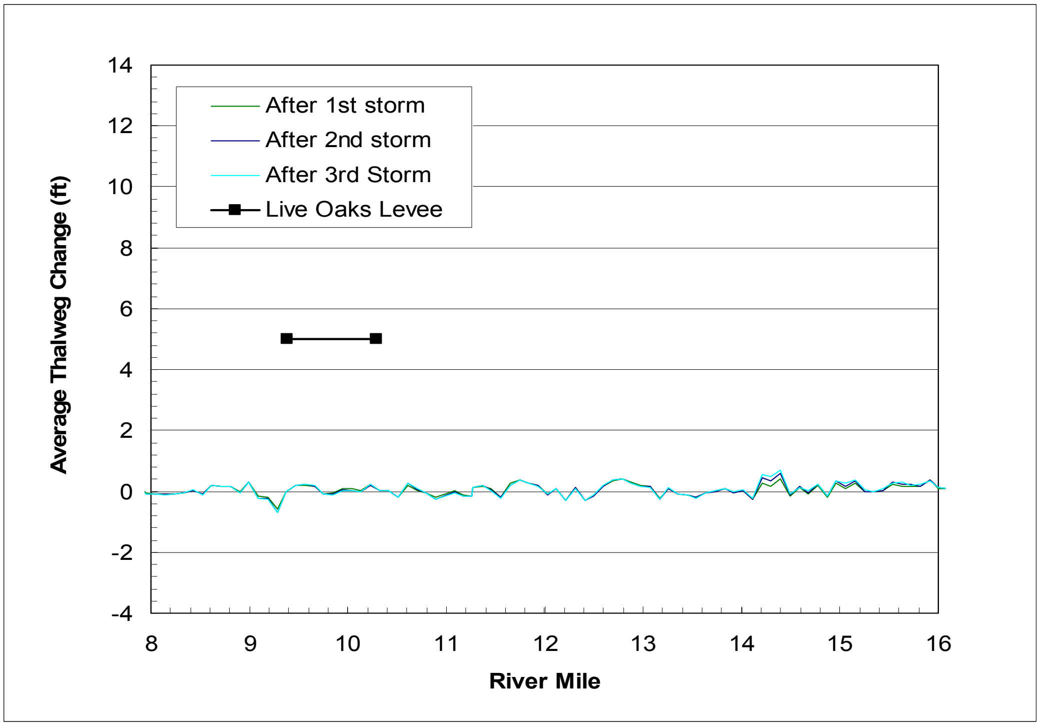


Figure 19.91. Change in thalweg elevation for 1991 flood (simulated 3 times in succession) in reach 4 at various times from start of simulation of alternative 1 and 4a.

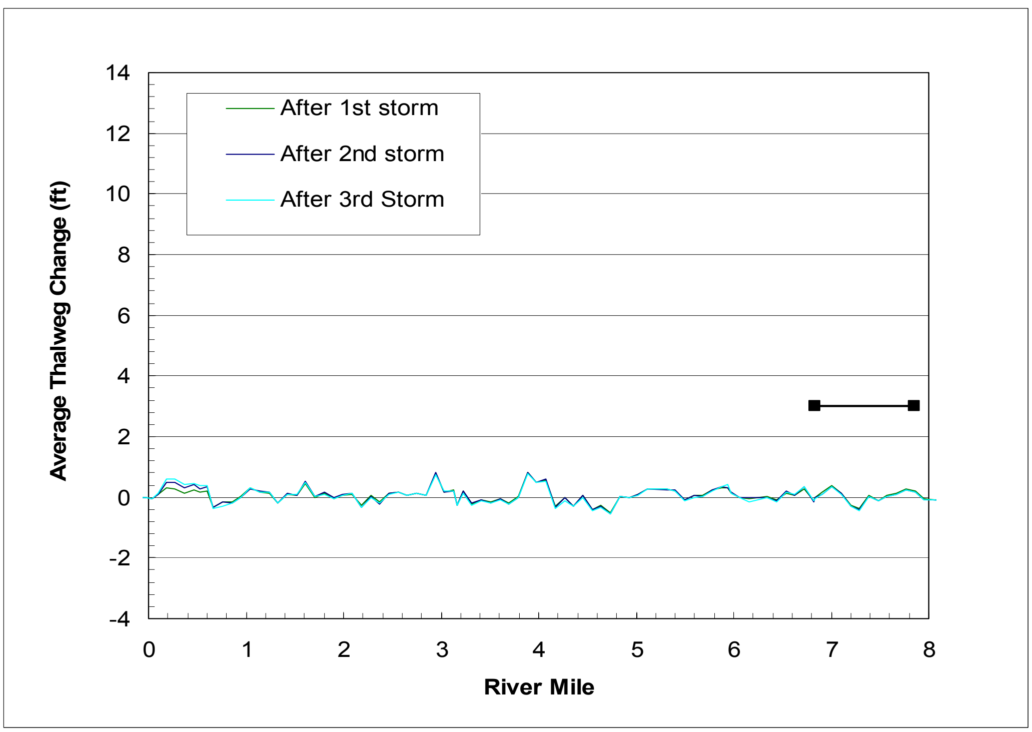


Figure 19.92. Change in thalweg elevation for 1991 flood (simulated 3 times in succession) in reach 3 at various times from start of simulation of alternative 1 and 4a.

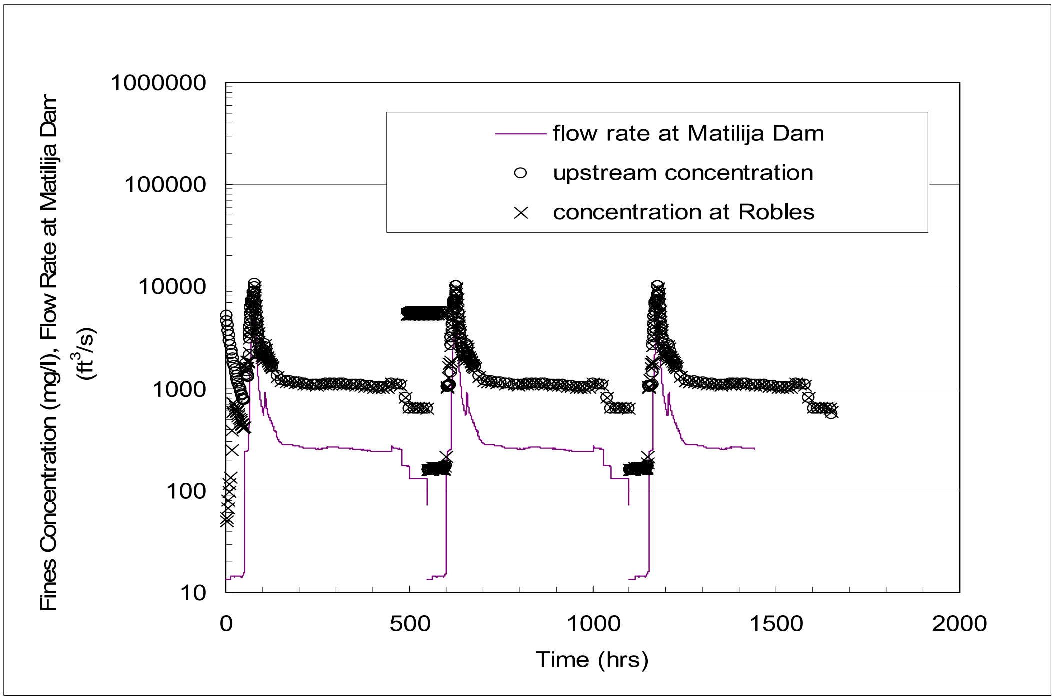


Figure 19.93. Concentrations following removal for 1991 flood (simulated 3 times in succession) for alternative 1 and 4a.

19.3. Model Results for 100-yr Flood

19.3.1. No Action Alternative

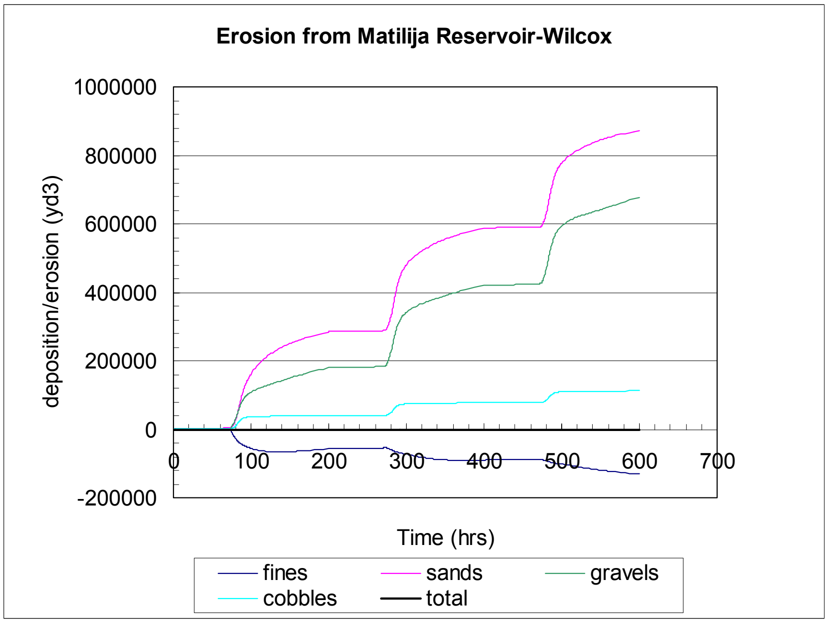


Figure 19.94. Cumulative erosion from the reservoir in No Action alternative for 100-yr flood.

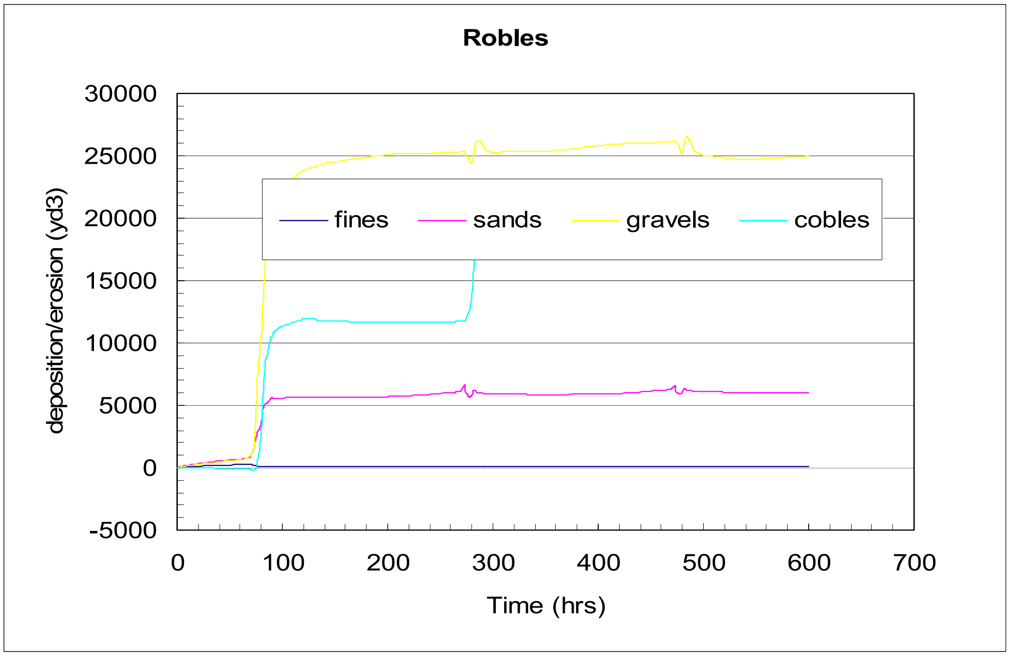


Figure 19.95. Cumulative deposition from RM 14.15 to RM 14.5 in No Action alternative for 100-yr flood. Robles Diversion Dam is at RM 14.15.

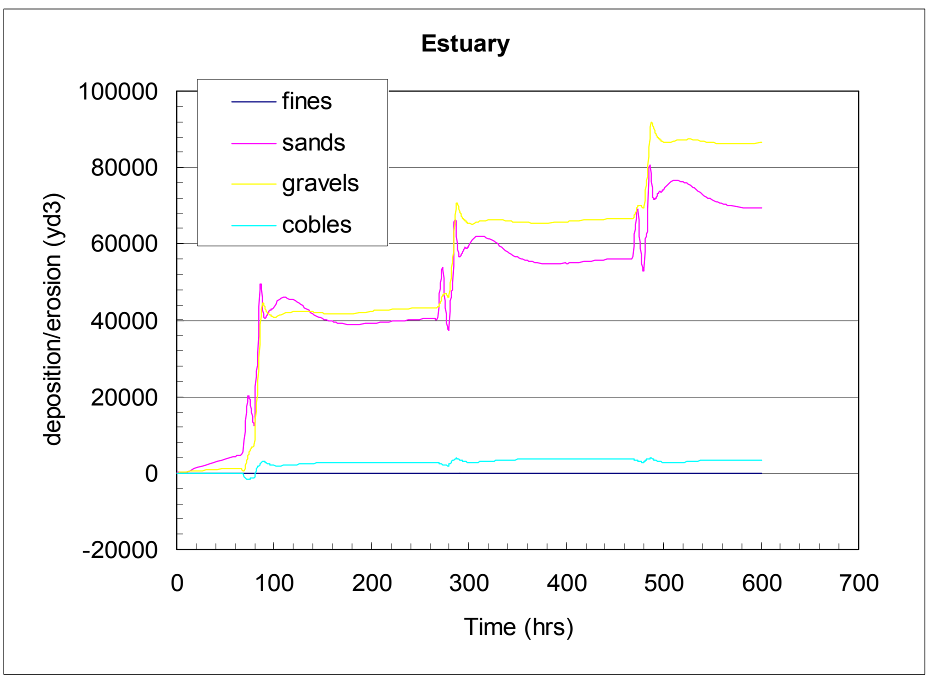


Figure 19.96. Cumulative deposition in Estuary (RM 0.6 - 0.2) in No Action alternative for 100-yr flood.

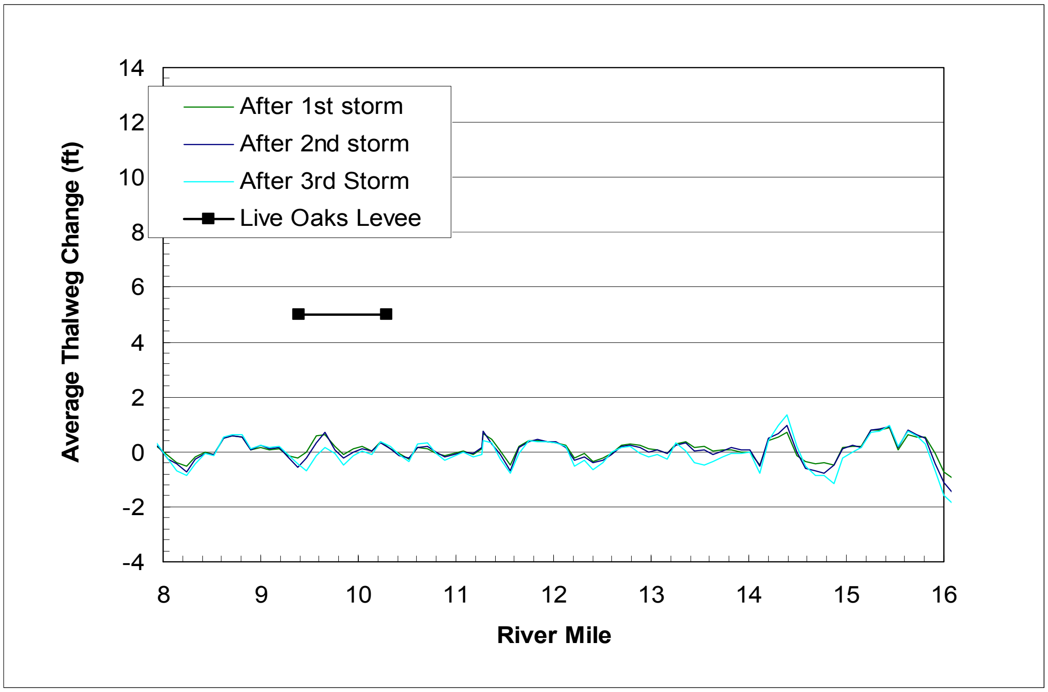


Figure 19.97. Change in thalweg elevation for 100-yr flood in reach 4 at various times from start of simulation of No Action alternative.

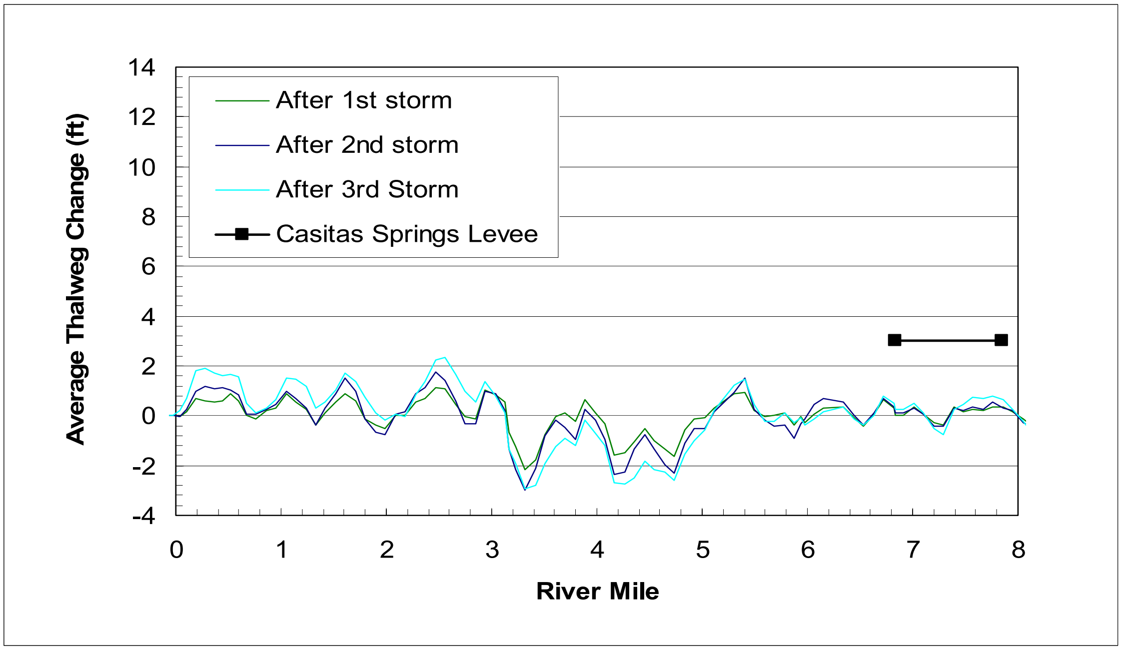


Figure 19.98. Change in thalweg elevation for 100-yr flood in reach 3 at various times from start of simulation of No Action alternative.

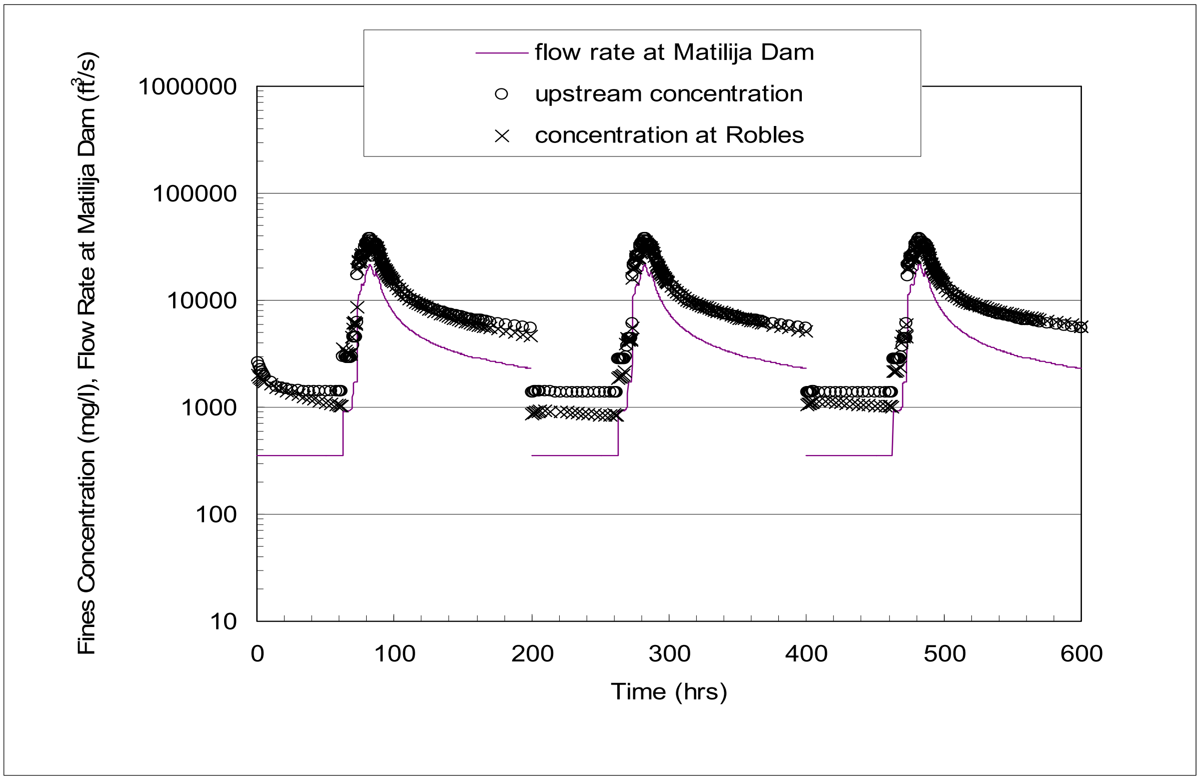


Figure 19.99. Concentrations downstream of dam following removal for 100-yr flood for No Action alternative.

19.3.2. Alternative 2a - Full Dam Removal/Natural Sediment Transport with Removal of Reservoir Area Fines Offsite

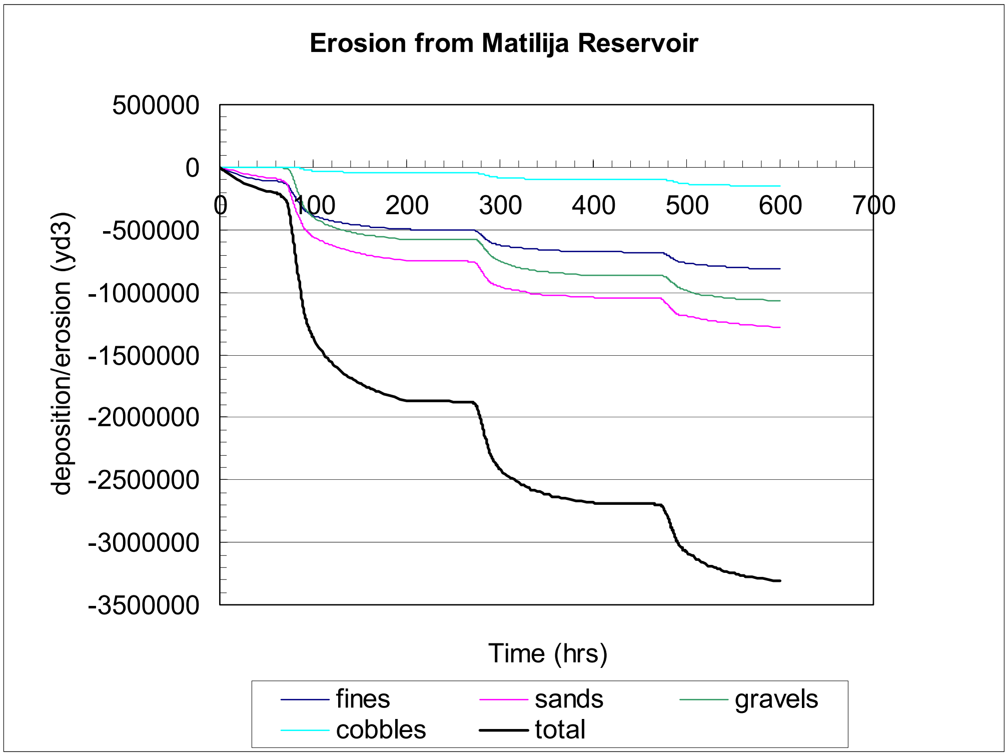


Figure 19.100. Cumulative erosion from the reservoir in alternative 2a for 100-yr flood.

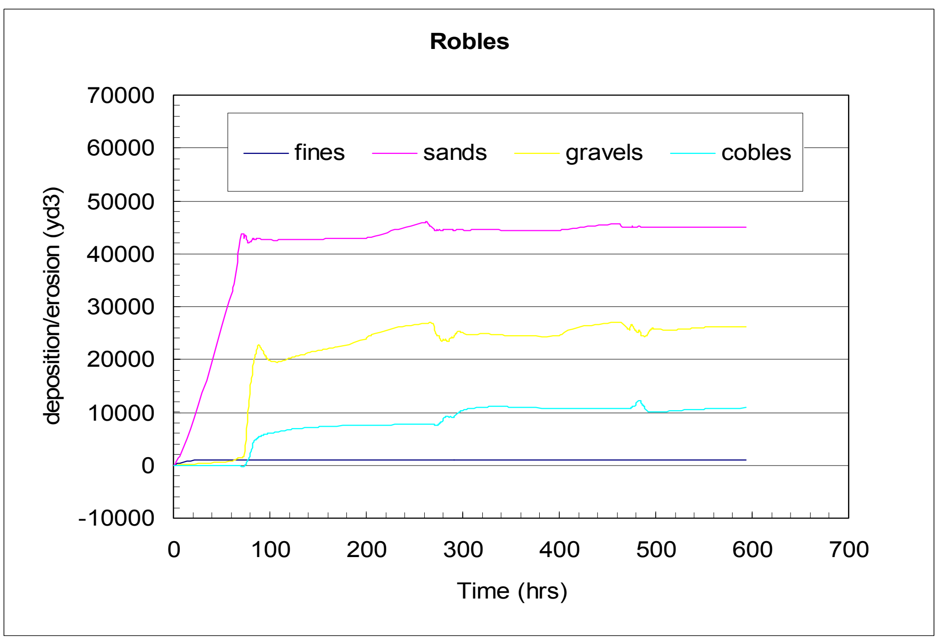


Figure 19.101. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 2a for 100-yr flood. Robles Diversion Dam is at RM 14.15.

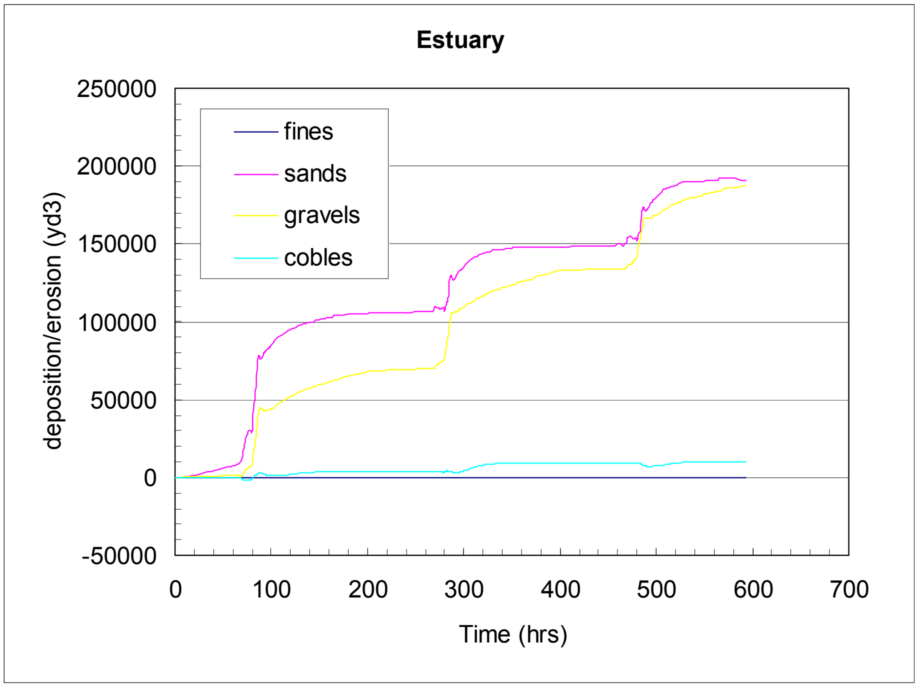


Figure 19.102. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 2a for 100-yr flood.

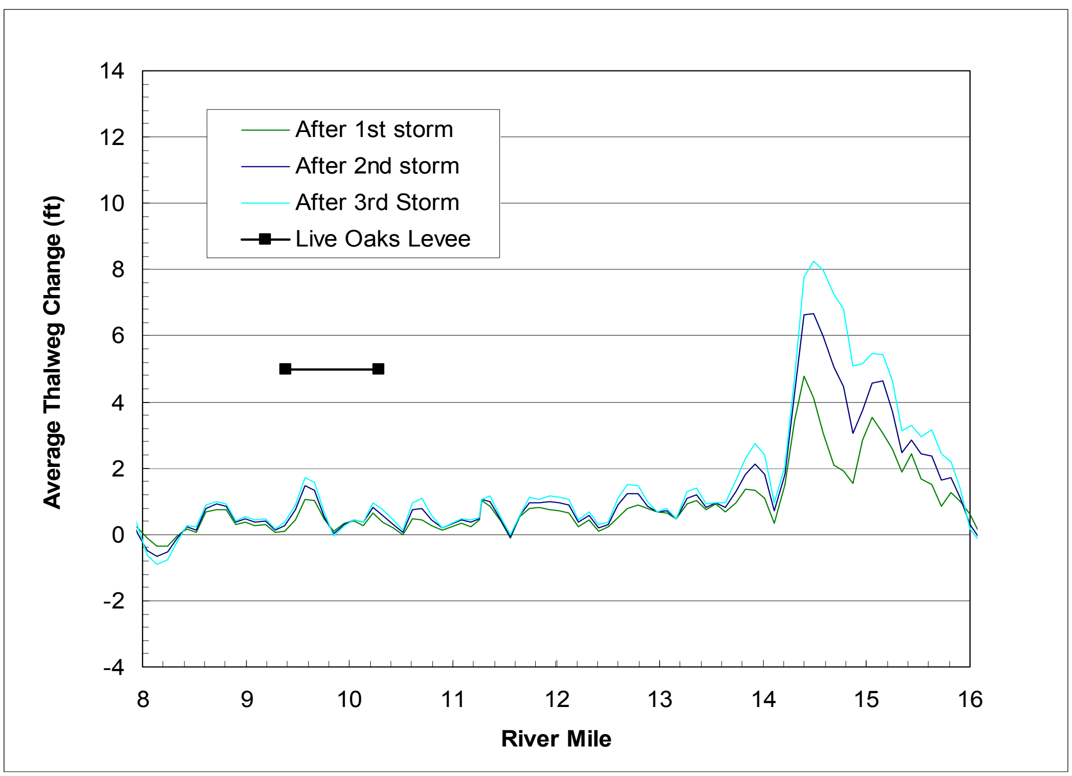


Figure 19.103. Change in thalweg elevation for 100-yr flood in reach 4 at various times from start of simulation of alternative 2a.

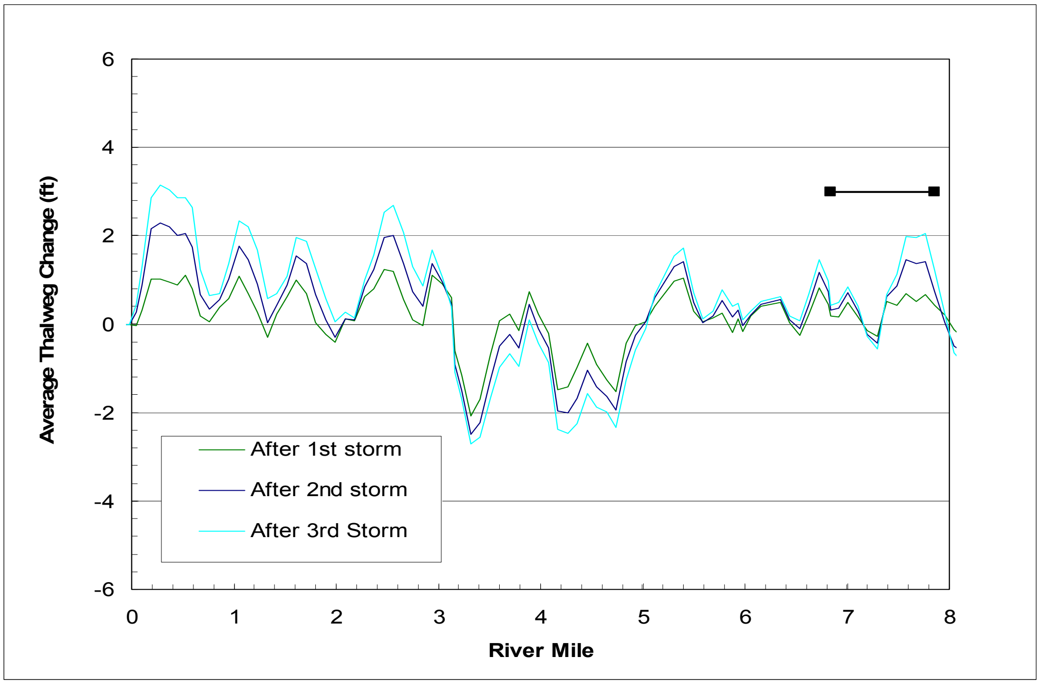


Figure 19.104. Change in thalweg elevation for 100-yr flood in reach 3 at various times from start of simulation of alternative 2a.

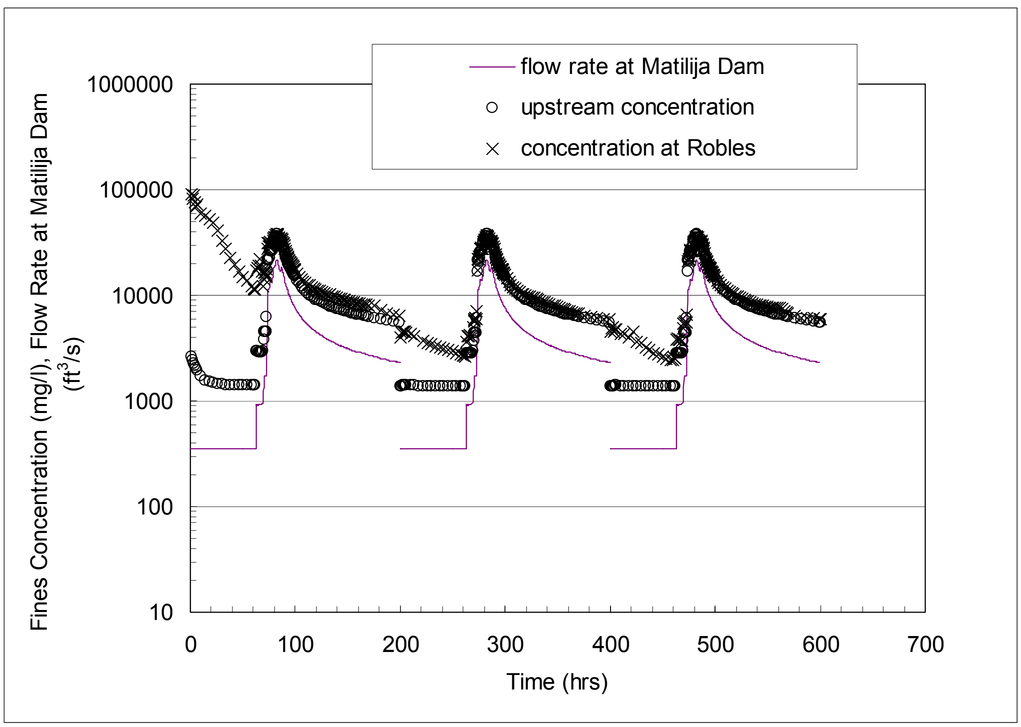


Figure 19.105. Concentrations downstream of dam following removal for 100-yr flood for alternative 2a.

19.3.3. Alternative 2b - Full dam removal/Natural sediment Transport with Natural Transport of Reservoir Area Fines

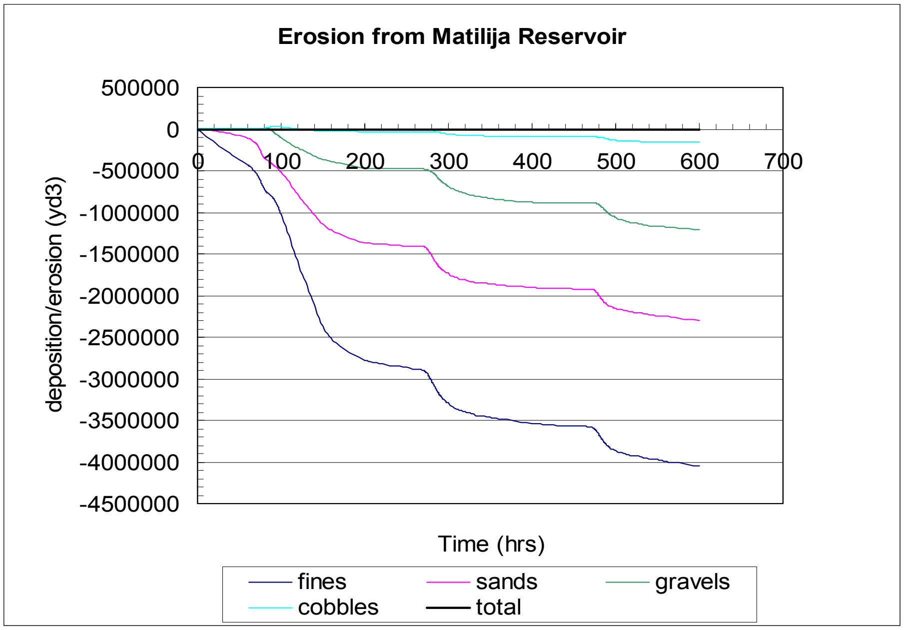


Figure 19.106. Cumulative erosion from the reservoir in alternative 2b for 100-yr flood.

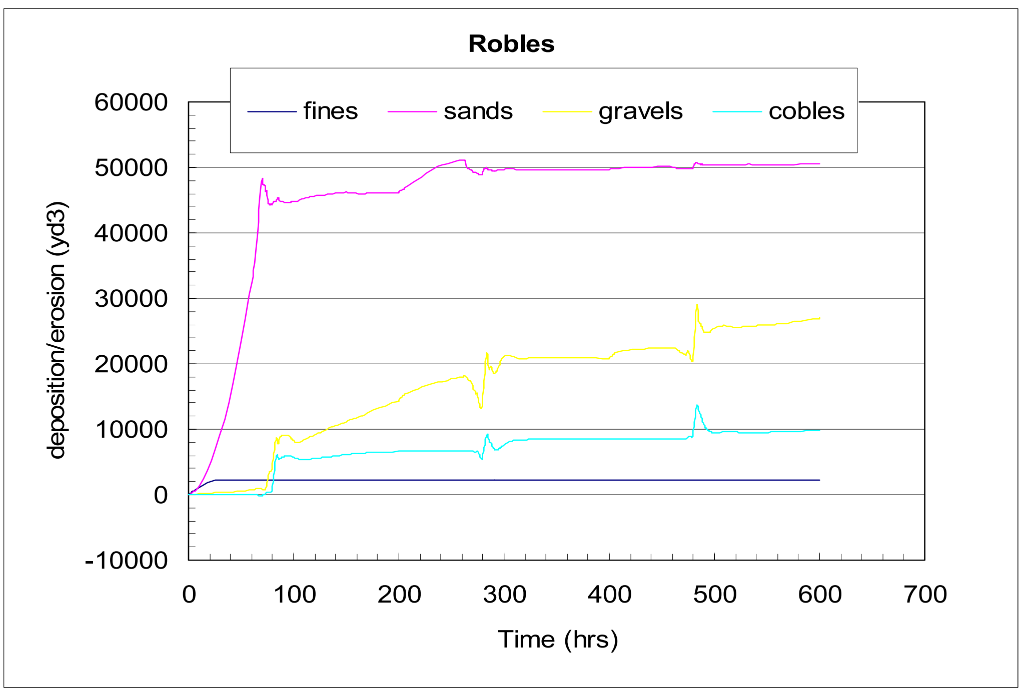


Figure 19.107. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 2b for 100-yr flood. Robles Diversion Dam is at RM 14.15.

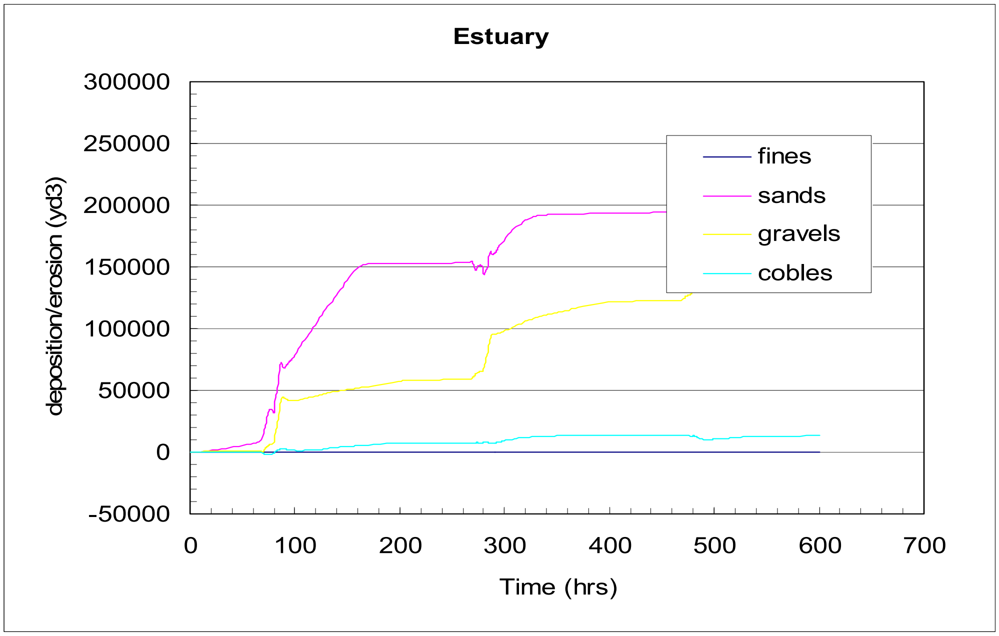


Figure 19.108. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 2b for 100-yr flood.

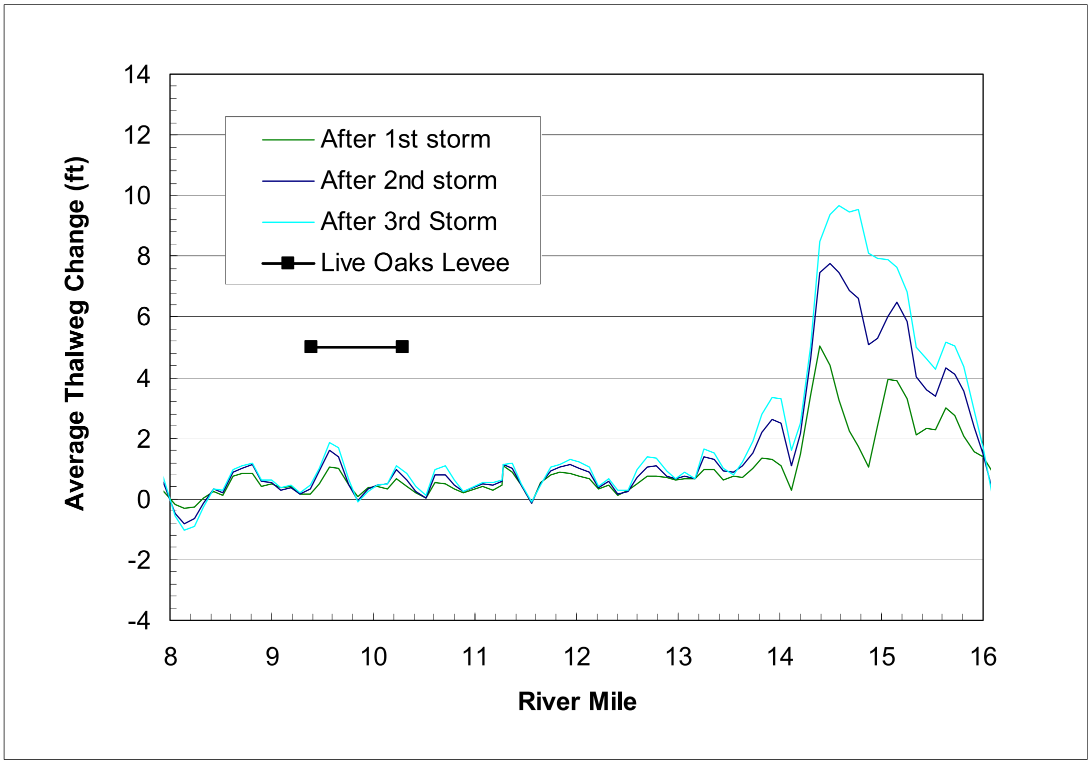


Figure 19.109. Change in thalweg elevation for 100-yr flood in reach 4 at various times from start of simulation of alternative 2b.

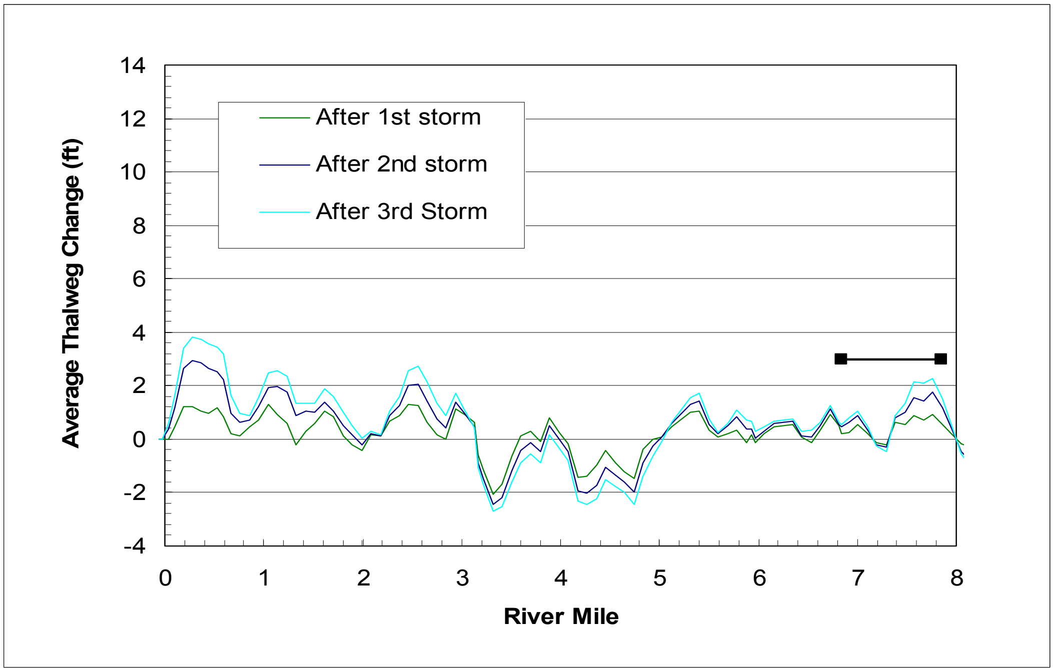


Figure 19.110. Change in thalweg elevation for 100-yr flood in reach 3 at various times from start of simulation of alternative 2b.

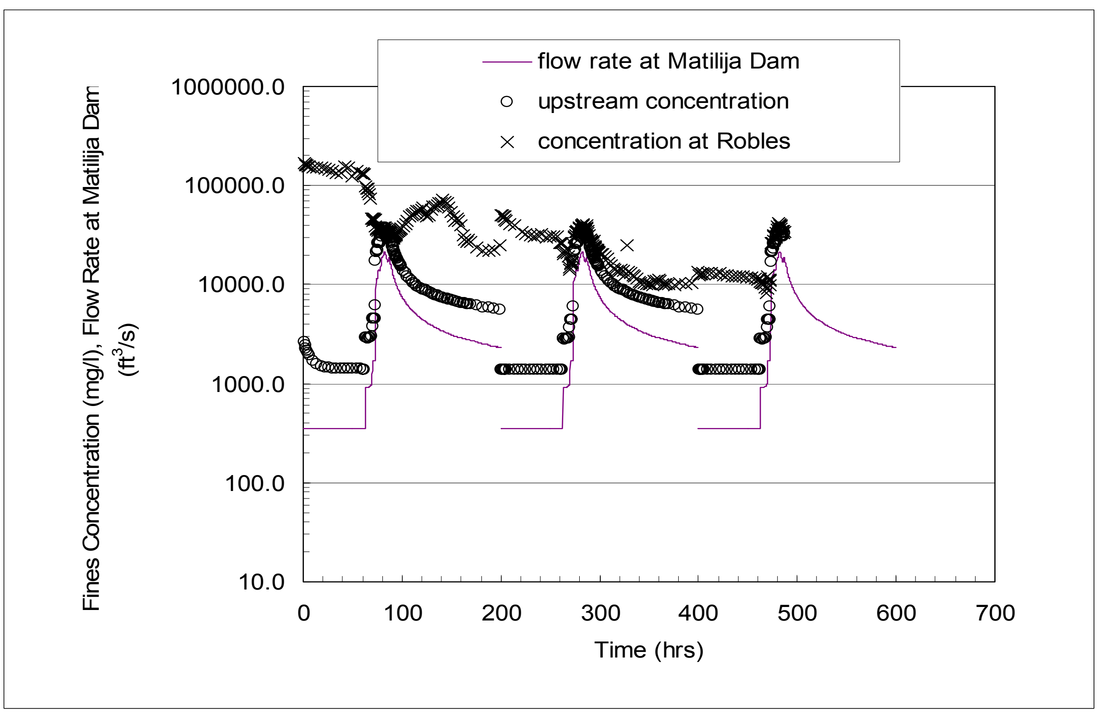


Figure 19.111. Concentrations downstream of dam following removal for 100-yr flood for alternative 2b.

19.3.4. ALTERNATIVE 3A – INCREMENTAL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH REMOVAL OF RESERVOIR AREA FINES OFFSITE

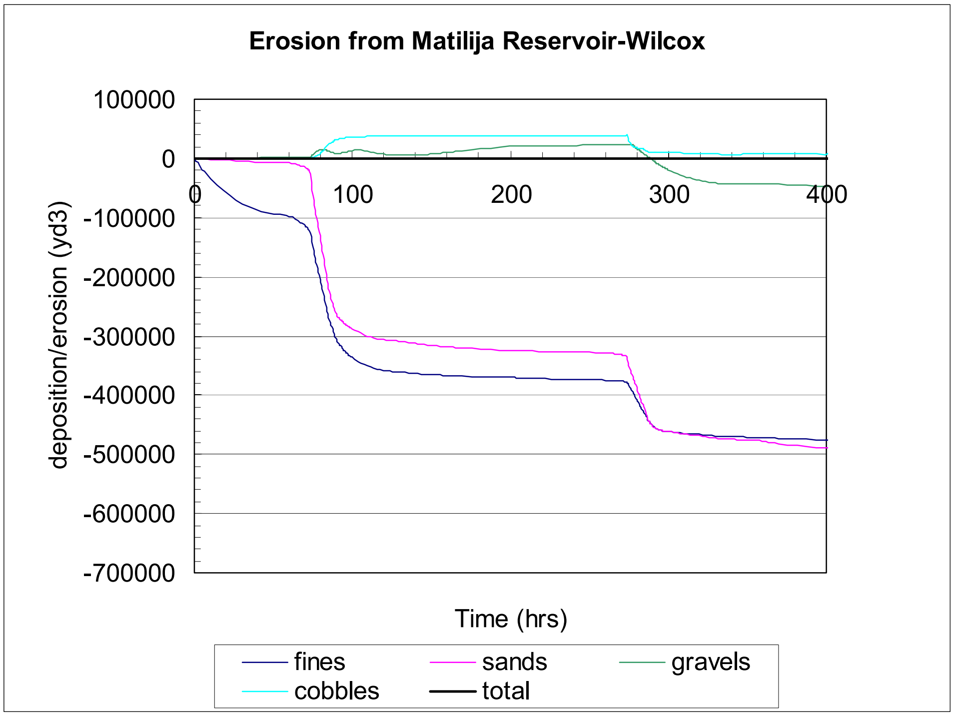


Figure 19.112. Cumulative erosion from the reservoir in alternative 3b for 100-yr flood.

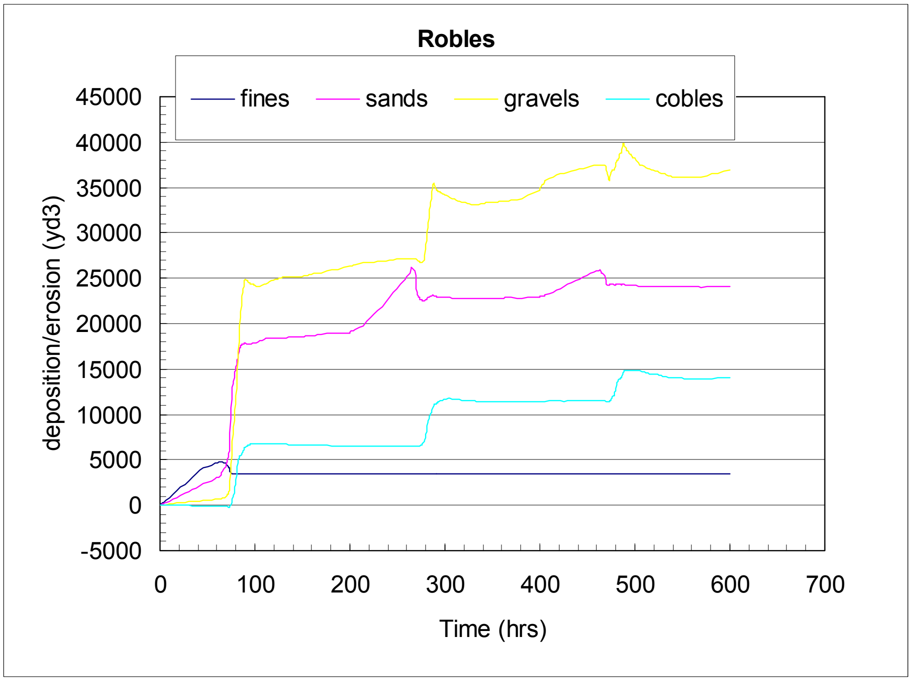


Figure 19.113. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 3b for 100-yr flood. Robles Diversion Dam is at RM 14.15

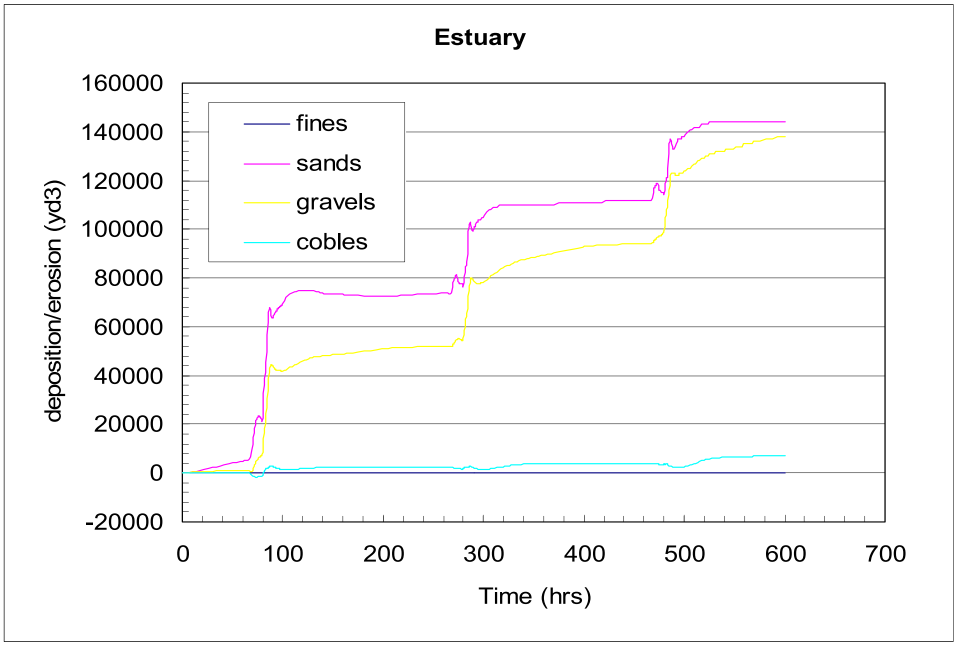


Figure 19.114. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 3a for 100-yr flood.

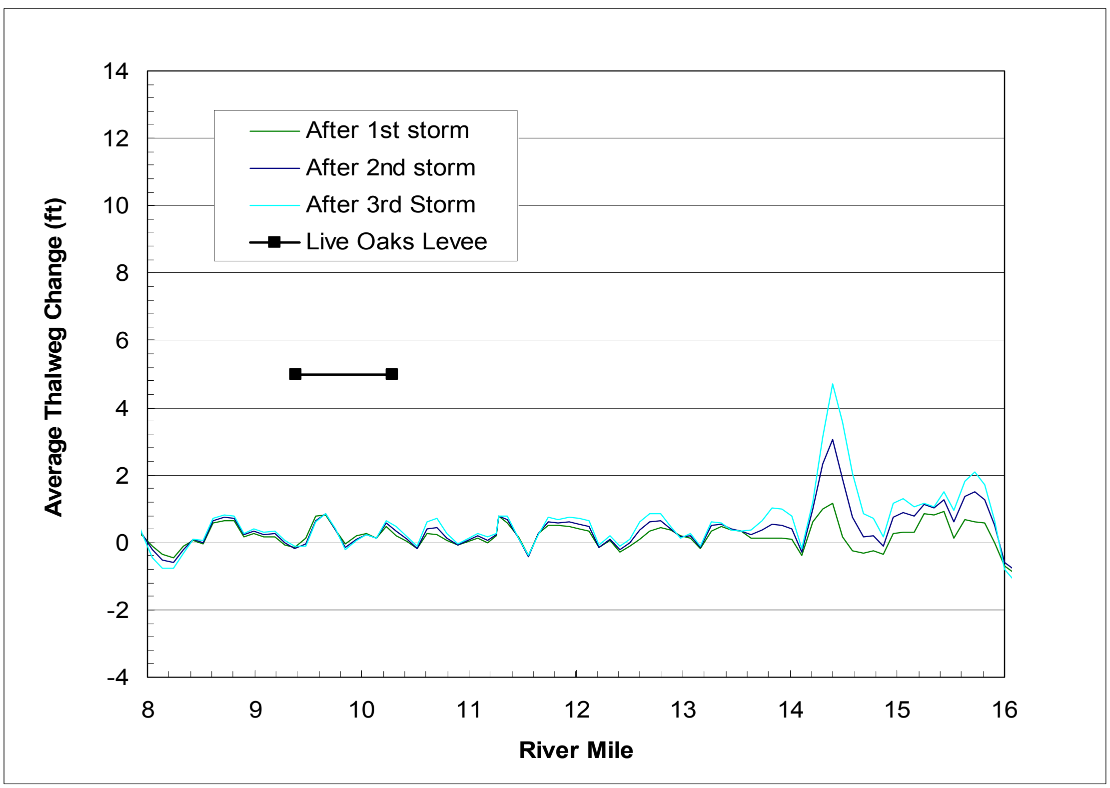


Figure 19.115. Change in thalweg elevation for 100-yr flood in reach 4 at various times from start of simulation of alternative 3a.

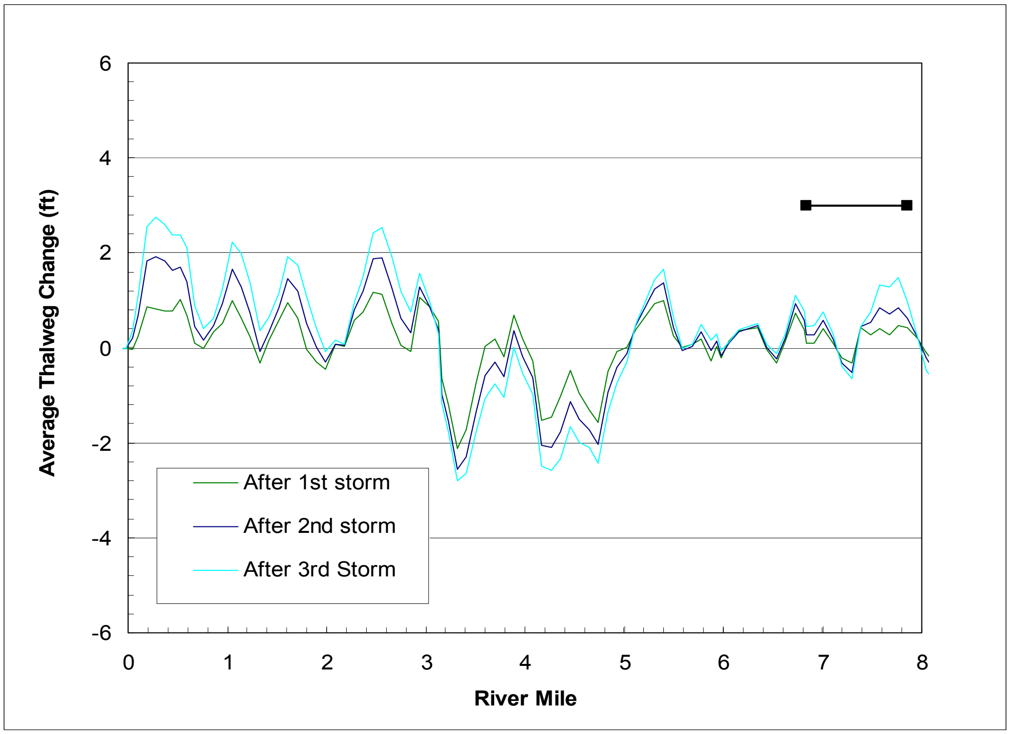


Figure 19.116. Change in thalweg elevation for 100-yr flood in reach 3 at various times from start of simulation of alternative 3a.

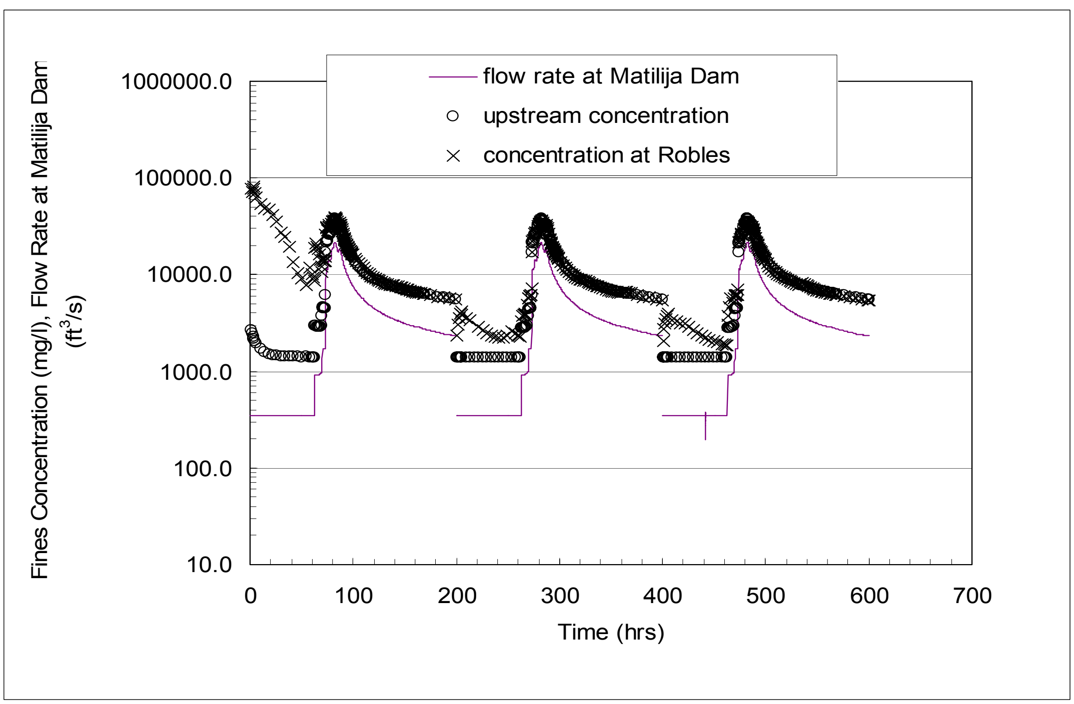


Figure 19.117. Concentrations downstream of dam following removal for 100-yr flood for alternative 3b.

19.3.5. ALTERNATIVE 3B – INCREMENTAL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH NO REMOVAL OF RESERVOIR AREA FINES OFFSITE

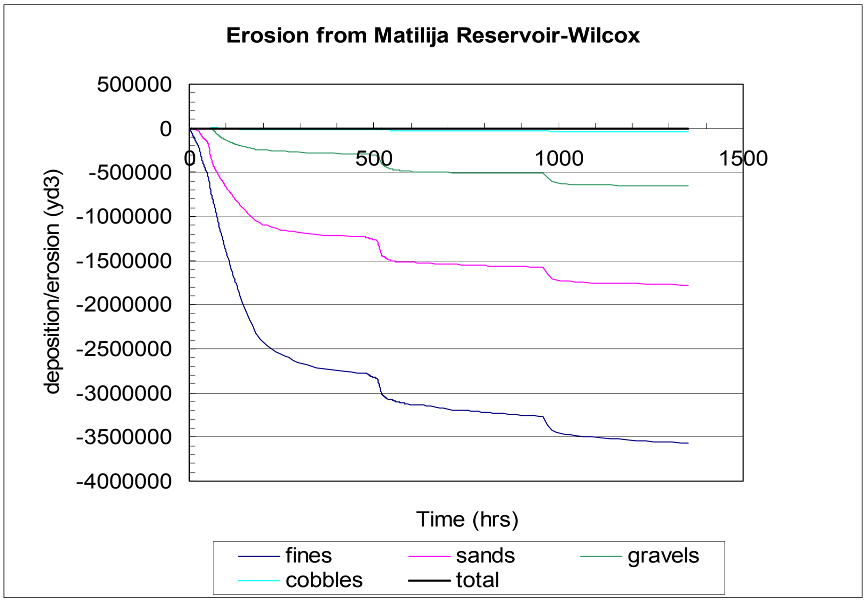


Figure 19.118. Cumulative erosion from the reservoir in alternative 3b for 100-yr flood.

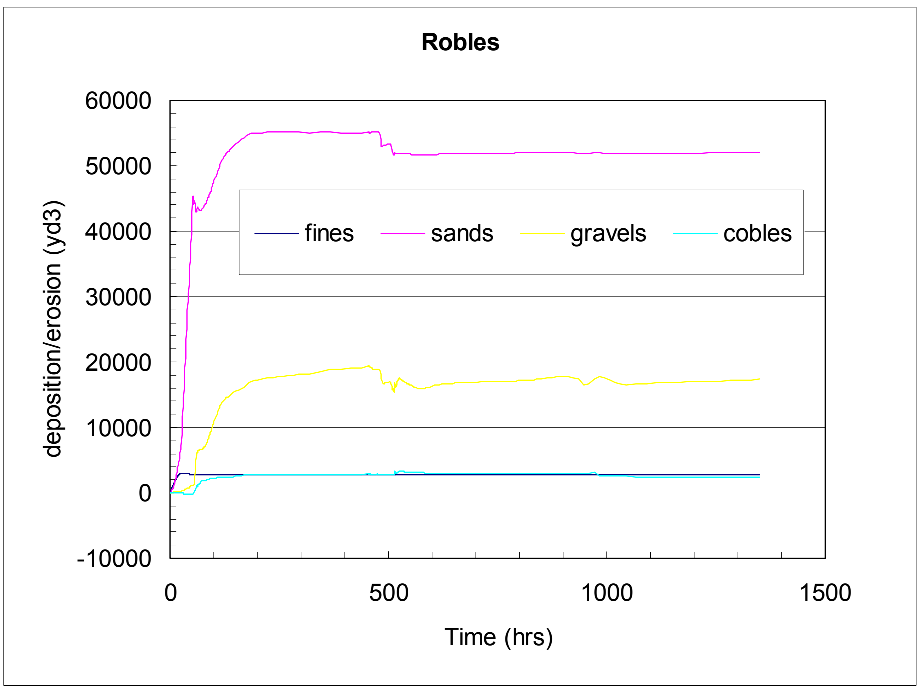


Figure 19.119. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 3b for 100-yr flood. Robles Diversion Dam is at RM 14.15.

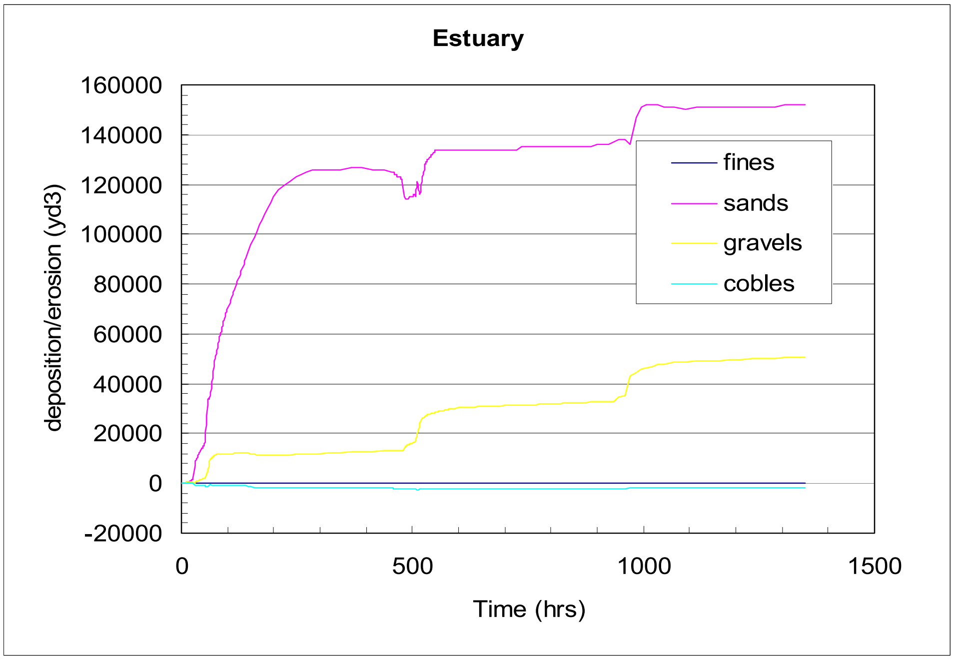


Figure 19.120. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 3b for 100-yr flood.

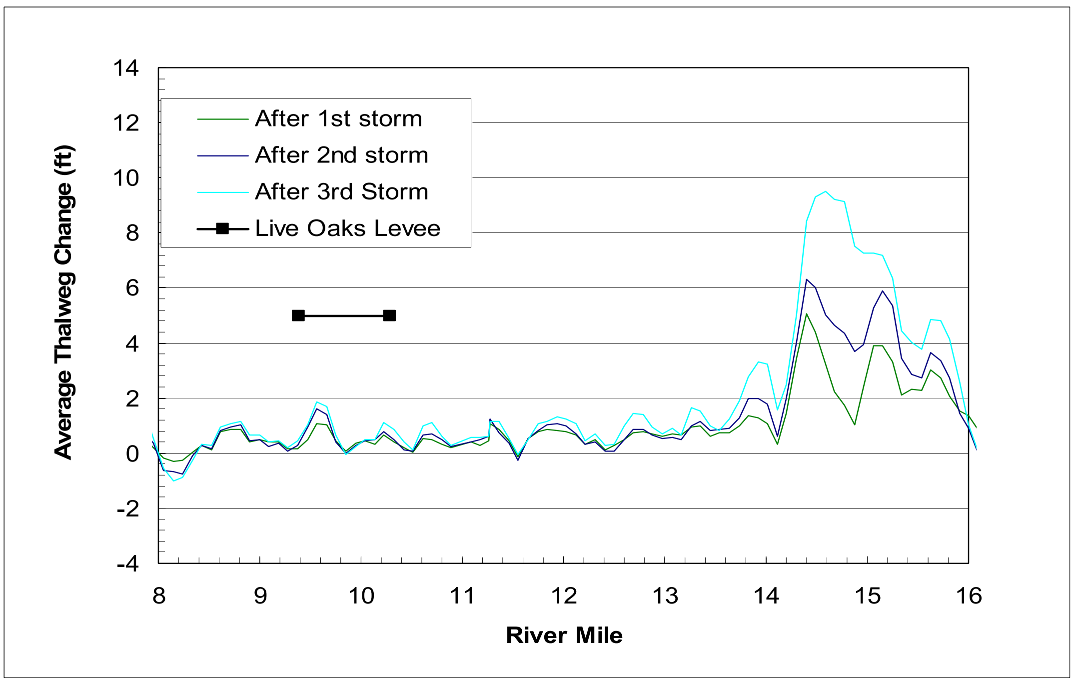


Figure 19.121. Change in thalweg elevation for 100-yr flood in reach 4 at various times from start of simulation of alternative 3b.

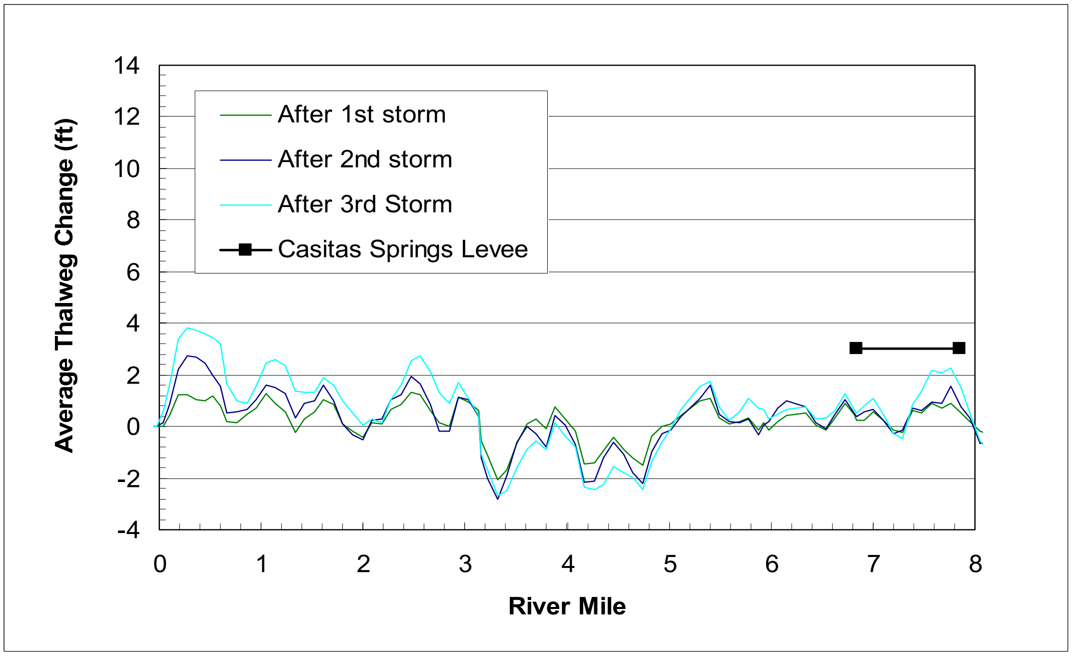


Figure 19.122. Change in thalweg elevation for 100-yr flood in reach 3 at various times from start of simulation of alternative 3b.

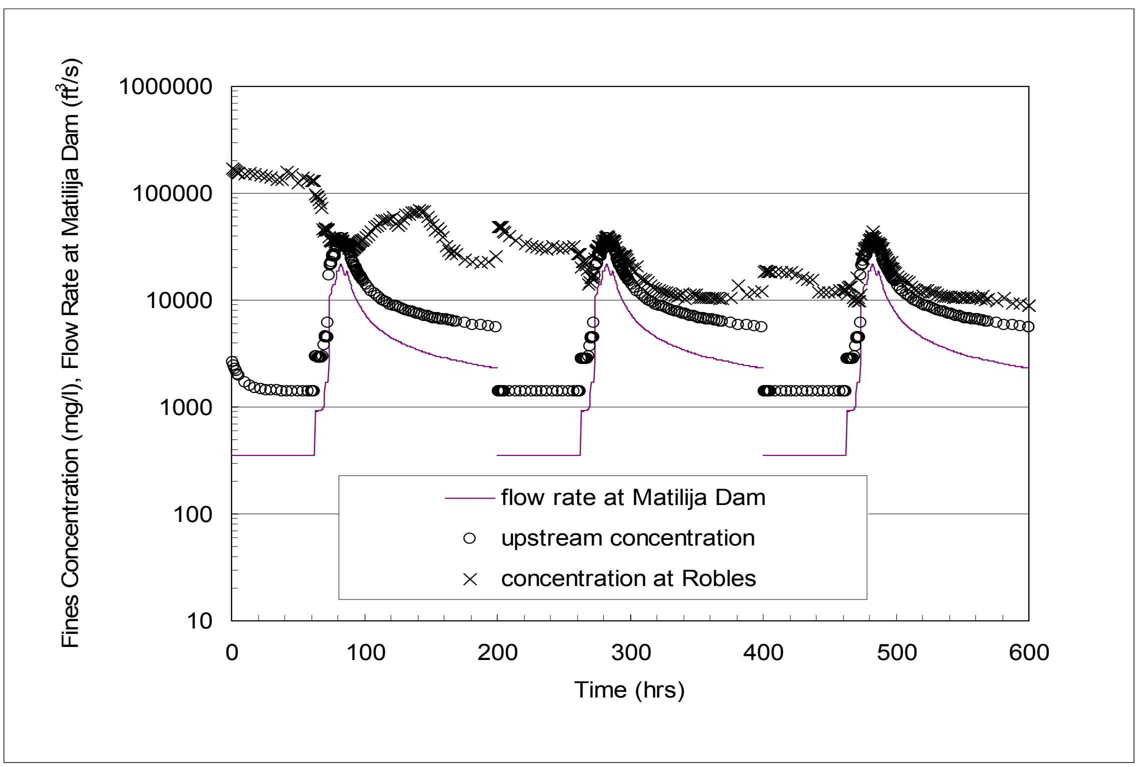


Figure 19.123. Concentrations downstream of dam following removal for 100-yr flood for alternative 3b.

19.3.6. Alternative 4b – Full Dam Removal/Temporarily Stabilize Sediments on Site

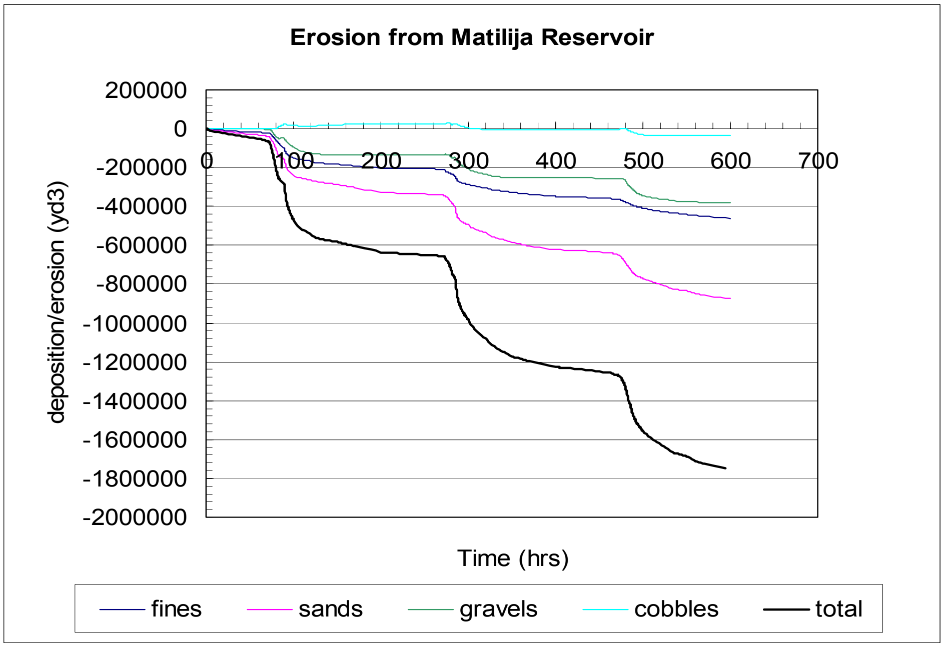


Figure 19.124. Cumulative erosion from the reservoir in alternative 4b for 100-yr flood.

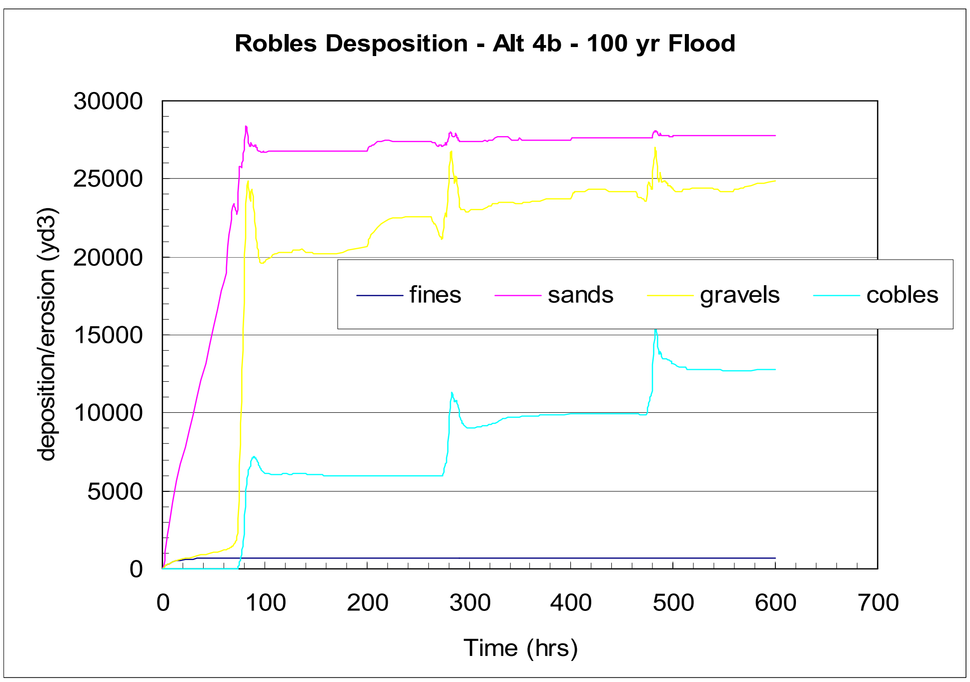


Figure 19.125. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 4b for 100-yr flood. Robles Diversion Dam is at RM 14.15.

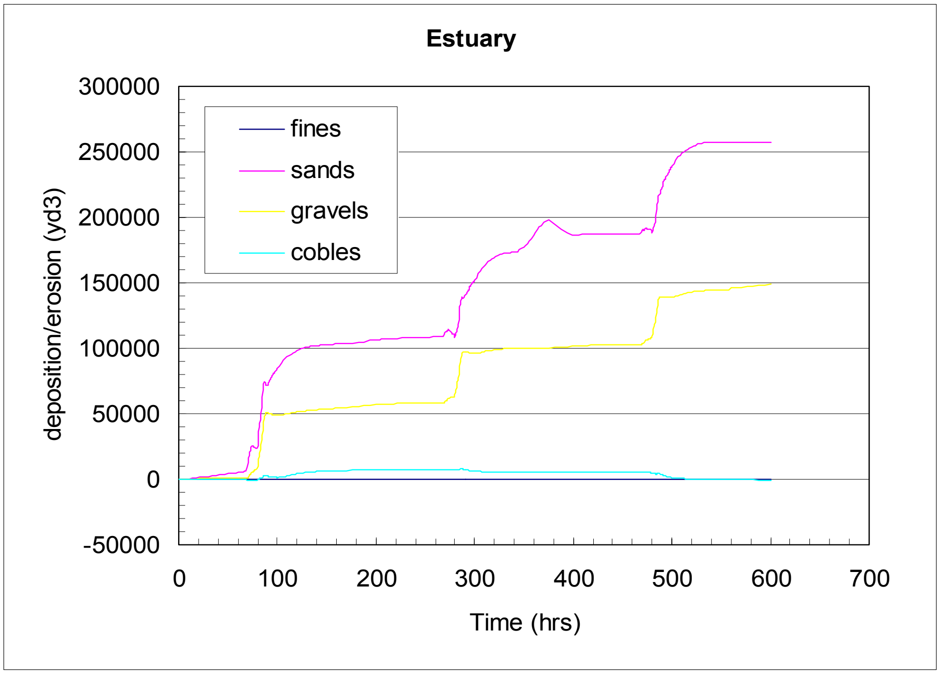


Figure 19.126. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 4b for 100-yr flood.

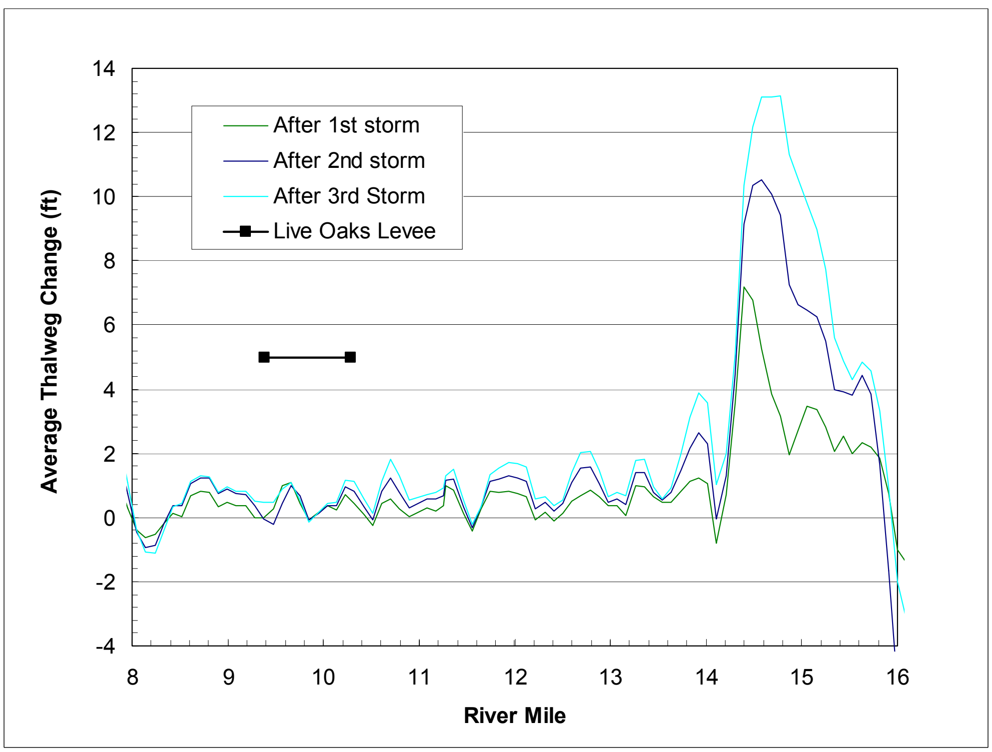


Figure 19.127. Change in thalweg elevation for 100-yr flood in reach 4 at various times from start of simulation of alternative 4b.

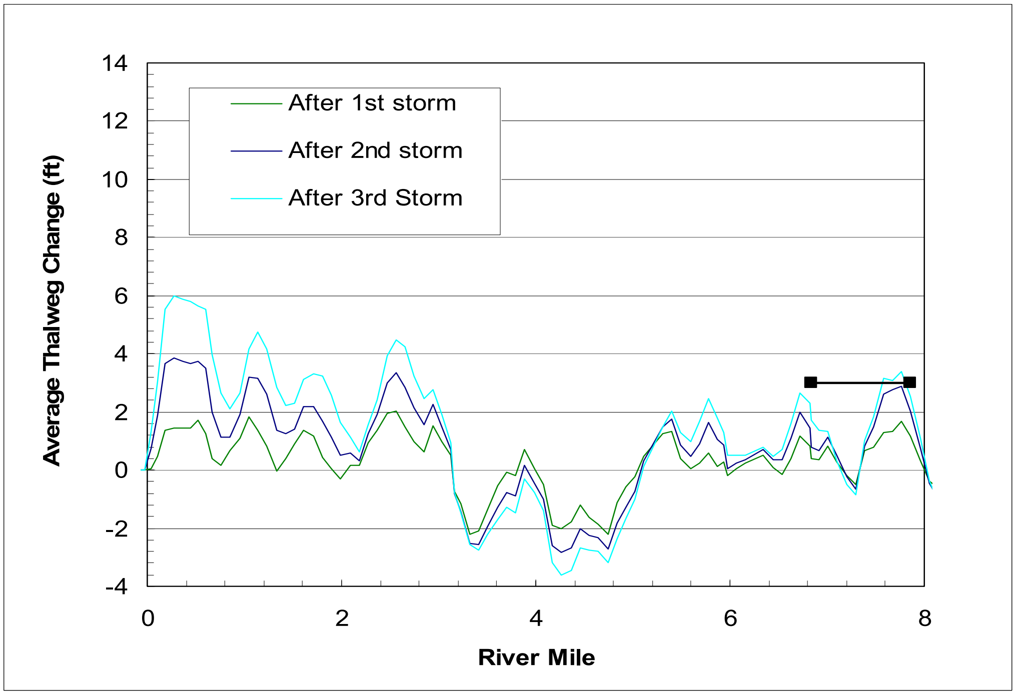


Figure 19.128. Change in thalweg elevation for 100-yr flood in reach 3 at various times from start of simulation of alternative 4b.

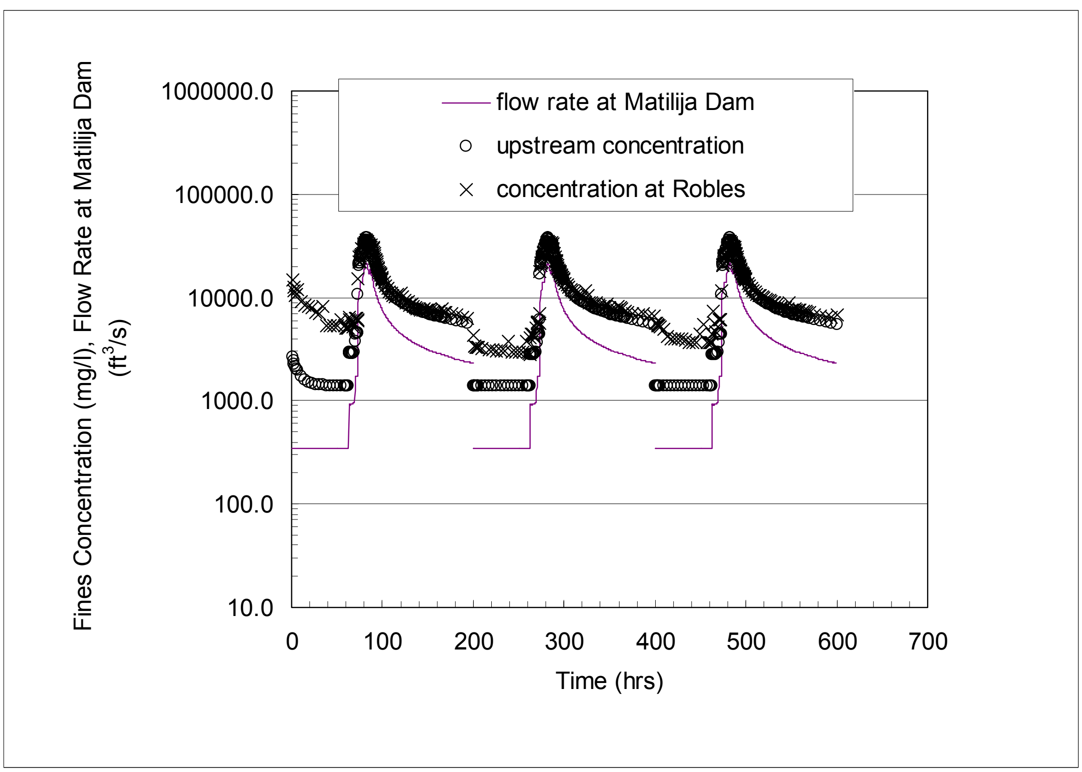


Figure 19.129. Concentrations downstream of dam following removal for 100-yr flood for alternative 4b.

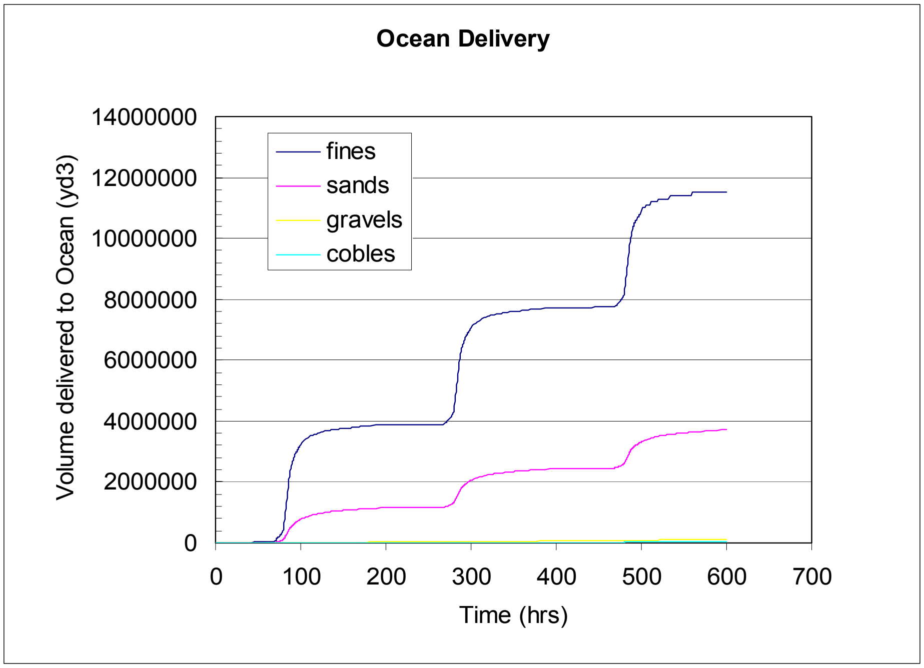


Figure 19.130. Sediment Delivery to ocean for 100-yr flood for alternative 4b.

19.3.7. ALTERNATIVES 1 AND 4A – COMPLETE REMOVAL OF DAM AND RESERVOIR SEDIMENTS FROM RIVER SYSTEM

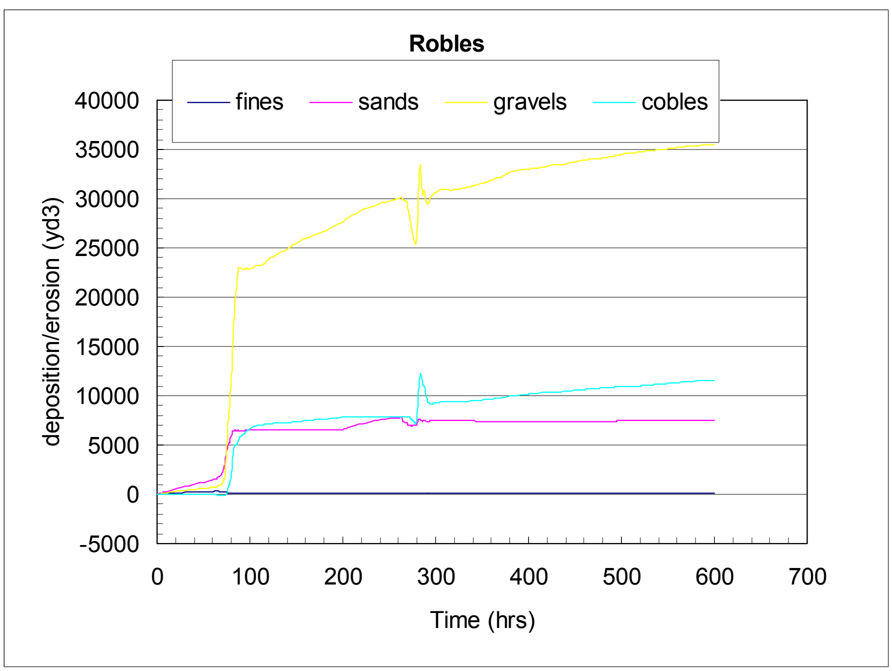


Figure 19.131. Cumulative deposition from RM 14.15 to RM 14.5 in alternative 1 and 4a for 100-yr flood. Robles Diversion Dam is at RM 14.15.

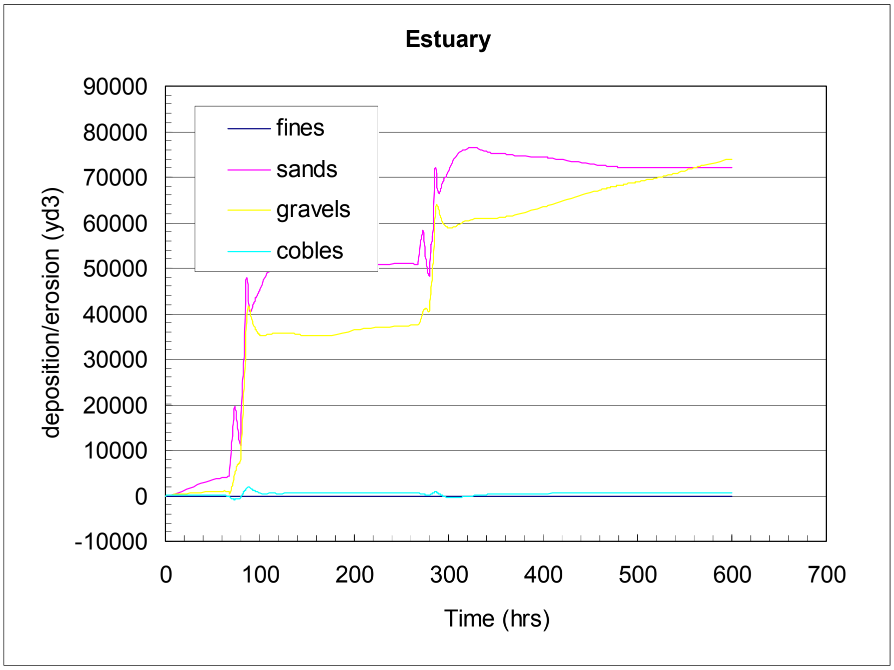


Figure 19.132. Cumulative deposition in Estuary (RM 0.6 - 0.2) in alternative 1 and 4a for 100-yr flood.

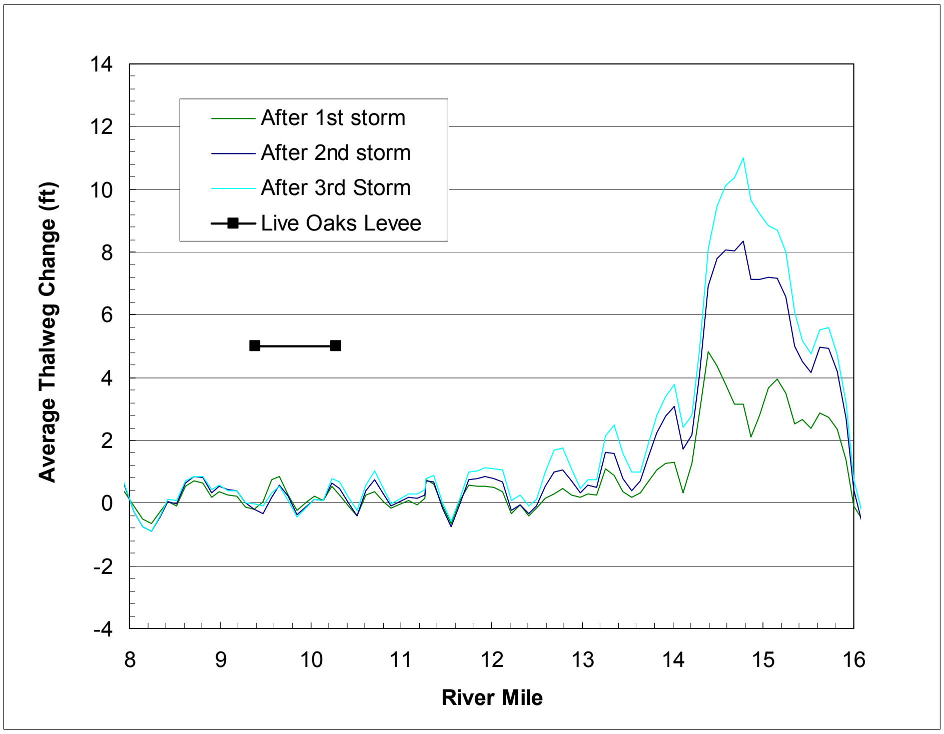


Figure 19.133. Change in thalweg elevation for 100-yr flood in reach 4 at various times from start of simulation of alternative 1 and 4a.

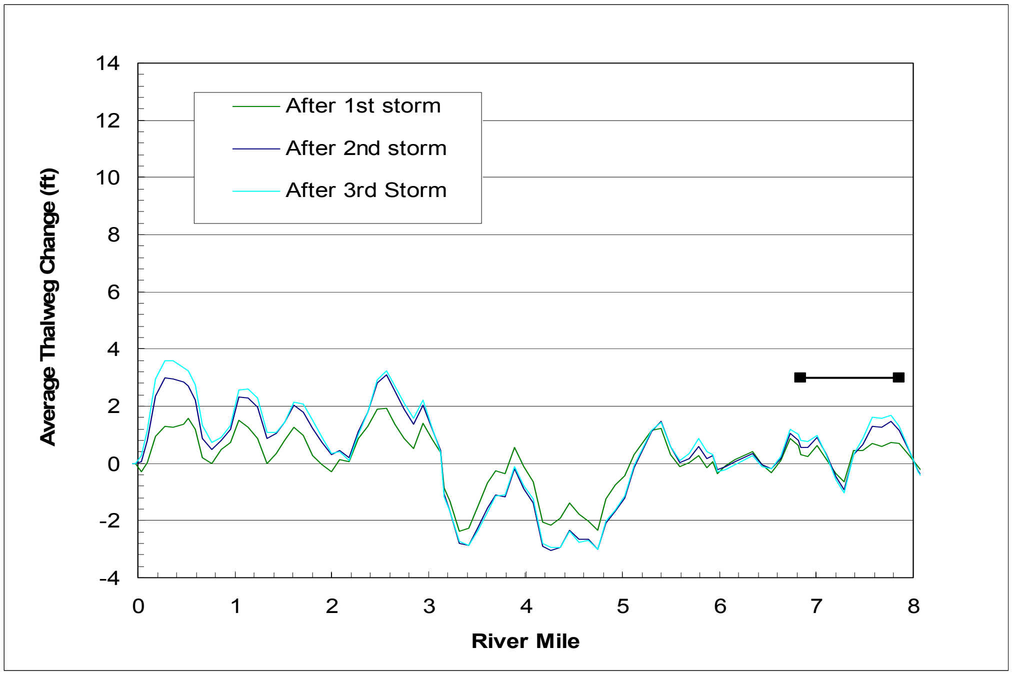


Figure 19.134. Change in thalweg elevation for 100-yr flood in reach 3 at various times from start of simulation of alternative 1 and 4a.

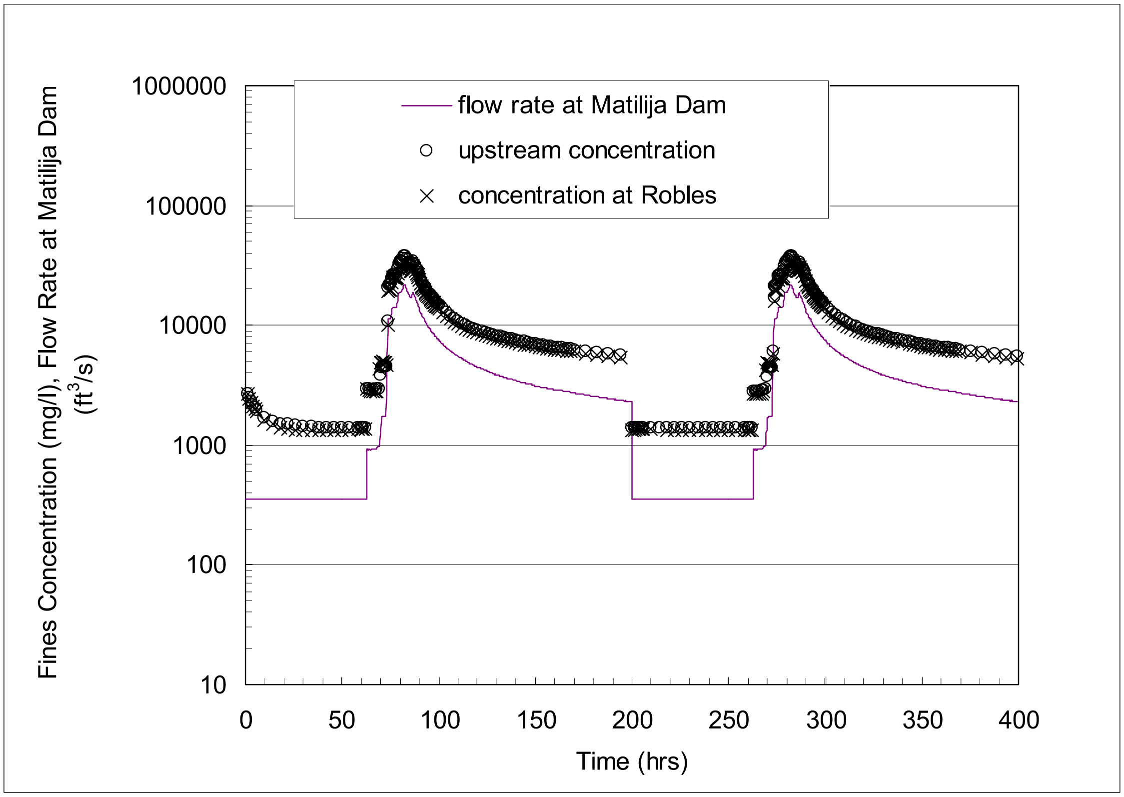


Figure 19.135. Concentrations following removal for 100-yr flood for alternative 1 and 4a.

19.4. Model Results for 50-yr simulation

19.4.1. No Action Alternative

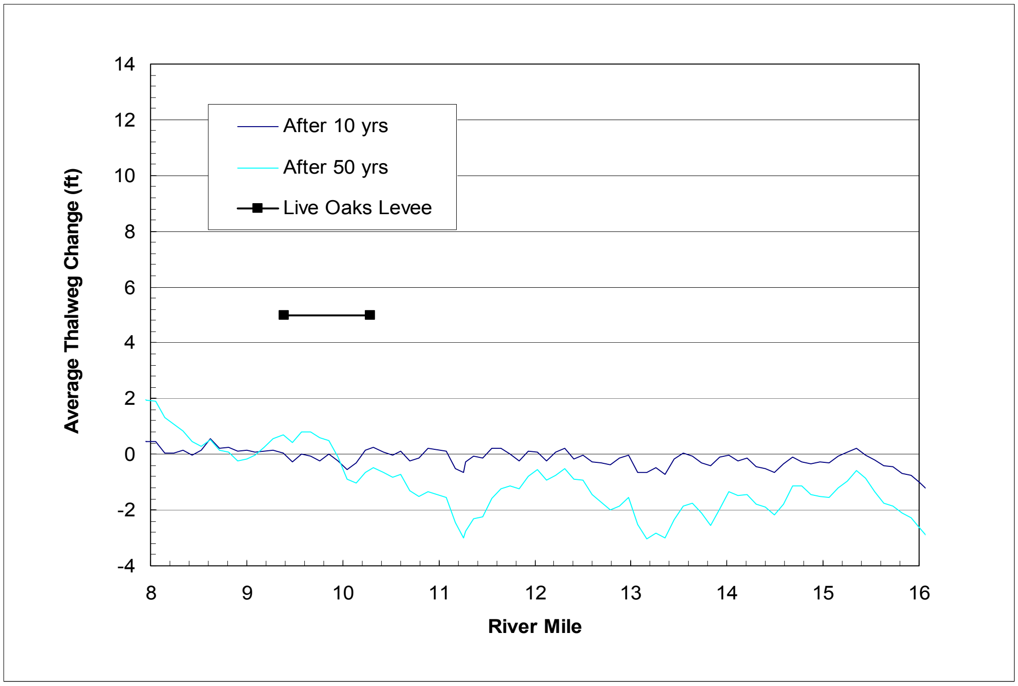


Figure 19.136. Thalweg elevation change for No Action alternative for 50-yr simulation.

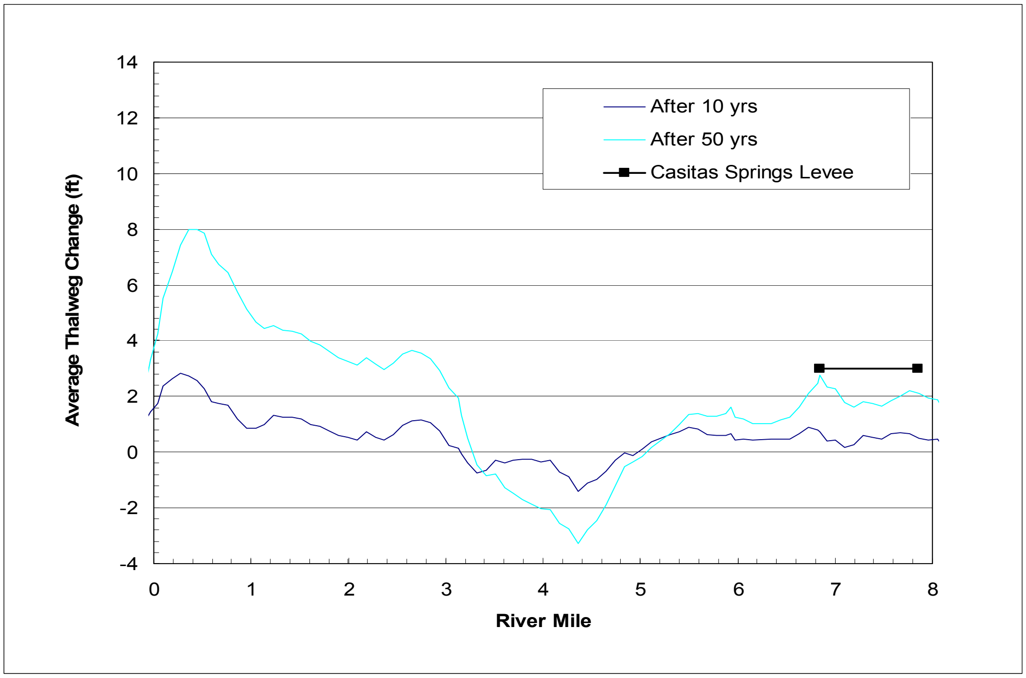


Figure 19.137. Thalweg elevation change for No Action alternative for 50-yr simulation.

19.4.2. ALTERNATIVE 2A - FULL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH REMOVAL OF RESERVOIR AREA FINES OFFSITE

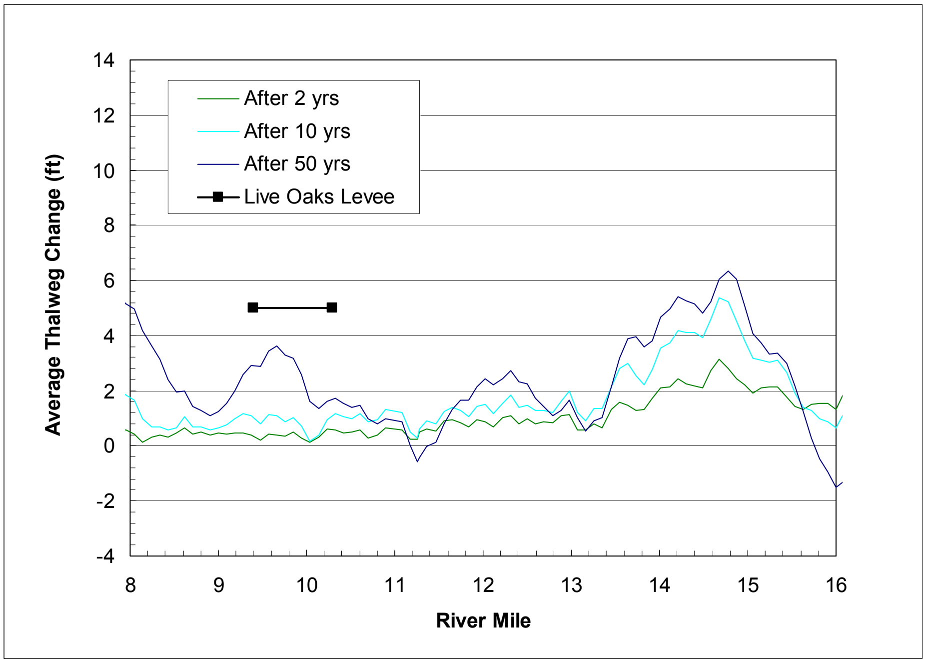


Figure 19.138. Thalweg elevation change for alternative 2a 50-yr simulation.

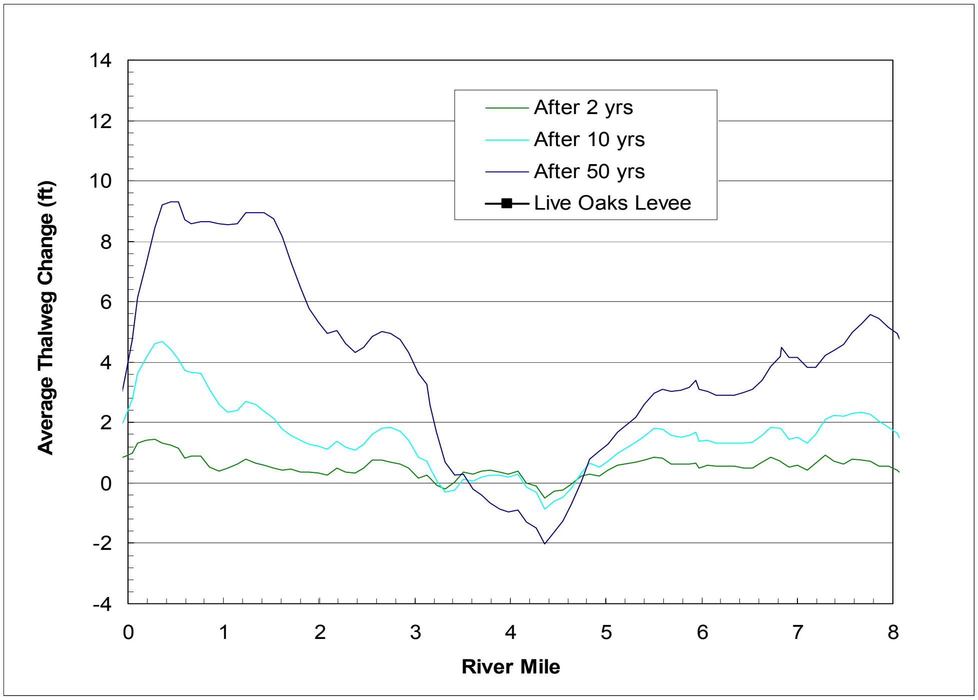


Figure 19.139. Thalweg elevation change for alternative 2a for 50-yr simulation.

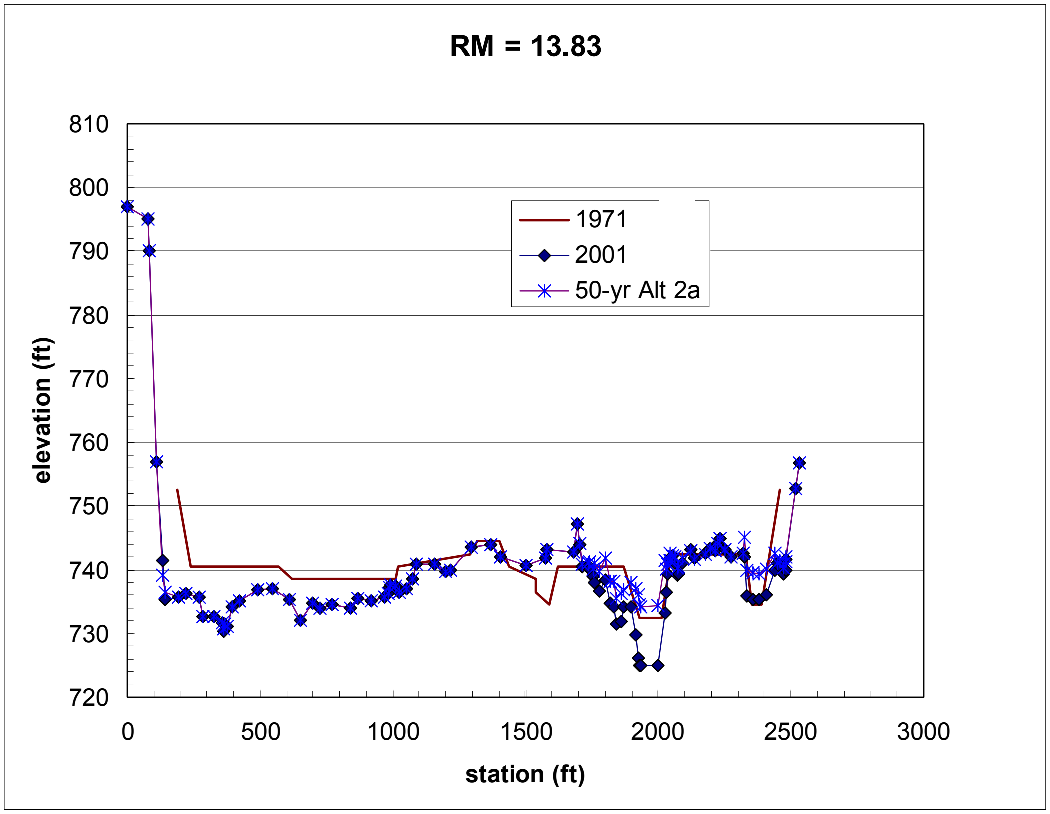


Figure 19.140. Change in cross section at RM 13.82 for alternative 2a for the 50-yr period

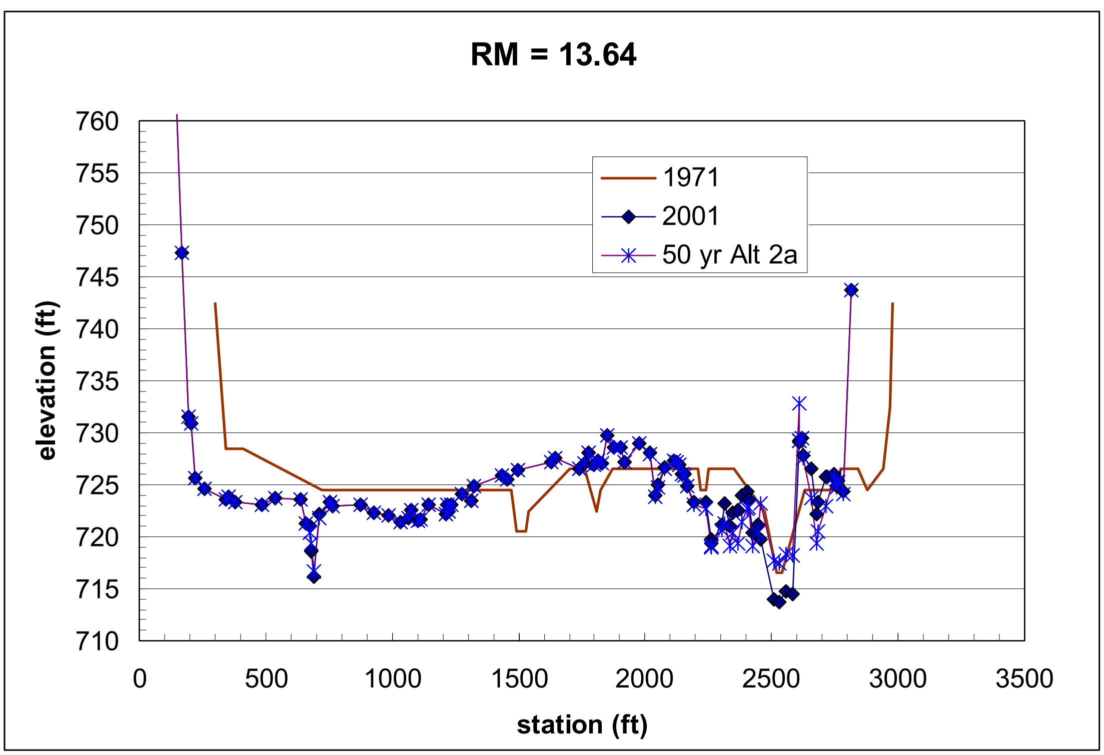


Figure 19.141. Change in cross section at RM 13.63 for alternative 2a for the 50-yr period.

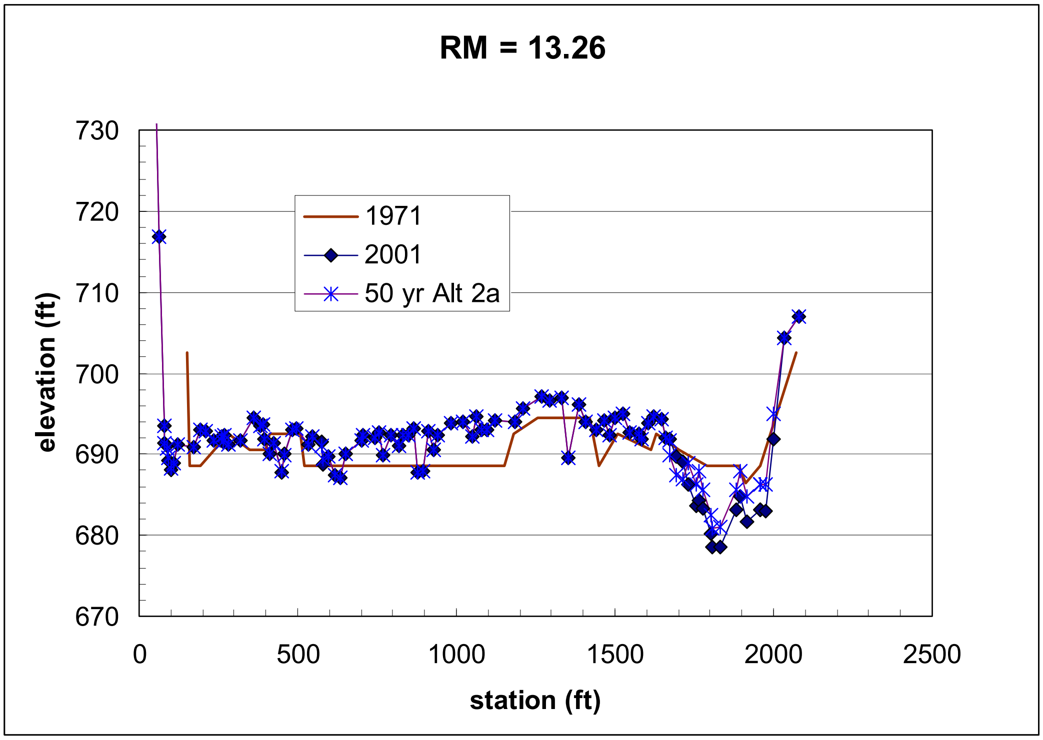


Figure 19.142. Change in cross section at RM 13.26 for alternative 2a for the 50-yr period.

19.4.3. ALTERNATIVE 2B - FULL DAM REMOVAL/NATURAL SEDIMENT TRANSPORT WITH NATURAL TRANSPORT OF RESERVOIR AREA FINE

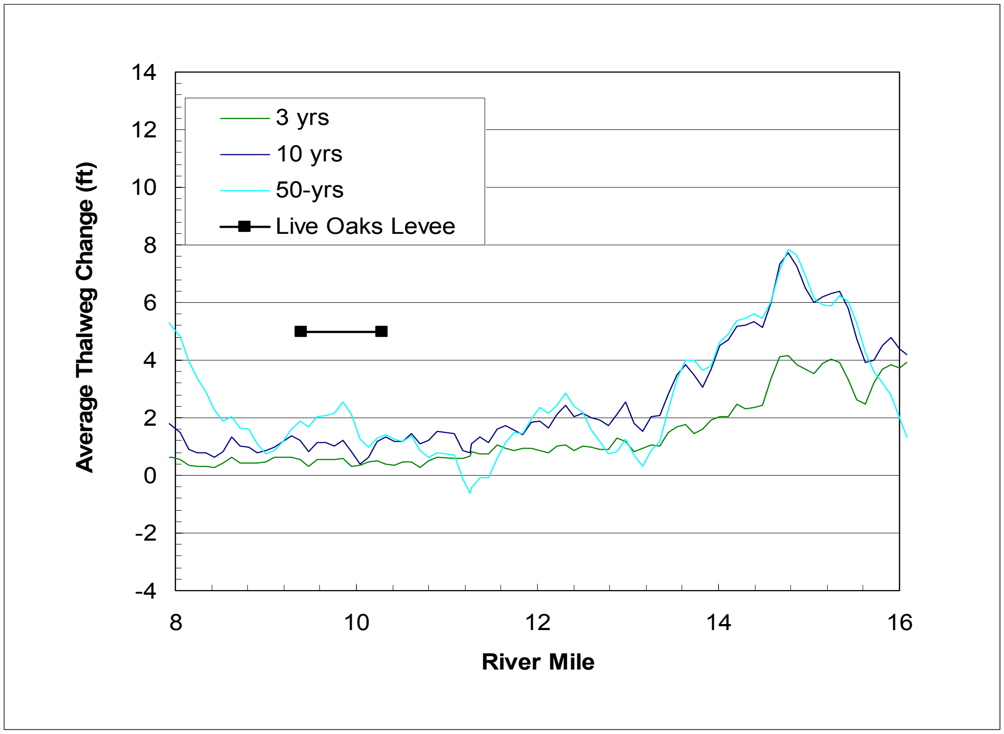


Figure 19.143. Thalweg elevation change for alternative 2b 50-yr simulation.

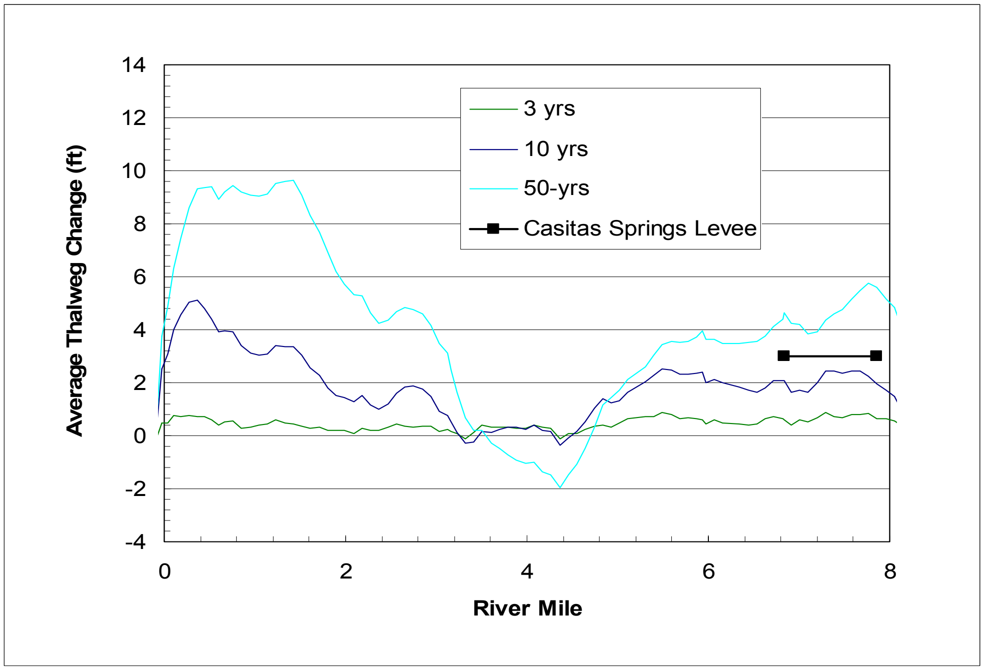


Figure 19.144. Thalweg elevation change for alternative 2b for 50-yr simulation.

19.4.4. ALTERNATIVES 1 AND 4A – COMPLETE REMOVAL OF DAM AND RESERVOIR SEDIMENTS FROM RIVER SYSTEM

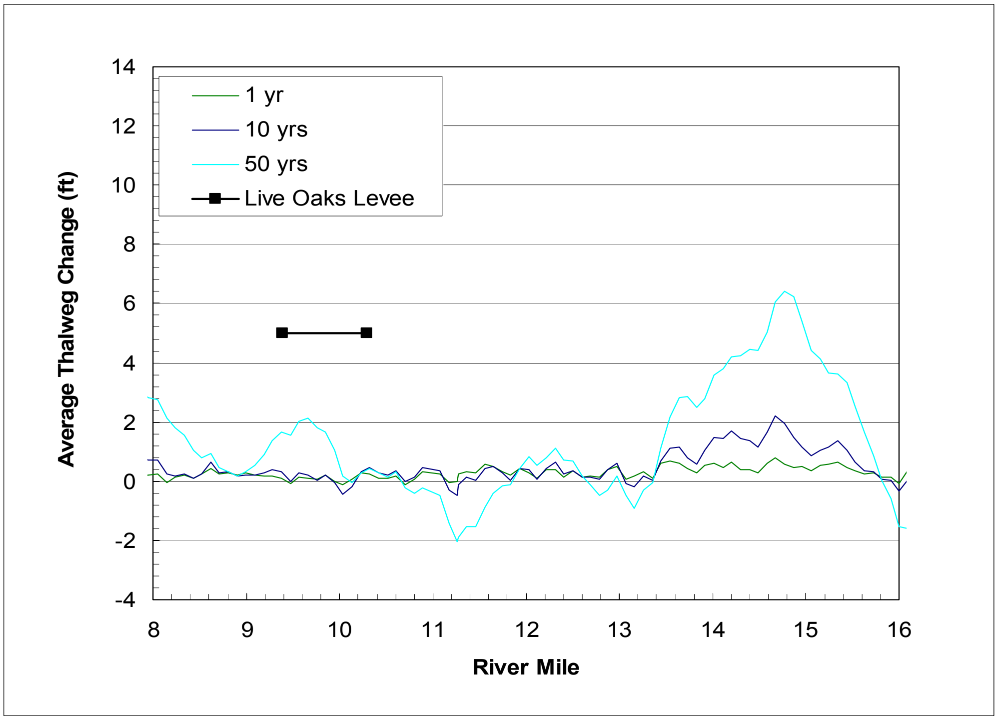


Figure 19.145. Thalweg elevation change for alternatives 1 and 4a for 50-yr simulation.

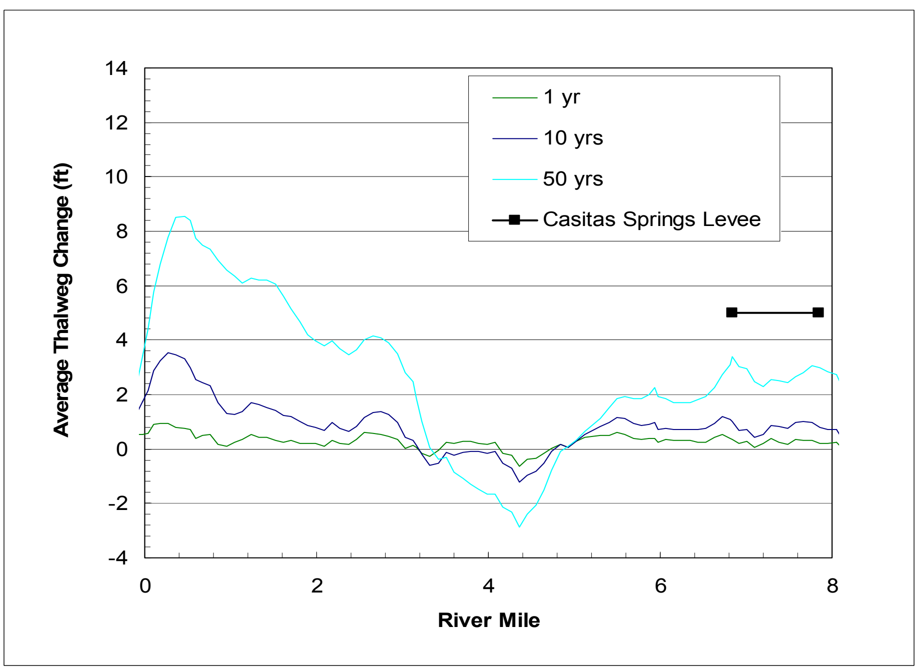


Figure 19.146. Thalweg elevation change for alternatives 1 and 4a for 50-yr simulation.

19.4.5. Alternative 4b – Full Dam Removal/Temporarily Stabilize Sediments on Site

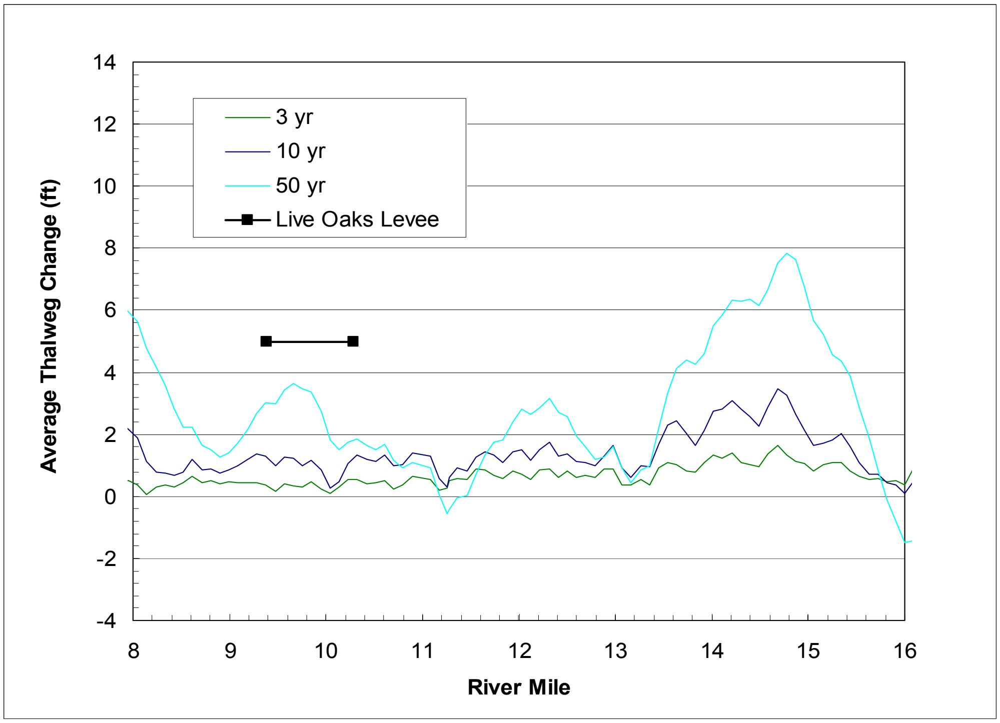


Figure 19.147. Thalweg elevation change for alternative 4b for 50-yr simulation.

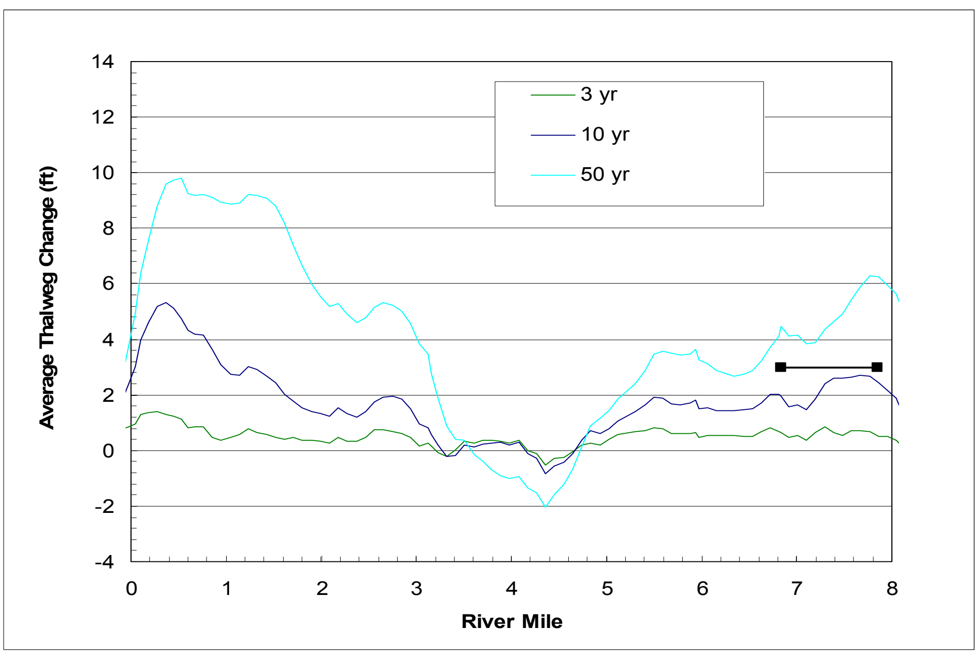


Figure 19.148. Thalweg elevation change for alternatives 4b for 50-yr simulation.

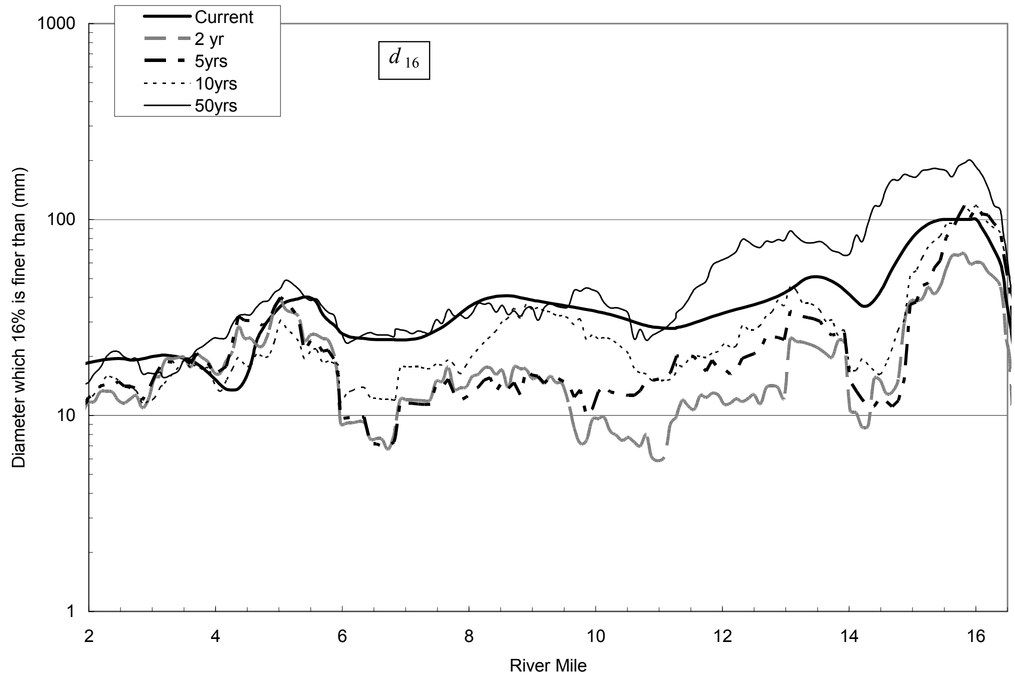


Figure 19.149. Change in d_{16} as a function of RM and time after dam removal for 50-yr simulation.

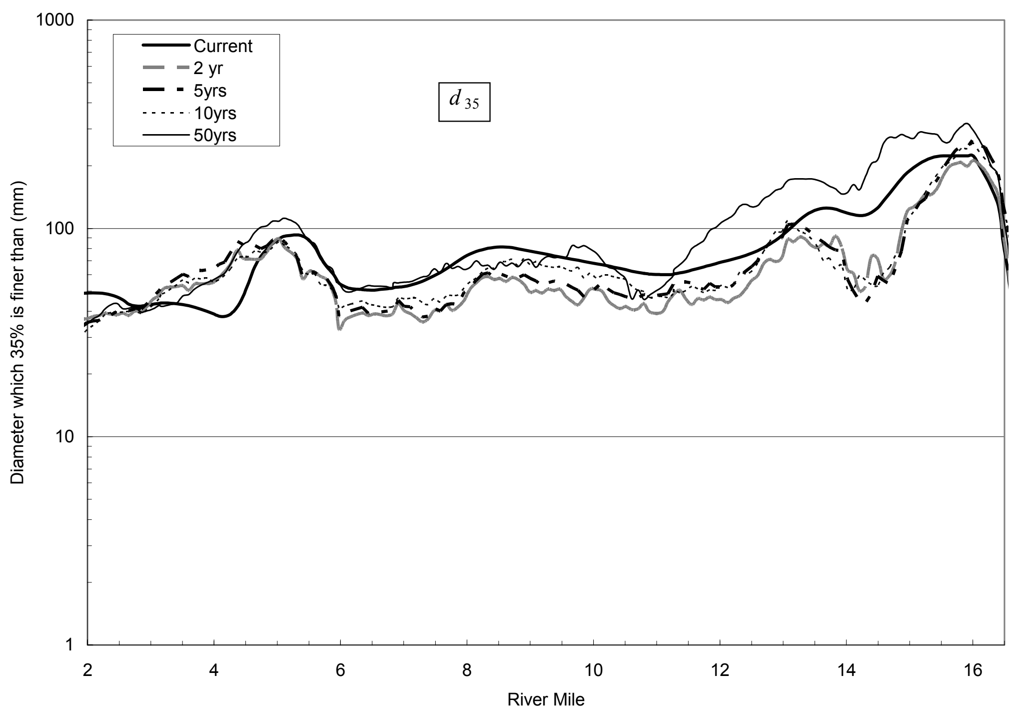


Figure 19.150. Change in d_{35} as a function of RM and time after dam removal for 50-yr simulation.

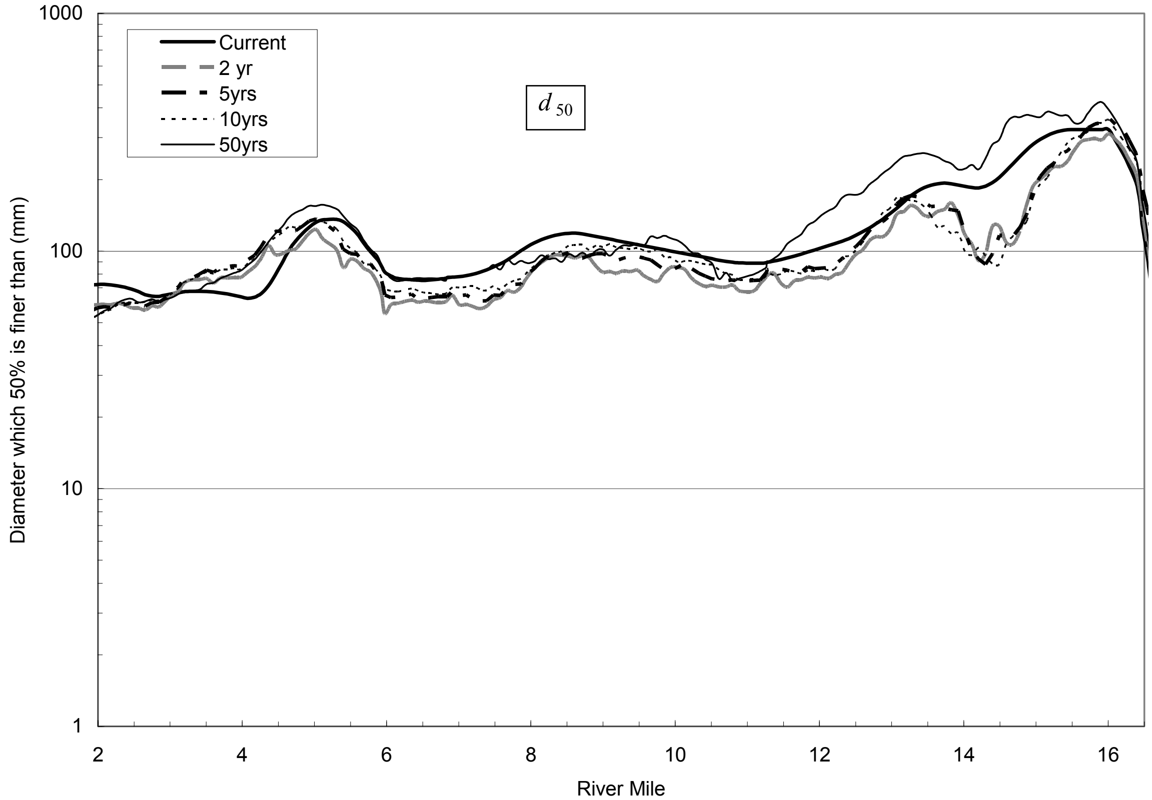


Figure 19.151. Change in d_{50} as a function of RM and time after dam removal for 50-yr simulation.

20. Exhibit H. Sediment delivery to ocean

Table 20.1. Delivery of sediment to the ocean for the Natural Transport Alternative – with removal of fines (Alternative 2a, 3a). The sediment delivery for Alternative 4b would be similar to Alternative 2a and 3a after 20 years. However, before that time, it is dependent upon the revetment height and upon the time at which the revetment is removed.

		Delivery to ocean		
Year	**fines**	**sands**	**gravels**	**cobles**
0	0	0	0	0
0.11	1,744,000	16,915	1,226	68
1.90	2,790,000	296,879	21,517	1,199
1.97	3,100,000	308,696	22,373	1,246
2.06	3,460,000	321,739	23,318	1,299
3.87	3,600,000	605,539	43,887	2,445
3.94	4,780,000	617,278	44,738	2,493
4.03	4,800,000	630,322	45,683	2,545
4.10	5,160,000	642,061	46,534	2,593
5.00	5,170,000	782,514	56,713	3,160
7.02	5,930,000	1,098,750	79,633	4,437
10.07	6,380,000	1,575,724	114,202	6,363
10.97	6,790,000	1,716,522	124,407	6,931
11.96	7,220,000	1,872,115	135,683	7,560
12.08	7,480,000	1,891,174	137,065	7,636
13.93	8,320,000	2,180,471	158,032	8,805
14.05	8,620,000	2,199,453	159,408	8,881
14.98	8,620,000	2,345,433	169,988	9,471
20.01	9,320,000	3,132,408	227,024	12,648
20.08	9,820,000	3,143,712	227,844	12,694
21.91	10,420,000	3,429,203	248,535	13,847
22.03	10,720,000	3,447,845	249,886	13,922
23.88	11,220,000	3,737,589	270,886	15,092
23.99	11,520,000	3,755,273	272,167	15,164
24.11	11,920,000	3,773,837	273,513	15,239
26.98	11,920,000	4,223,498	306,102	17,054
30.07	12,920,000	4,707,209	341,160	19,007
30.97	13,320,000	4,848,007	351,364	19,576
31.97	13,720,000	5,003,605	362,641	20,204
32.09	13,920,000	5,022,664	364,023	20,281
33.94	14,720,000	5,311,938	384,988	21,449
34.06	15,020,000	5,330,943	386,365	21,526
34.99	15,020,000	5,476,917	396,945	22,116
40.02	15,620,000	6,263,892	453,982	25,293
40.09	16,120,000	6,275,196	454,801	25,339
41.92	16,720,000	6,560,688	475,492	26,492
42.03	17,020,000	6,579,330	476,843	26,567
43.89	17,420,000	6,869,074	497,843	27,737
44.00	17,820,000	6,886,757	499,125	27,808
44.12	18,120,000	6,905,310	500,469	27,883
46.99	18,220,000	7,354,893	533,053	29,699
50.00	19,120,000	7,826,793	567,255	31,604

Table 20.2. Delivery of sediment to the ocean for the Natural Transport Alternative – no removal of fines (Alternative 2b, 3b).

	Delivery to ocean			
year	fines	sands	gravels	cobles
0	0	0	0	0
0.11	2,920,000	17,688	1,226	68
1.90	4,470,000	310,454	21,517	1,199
1.97	4,780,000	322,811	22,373	1,246
2.06	5,140,000	336,451	23,318	1,299
3.87	5,280,000	633,228	43,887	2,445
3.94	6,460,000	645,503	44,738	2,493
4.03	6,480,000	659,143	45,683	2,545
4.10	6,840,000	671,419	46,534	2,593
5.00	6,850,000	818,294	56,713	3,160
7.02	7,610,000	1,148,990	79,633	4,437
10.07	8,060,000	1,647,774	114,202	6,363
10.97	8,470,000	1,795,010	124,407	6,931
11.96	8,900,000	1,957,718	135,683	7,560
12.08	9,160,000	1,977,648	137,065	7,636
13.93	10,000,000	2,280,174	158,032	8,805
14.05	10,300,000	2,300,023	159,408	8,881
14.98	10,300,000	2,452,678	169,988	9,471
20.01	11,000,000	3,275,637	227,024	12,648
20.08	11,500,000	3,287,458	227,844	12,694
21.91	12,100,000	3,586,004	248,535	13,847
22.03	12,400,000	3,605,498	249,886	13,922
23.88	12,900,000	3,908,491	270,886	15,092
23.99	13,200,000	3,926,983	272,167	15,164
24.11	13,600,000	3,946,396	273,513	15,239
26.98	13,600,000	4,416,617	306,102	17,054
30.07	14,600,000	4,922,446	341,160	19,007
30.97	15,000,000	5,069,682	351,364	19,576
31.97	15,400,000	5,232,395	362,641	20,204
32.09	15,600,000	5,252,326	364,023	20,281
33.94	16,400,000	5,554,826	384,988	21,449
34.06	16,700,000	5,574,701	386,365	21,526
34.99	16,700,000	5,727,349	396,945	22,116
40.02	17,300,000	6,550,308	453,982	25,293
40.09	17,800,000	6,562,130	454,801	25,339
41.92	18,400,000	6,860,675	475,492	26,492
42.03	18,700,000	6,880,170	476,843	26,567
43.89	19,100,000	7,183,162	497,843	27,737
44.00	19,500,000	7,201,654	499,125	27,808
44.12	19,800,000	7,221,055	500,469	27,883

46.99	19,900,000	7,691,196	533,053	29,699
50.00	20,800,000	8,184,673	567,255	31,604

Table 20.3. Delivery of sediment to the ocean for the Mechanical Removal and Permanent Stabilization Alternatives (Alternative 1 and 4a).

	Delivery to ocean			
year	**fines**	**sands**	**gravels**	**cobles**
0	0	0	0	0
0.11	109,000	15,330	1,062	59
1.90	668,000	269,060	18,648	1,039
1.97	797,000	279,769	19,390	1,080
2.06	1,010,000	291,591	20,209	1,126
3.87	1,100,000	548,797	38,035	2,119
3.94	1,790,000	559,436	38,773	2,160
4.03	1,810,000	571,258	39,592	2,206
4.10	2,090,000	581,897	40,329	2,247
5.00	2,090,000	709,188	49,152	2,738
7.02	2,640,000	995,791	69,015	3,845
10.07	2,980,000	1,428,071	98,975	5,514
10.97	3,310,000	1,555,676	107,819	6,007
11.96	3,720,000	1,696,689	117,592	6,552
12.08	3,970,000	1,713,962	118,789	6,618
13.93	4,680,000	1,976,150	136,961	7,631
14.05	4,920,000	1,993,353	138,153	7,697
14.98	4,970,000	2,125,654	147,323	8,208
20.01	5,520,000	2,838,885	196,754	10,962
20.08	5,930,000	2,849,130	197,464	11,002
21.91	6,470,000	3,107,870	215,397	12,001
22.03	6,770,000	3,124,765	216,568	12,066
23.88	7,210,000	3,387,359	234,767	13,080
23.99	7,540,000	3,403,385	235,878	13,142
24.11	7,810,000	3,420,210	237,044	13,207
26.98	7,850,000	3,827,735	265,289	14,780
30.07	8,680,000	4,266,120	295,672	16,473
30.97	9,040,000	4,393,724	304,516	16,966
31.97	9,410,000	4,534,743	314,289	17,510
32.09	9,670,000	4,552,016	315,486	17,577
33.94	10,400,000	4,814,183	333,656	18,589
34.06	10,600,000	4,831,407	334,850	18,656
34.99	10,700,000	4,963,703	344,019	19,167
40.02	11,200,000	5,676,934	393,451	21,921
40.09	11,600,000	5,687,179	394,161	21,960
41.92	12,100,000	5,945,919	412,093	22,959
42.03	12,400,000	5,962,814	413,264	23,025
43.89	12,900,000	6,225,407	431,464	24,039
44.00	13,200,000	6,241,434	432,575	24,101
44.12	13,500,000	6,258,248	433,740	24,166
46.99	13,500,000	6,665,703	461,979	25,739
50.00	14,300,000	7,093,384	491,621	27,390

Table 20.4. Delivery of sediment to the ocean for the No Action Alternative.

		Delivery to ocean		
year	**fines**	**sands**	**gravels**	**cobles**
0	0	0	0	0
0.11	165,000	14,727	1,021	57
1.90	818,000	254,997	17,673	985
1.97	936,000	265,132	18,375	1,024
2.06	1,130,000	276,312	19,150	1,067
3.87	1,200,000	515,971	35,760	1,992
3.94	1,970,000	525,878	36,447	2,031
4.03	1,980,000	536,879	37,209	2,073
4.10	2,270,000	546,773	37,895	2,111
5.00	2,270,000	664,274	46,039	2,565
7.02	2,860,000	924,377	64,066	3,569
10.07	3,230,000	1,306,555	90,553	5,045
10.97	3,590,000	1,418,488	98,311	5,477
11.96	4,010,000	1,541,103	106,809	5,951
12.08	4,240,000	1,556,106	107,849	6,009
13.93	5,000,000	1,780,115	123,374	6,874
14.05	5,260,000	1,794,796	124,392	6,930
14.98	5,310,000	1,906,759	132,152	7,363
20.01	5,890,000	2,482,761	172,073	9,587
20.08	6,340,000	2,491,029	172,646	9,619
21.91	6,930,000	2,696,212	186,866	10,411
22.03	7,230,000	2,709,595	187,794	10,463
23.88	7,700,000	2,913,854	201,950	11,252
23.99	8,050,000	2,926,306	202,813	11,300
24.11	8,350,000	2,939,364	203,718	11,350
26.98	8,380,000	3,246,627	225,014	12,536
30.07	9,300,000	3,566,736	247,200	13,773
30.97	9,700,000	3,659,030	253,596	14,129
31.97	10,100,000	3,759,948	260,590	14,519
32.09	10,400,000	3,772,293	261,446	14,566
33.94	11,100,000	3,955,936	274,174	15,275
34.06	11,400,000	3,967,985	275,009	15,322
34.99	11,400,000	4,059,583	281,357	15,676
40.02	12,000,000	4,525,927	313,678	17,476
40.09	12,500,000	4,536,172	314,388	17,516
41.92	13,100,000	4,794,911	332,321	18,515
42.03	13,400,000	4,811,806	333,492	18,580
43.89	13,900,000	5,074,400	351,691	19,594
44.00	14,200,000	5,090,426	352,802	19,656
44.12	14,500,000	5,107,240	353,967	19,721
46.99	14,600,000	5,514,695	382,207	21,294
50.00	15,500,000	5,942,376	411,848	22,946

21. Exhibit I. Appraisal Level Design of High flow/Sediment By-pass

The current design of Robles Diversion is shown in Figure 21.2. The primary components of the current structure consist of a diversion dam at a fixed elevation of 765 ft (NGVD 27), radial gates with an approximate capacity of 7200 cfs and a diversion canal with a capacity of 500 cfs.

Four options for passing increased sediment loads anticipated following the decommissioning of Matilija Dam were developed to an appraisal design level. The options were developed to provide a comparison of operational flexibility and cost. Aspects of gate reliability are not addressed in detail herein. At this design level, reliability was limited to feedback received from a phone survey of operators of similar facilities. Design objectives used for the appraisal level design were based on comments provided by the Casitas Municipal Water District (CMWD) and reconnaissance level development presented by Borcalli and Associates (USBR, 2000). Project design data was drawn from Robles Diversion Dam Fish Screen and Fishway Project Specification (2003), Robles Diversion Dam and Robles-Casitas Diversion Canal Specification (1957) and HEC-RAS one-dimensional hydraulic modeling using 2002 river topography. All elevations presented are referenced to NGVD 29 to maintain consistency with fish screen, fishway, and diversion drawings. HEC-RAS modeling was based on topography referenced to NGVD 88 datum. Elevation data presented from the HEC-RAS model are adjusted down by 2.5 ft to match NGVD 29.

Project Goals:

1. Provide for maintaining a nearly constant water surface of 767.0 NAVD 88 for river flows up to 10,000 ft^3/s, (allocation of fish releases and diversion flow may result in lower pool elevations during low river flows.)
2. Provide for sediment sluicing near the left bank. Providing a strong left bank flow may reduce the transport of bed sediments toward the fishway and diversion. (Sediment movement in relation to spillway location would require further analysis.)
3. Begin left bank sluicing when river flows exceed 1,500 ft^3/s. Transport of sediment to the diversion pool is expected to increase significantly as flow exceeds 1,500 ft^3/s.
4. Increase the flexibility of spillway flow releases to enhance fish passage.

Spillway crest length is a function of achieving Project Goal 1. The total spillway flow release capacity must be sufficient to limit the diversion pool elevation to 764.5 for inflows less than 10,000 ft^3/s. Currently, impacts of the right bank spillway operation on fish passage have not been fully identified (Goal 4). In the event a restriction on releases through the right bank spillway is found necessary, the left bank spillway must be sized to pass the additional flow. Figure 21.2 gives the diversion pool water surface elevation as a function of river inflow minus 500 ft^3/s diversion flow for 90 ft and 120 ft long left bank spillways. The data presented represents maximum flow conditions. Gates are fully open and the gate sill and the influence of backwater control flow. For the case of no flow releases from the right bank spillway, a 120-ft- and 90-ft-long gated crest could control pool elevation at 764.5 for flows less than 11,800 ft^3/s and 8,000 ft^3/s, respectively. The addition of full right bank spillway capacity would increase the control of spillway flows to 15,000 ft^3/s and 12,800 ft^3/s. River flows less than those shown on Figure 21.1would be controlled by gated releases.

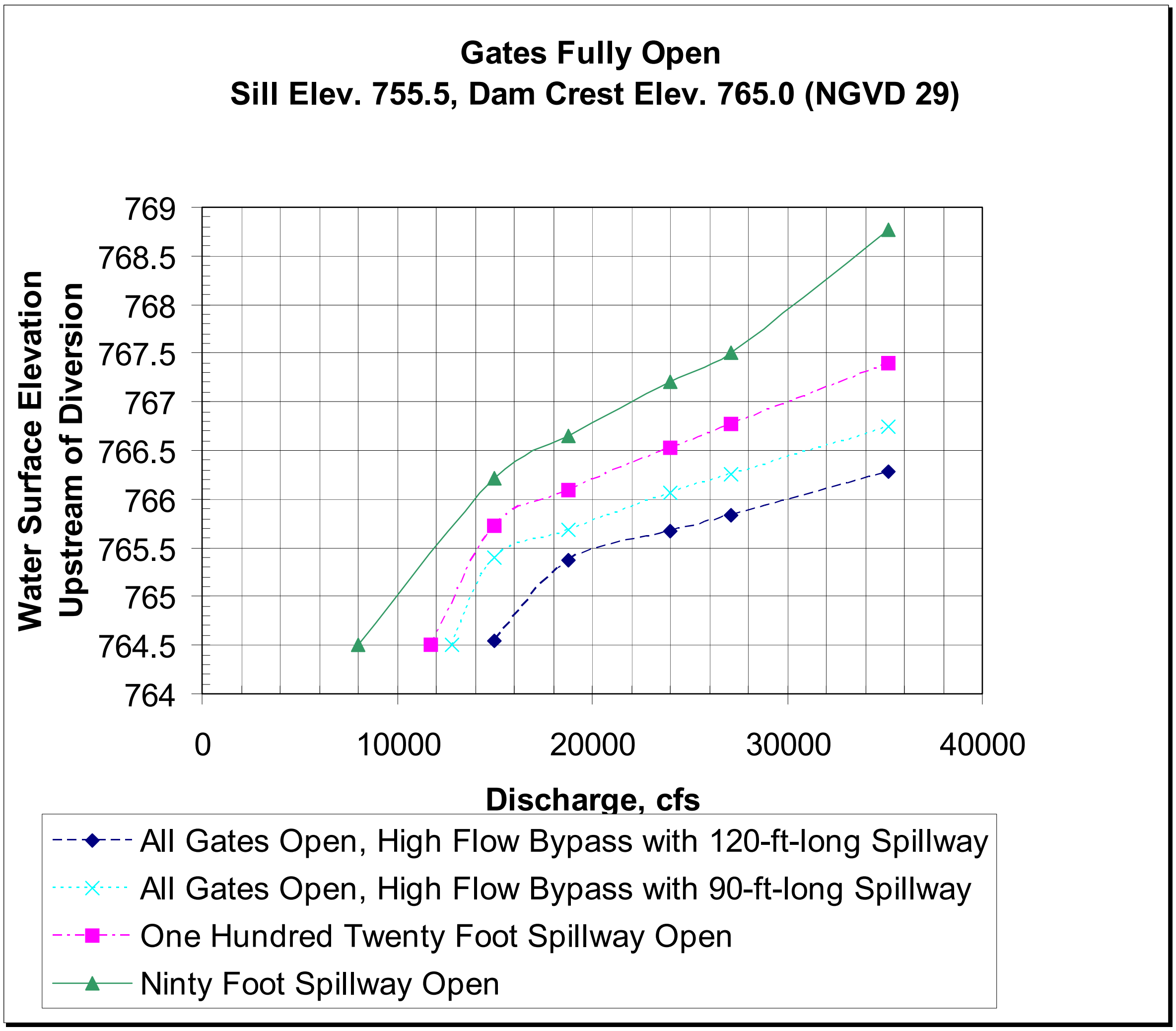


Figure 21.1. Control of Diversion Pool Elevation as a Function of Spillway Length.

Gated spillways using overshot sill gates and radial gates were considered. Four layouts are present herein for comparison. The drawings for each alternative are given in this section.

1. A four bay, 120-ft-long radial gate structure on left side of channel
2. A four bay, 120-ft-long radial gate structure on right side of channel
3. A 330-ft long, air bladder operated overshot gated spillway
4. A 120-ft-long, air bladder operated overshot gated spillway

A short description of each structure is given below.

1. The forebay, 120-ft-long radial gate structure with the structure on the left side is shown of the drawings in this section labeled Alternative 1. The structure shown is similar to the

existing right bank spillway with 30-ft-long radial gates. The structure could control pool elevation for flow releases between 500 ft^3/s and 11,800 ft^3/s. A smaller radial gate would be added to the structure if releases less than 500 ft^3/s were foreseen. Experience with radial gates has shown excellent reliability and minimal water leakage is achievable with good construction. The appraisal level construction cost of a 120-ft-long, 4-bay radial gate spillway for Robles Diversion Dam is $3,500,000.

2. The 4-bay, 120-ft-long radial gate structure on right side of channel is shown as Alternative 2.

3. The 330-ft overshot gated spillway option is given on the drawings in this section labeled Alternative 3. An overshot gate operates by rotating a gate leaf about a hinge mounted along the spillway sill. Overshot gates are commercially available that operate using overhead cable hoists or underlying air bladders. Cable hoist systems require piers between gates similar to radial gates (not shown). The air bladder design does not require piers. The gate leaf is raised by inflating the air bladder that rests downstream of the gate hinge. These gates are designed to pass flow from the fully raised too fully lowered position. Air bladder gates can be designed as a single long span gate or as a series of shorter spans that operate independently. This style of gates should provide good control during high river flow; however, bottom sluicing would require large gate openings and may restrict sluicing at lower river flows. Assuming normal diversion pool and a gate fully lowered to sluice bed load, limiting flows to 500 ft^3/s, 1000 ft^3/s, and 1500 ft^3/s requires gate lengths of 6.3 ft, 12.7 ft, and 20 ft, respectively. Conversations with operators of cable hoist and air bladder operated overshot gates found both gates to have generally good reliability. However, water users did identify several cases where unintended releases occurred due to problems with the gate support systems. On cable hoist systems, some owners have experienced cable fraying and stretching due to vibration of cables exposed to flow and debris. For air bladder systems, several owners reported unintended partial lowering due to loss of air bladder pressure. Leakage through gate seals is generally expected to be higher for overshot gates compared to radial or vertical lift gates. Seals on overshot gates must seal along an entire vertical plain as the gate position changes as compared to a line for radial or vertical lift gates. The appraisal level construction cost of a 330-ft-long, air bladder operated overshot gated spillway for Robles Diversion Dam is $5,600,000.

4. The 120-ft overshot gated spillway option is given on the drawings in this section labeled Alternative 4. The appraisal level construction cost of a 120-ft-long, air bladder operated overshot gated spillway for Robles Diversion Dam is $2,100,000.

CODE:D-8140

ESTIMATE WORKSHEET

SHEET__1__ OF __1___

FEATURE: ROBLES DAM Radial Gates and Structure **20-Dec-03**	**PROJECT:**
	DIVISION:
	UNIT:

C:\Documents and Settings\BMEFFORD\Local Settings\Temp\[Robles Radial Gate Cost __ROUGH__.xls]Obermeyer Gates

PLANT ACCT.	PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
		Structural Concrete		2,021	CY	$500	$1,010,500
		Cement		570	TON	$135	$76,939
		Reinforcement		303,150	LB	$1	$303,150
		10'x30' Radial Gates		68,000	LB	$5	$340,000
		4 radial gates @17,000 lb/gate = 68,000 lb					
		Excavation		4,200	CY	$15	$63,000
		Riprap		1,112	CY	$50	$55,600
		Temporary cofferdam		1	LS	$50,000	$50,000
		Dewatering/Unwatering		1	LS	$50,000	$50,000
		Electrical wiring and control boxes (10% of above value)					$194,919
		Unlisted Items (25% of above value)					$536,027
		Mobility (5% of above value)					$134,007
		Contingencies (25% of above value)					$703,536
		TOTAL					$3,517,678

QUANTITIES		PRICES	
BY JASON WAGNER	APPROVED	BY	CHECKED
DATE PREPARED 12/19/2003	DATE	DATE	PRICE LEVEL appraisal

CODE:D-8140

ESTIMATE WORKSHEET

SHEET__1__ OF __1___

FEATURE: 05-Jan-04 **ROBLES DAM** **Obermeyer Gates and Structure** **330' Width of Obermeyer Structure**	**PROJECT:** **DIVISION:** **UNIT:**

G:\My Documents\Matilija-Feasibility\alternatives\high flow bypass\[Robles Replacement Options__ROUGH__.xls]Summary

PLANT ACCT.	PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
		330' wide section Obermeyer Gates (Assumed $400 ft^2)		1	LS	$1,375,000	$1,375,000
		Concrete		1,956	CY	$500	$978,000
		Reinforcement		293,400	LB	$1	$293,400
		Excavation		11,550	CY	$15	$173,250
		Riprap		3,058	CY	$50	$152,900
		Sheetpile (Assumed AZ 26, 30' deep)		157	TON	$1,850	$289,988
		Temporary cofferdam		1	LS	$50,000	$50,000
		Dewatering/Unwatering		1	LS	$50,000	$50,000
		Electrical wiring and control boxes (5% of above value)					$168,127
		Unlisted Items (20% of above value)					$706,133
		Mobilization (5% of above values)					$211,840
		Contingencies (25% of above value)					$1,112,159
		TOTAL					$5,560,796

QUANTITIES		PRICES	
BY JW	APPROVED	BY	CHECKED
DATE PREPARED 12/19/2003	DATE	DATE	PRICE LEVEL Appraisal

120' Width of Obermeyer Structure

DIVISION:

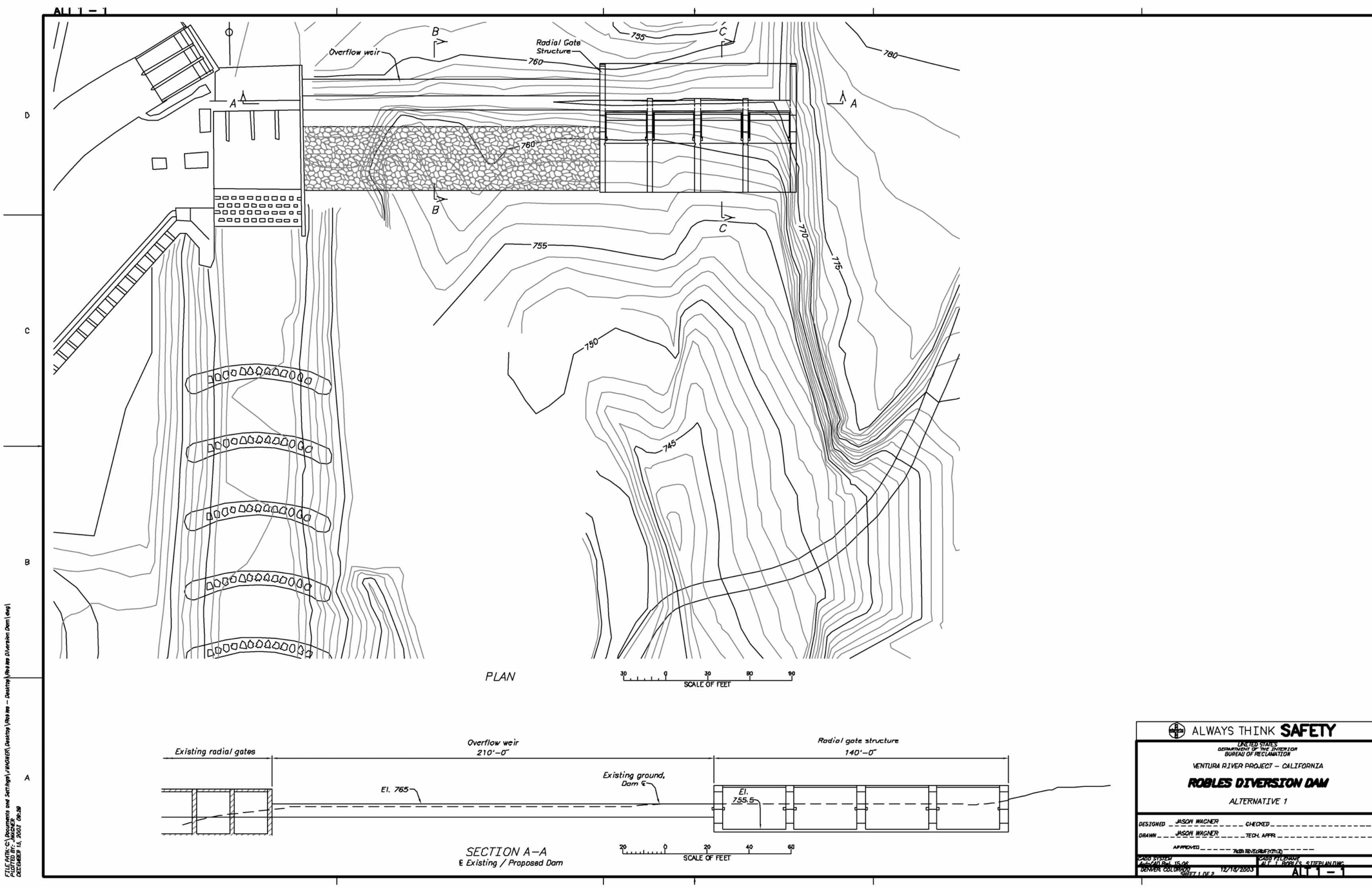
UNIT:

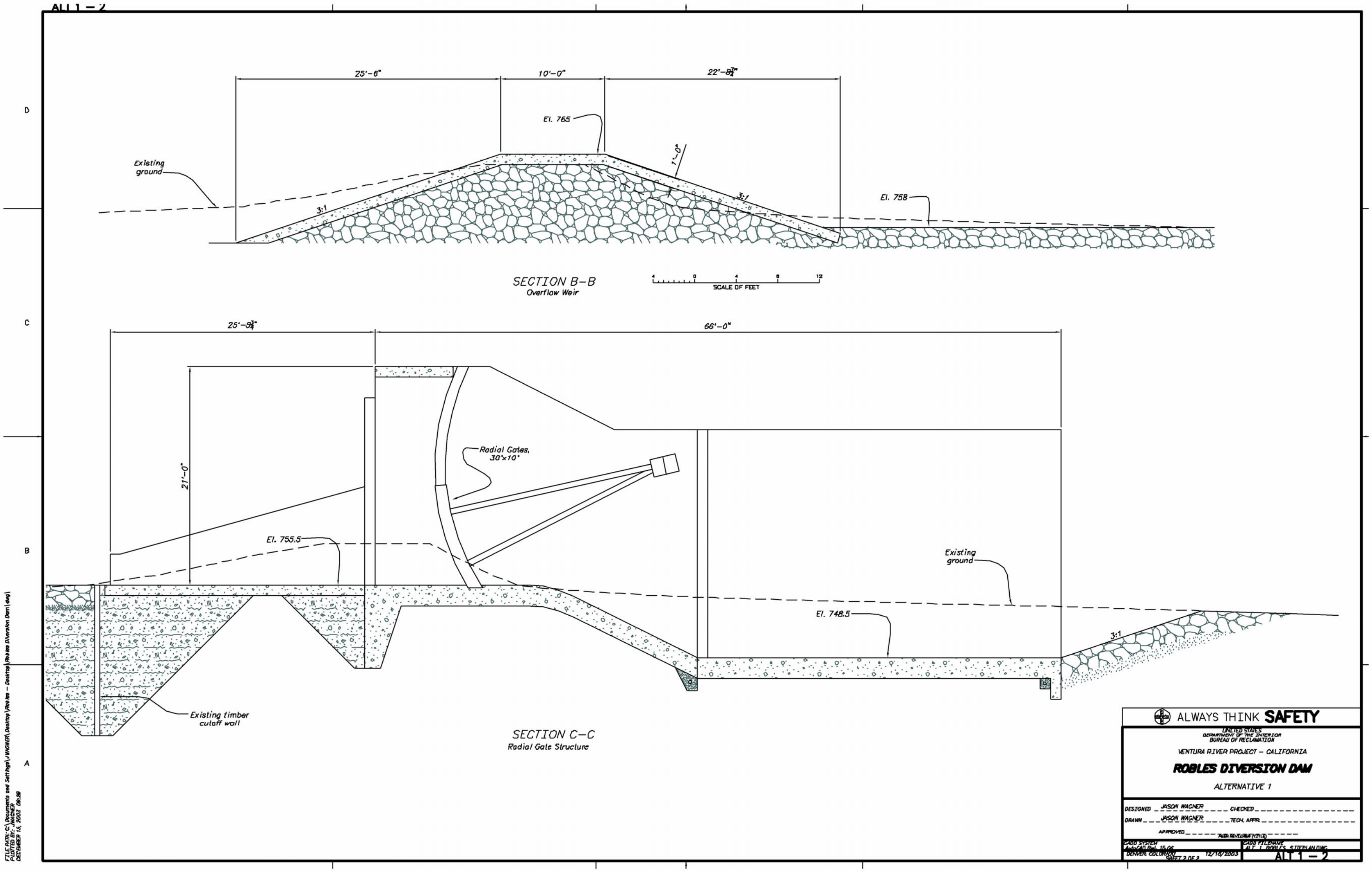
G:\My Documents\Matilija-Feasibility\alternatives\high flow bypass\[Robles Replacement Options__ROUGH__.xls]Obermeyer Gates, 120ft

PLANT ACCT.	PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
		120' wide section Obermeyer Gates (Assumed $400 ft^2)		1	LS	$500,000	$500,000
		Concrete		712	CY	$500	$356,000
		Reinforcement		106,800	LB	$1	$106,800
		Excavation		4,200	CY	$15	$63,000
		Riprap		1,112	CY	$50	$55,600
		Sheetpile (Assumed AZ 26, 30' deep)		57	TON	$1,850	$105,450
		Temporary cofferdam		1	LS	$50,000	$50,000
		Dewatering/Unwatering		1	LS	$50,000	$50,000
		Electrical wiring and control boxes (5% of above value)					$64,343
		Unlisted Items (20% of above value)					$270,239
		Mobilization (5% of above values)					$81,072
		Contingencies (25% of above value)					$425,626
		TOTAL					$2,128,128

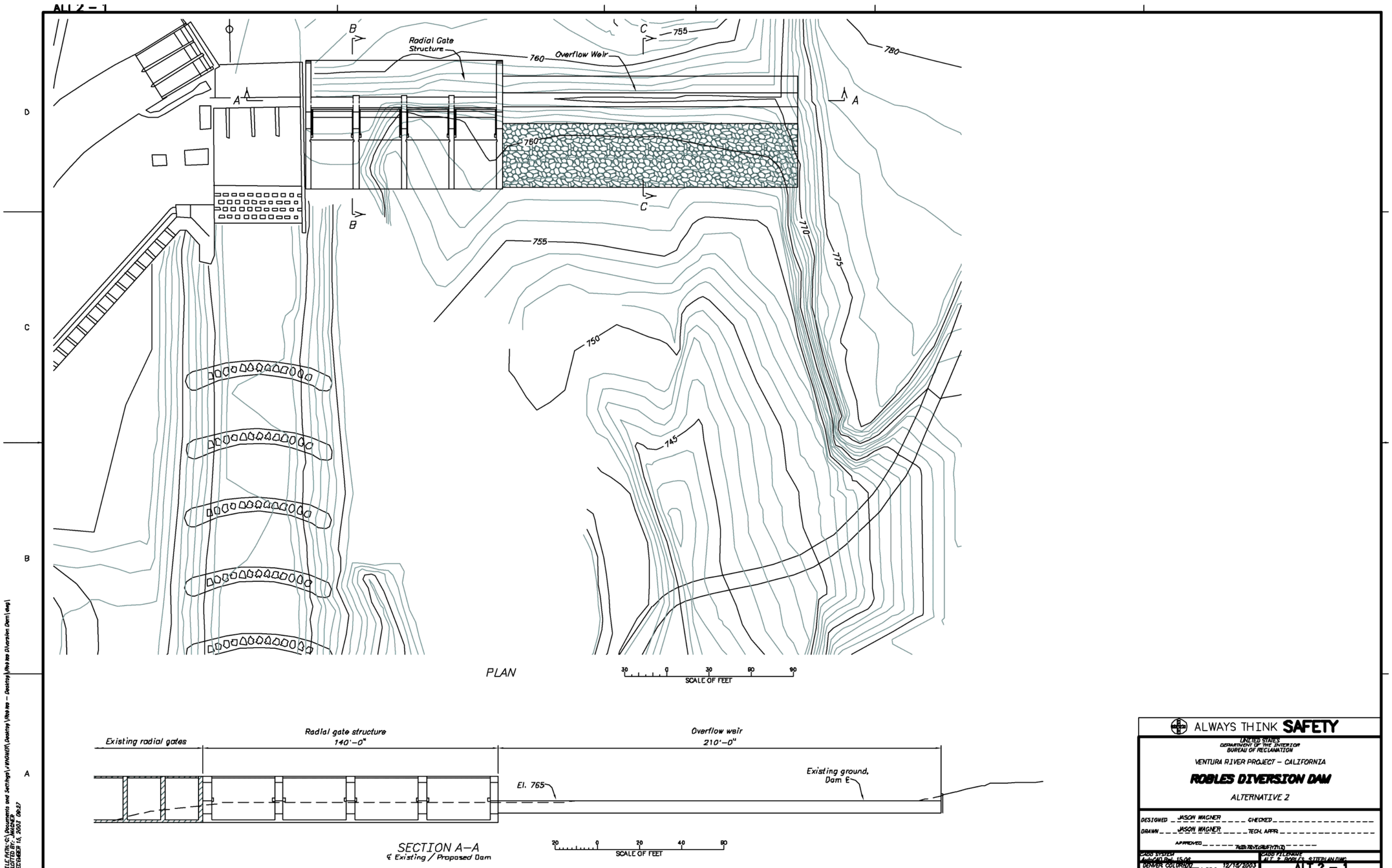
QUANTITIES		PRICES	
BY JW	APPROVED	BY	CHECKED
DATE PREPARED 12/19/2003	DATE	DATE	PRICE LEVEL Appraisal

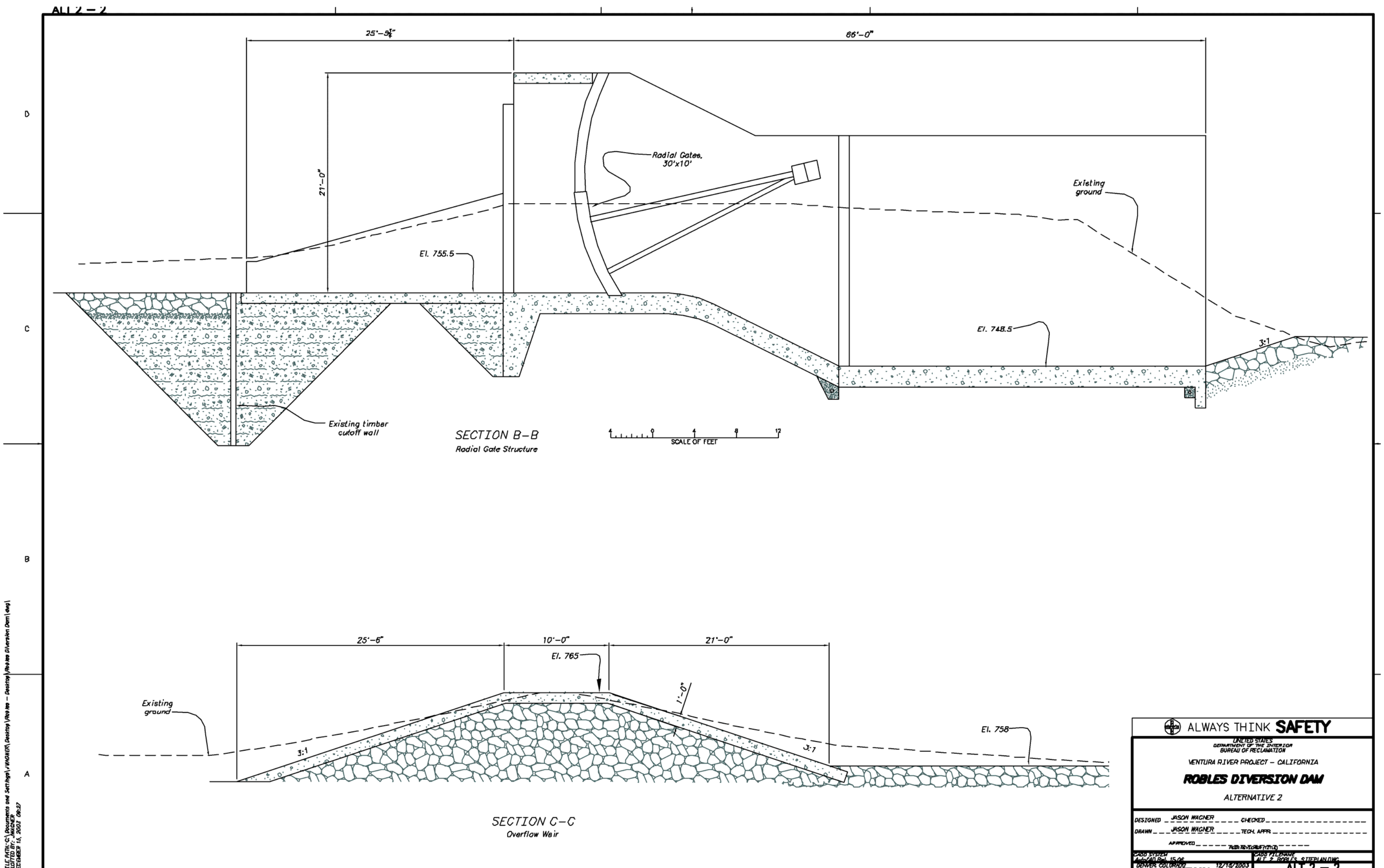
Figure 21.2. Aerial Photograph of Robles Diversion taken in 2001.

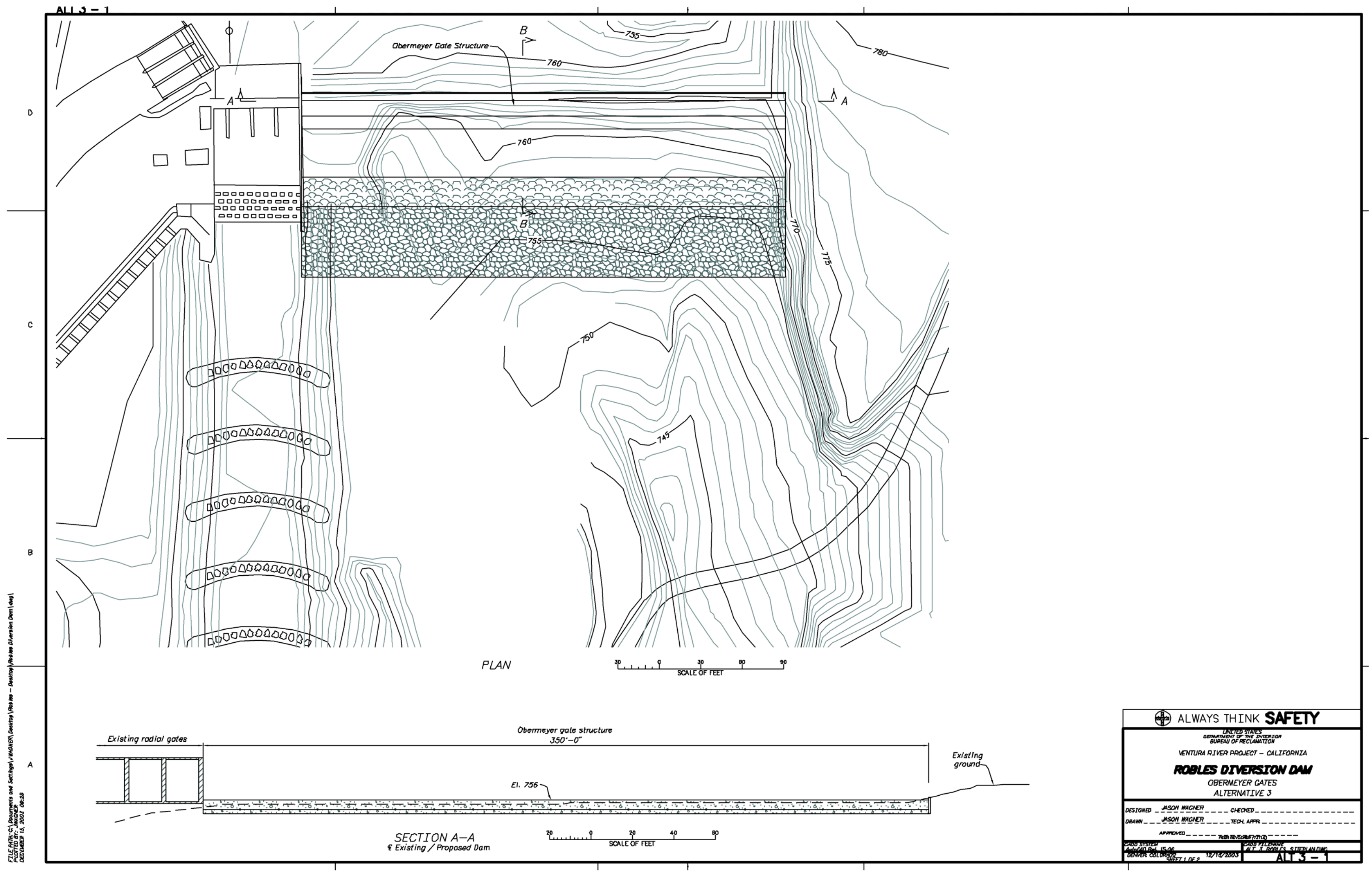

ALT 1 – 1
Overflow weir
B
Radial Gate Structure
760
755
C
780
A
760
770
775
755
750
745
PLAN
SCALE OF FEET
Existing radial gates
Overflow weir
210'–0"
Radial gate structure
140'–0"
Existing ground, Dam ℄
El. 765
El. 755.5
SECTION A–A
℄ Existing / Proposed Dam
ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
VENTURA RIVER PROJECT – CALIFORNIA
ROBLES DIVERSION DAM
ALTERNATIVE 1
DESIGNED JASON WAGNER
CHECKED
DRAWN JASON WAGNER
TECH. APPR.
APPROVED
CADD SYSTEM
AutoCAD Rel. 15.06
CADD FILENAME
ALT 1 ROBLES SITEPLAN.DWG
DENVER, COLORADO
SHEET 1 OF 2
12/15/2003
ALT 1 – 1



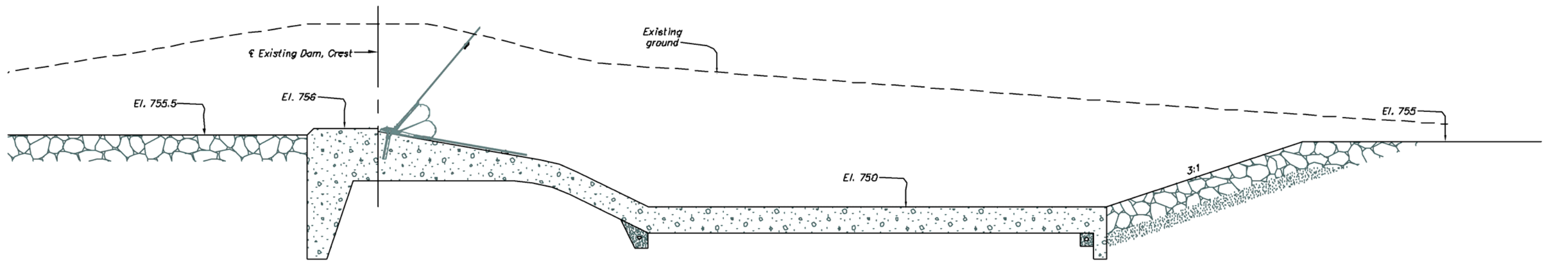
ALT 1 – 2
25'-6"
10'-0"
22'-8¾"
El. 765
1'-0"
Existing ground
3:1
3:1
El. 758
SECTION B–B
Overflow Weir
SCALE OF FEET
25'-8¾"
66'-0"
21'-0"
Radial Gates, 30'x10'
El. 755.5
Existing ground
El. 748.5
3:1
Existing timber cutoff wall
SECTION C–C
Radial Gate Structure
ALWAYS THINK SAFETY
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION
VENTURA RIVER PROJECT – CALIFORNIA
ROBLES DIVERSION DAM
ALTERNATIVE 1
DESIGNED JASON WAGNER CHECKED
DRAWN JASON WAGNER TECH. APPR.
APPROVED
DENVER, COLORADO
12/16/2003
ALT 1 – 2


ALT 2 - 1
Radial Gate Structure
Overflow Weir
760
755
780
770
775
750
745
PLAN
SCALE OF FEET
Existing radial gates
Radial gate structure
140'-0"
Overflow weir
210'-0"
El. 765
Existing ground, Dam ℄
SECTION A–A
℄ Existing / Proposed Dam
ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
VENTURA RIVER PROJECT - CALIFORNIA
ROBLES DIVERSION DAM
ALTERNATIVE 2
DESIGNED JASON WAGNER
DRAWN JASON WAGNER
CHECKED
TECH. APPR.
APPROVED
DENVER, COLORADO
12/15/2003
ALT 2 - 1


ALT 2 – 2
25'-9¾"
86'-0"
21'-0"
Radial Gates, 30'x10'
Existing ground
El. 755.5
El. 748.5
3:1
Existing timber cutoff wall
SECTION B–B
Radial Gate Structure
4 0 4 8 12
SCALE OF FEET
25'-6"
10'-0"
21'-0"
El. 765
1'-0"
Existing ground
3:1
3:1
El. 758
SECTION C–C
Overflow Weir
ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
VENTURA RIVER PROJECT – CALIFORNIA
ROBLES DIVERSION DAM
ALTERNATIVE 2
DESIGNED JASON WAGNER CHECKED
DRAWN JASON WAGNER TECH. APPR.
APPROVED
CADD SYSTEM
AutoCAD Rel. 15.06
CADD FILENAME
ALT 2 ROBLES SITEPLAN.DWG
DENVER, COLORADO 12/18/2003
SHEET 2 OF 2
ALT 2 – 2


ALT 3 – 1
Obermeyer Gate Structure
B
755
760
780
A
A
760
B
755
770
775
750
745
PLAN
SCALE OF FEET
30 0 30 60 90
Existing radial gates
Obermeyer gate structure
350'–0"
El. 756
Existing ground
SECTION A–A
℄ Existing / Proposed Dam
SCALE OF FEET
20 0 20 40 60
ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
VENTURA RIVER PROJECT – CALIFORNIA
ROBLES DIVERSION DAM
OBERMEYER GATES
ALTERNATIVE 3
DESIGNED JASON WAGNER CHECKED
DRAWN JASON WAGNER TECH. APPR.
APPROVED
CADD SYSTEM
CADD FILENAME
DENVER, COLORADO 12/16/2003
SHEET 1 OF 2
ALT 3 – 1

ALT 3 – 2



SECTION B–B
Obermeyer Gate Structure

SCALE OF FEET

ALWAYS THINK **SAFETY**

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

VENTURA RIVER PROJECT – CALIFORNIA

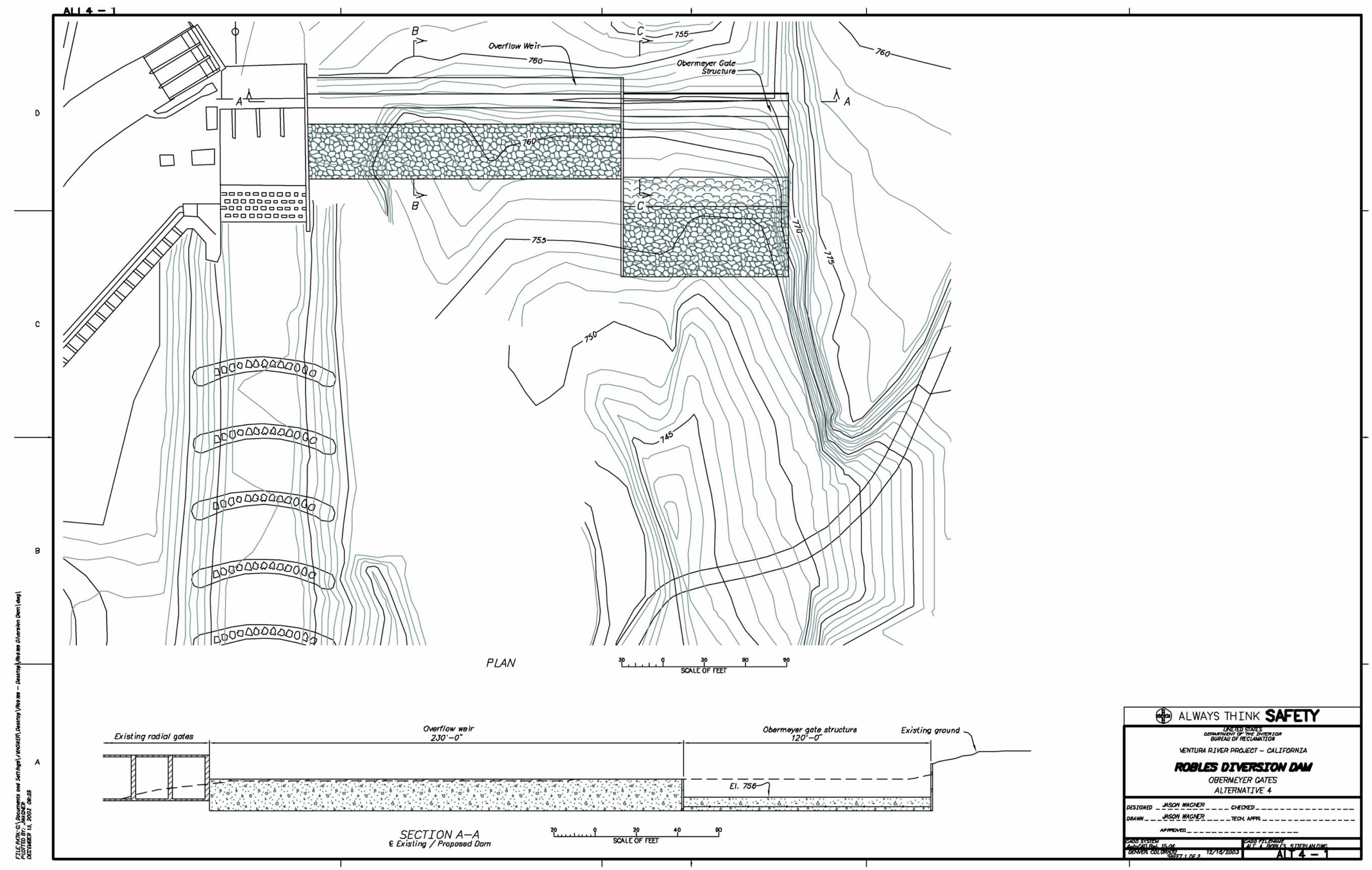
ROBLES DIVERSION DAM

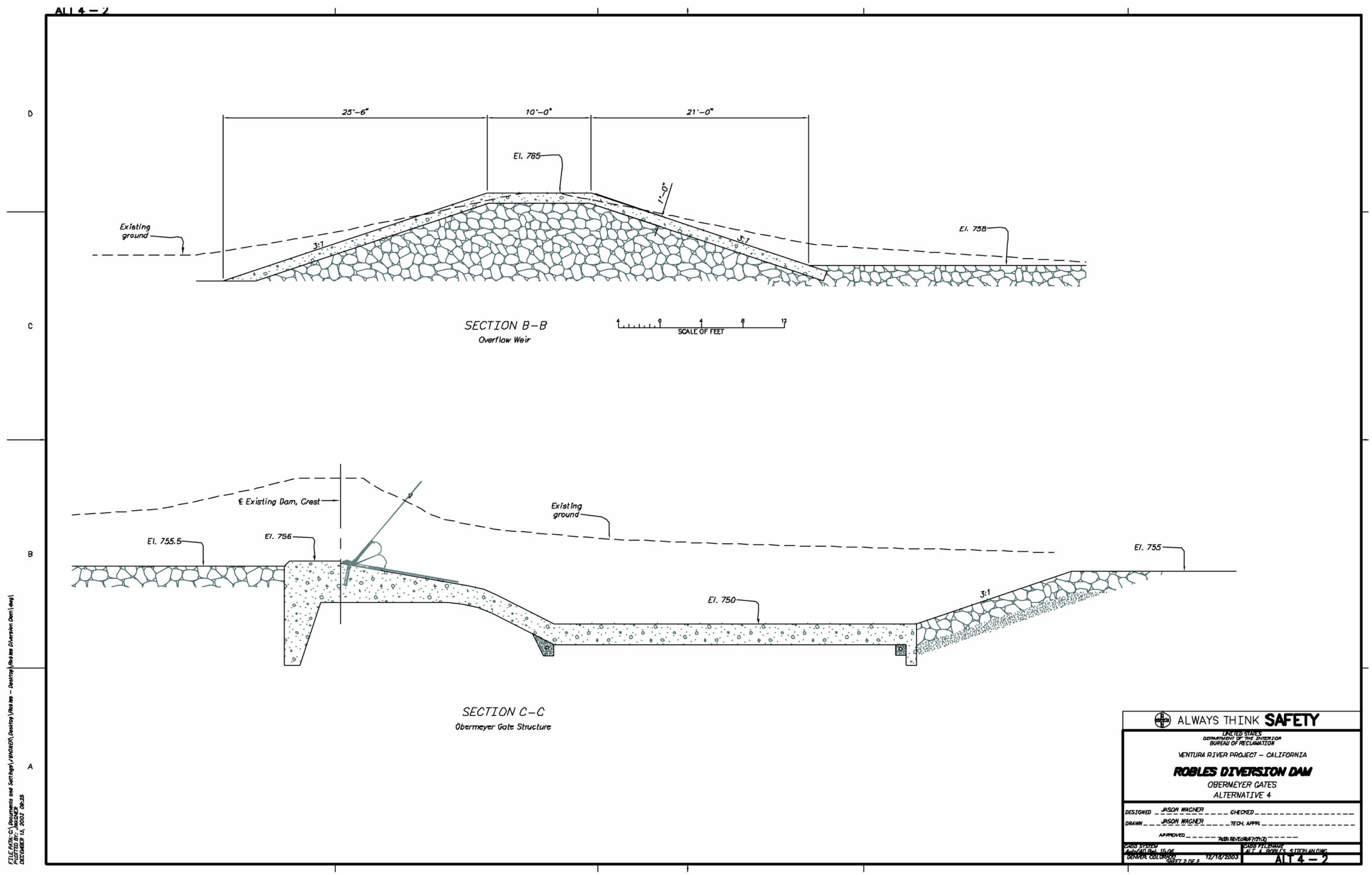
OBERMEYER GATES
ALTERNATIVE 3

DESIGNED JASON WAGNER CHECKED
DRAWN JASON WAGNER TECH. APPR.
APPROVED

DENVER, COLORADO 12/14/2001

ALT 3 – 2


ALT 4 – 1
Overflow Weir
Obermeyer Gate Structure
PLAN
SCALE OF FEET
Existing radial gates
Overflow weir 230'–0"
Obermeyer gate structure 120'–0"
Existing ground
El. 756
SECTION A–A
℄ Existing / Proposed Dam
ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
VENTURA RIVER PROJECT – CALIFORNIA
ROBLES DIVERSION DAM
OBERMEYER GATES
ALTERNATIVE 4
DESIGNED JASON WAGNER
CHECKED
DRAWN JASON WAGNER
TECH. APPR.
APPROVED
DENVER, COLORADO
12/15/2003
SHEET 1 OF 2
ALT 4 – 1


ALT 4 – 2
25'-6"
10'-0"
21'-0"
El. 765
1'-0"
Existing ground
3:1
3:1
El. 758
SECTION B–B
Overflow Weir
SCALE OF FEET
¢ Existing Dam, Crest
Existing ground
El. 755.5
El. 756
El. 755
El. 750
3:1
SECTION C–C
Obermeyer Gate Structure
ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
VENTURA RIVER PROJECT – CALIFORNIA
ROBLES DIVERSION DAM
OBERMEYER GATES
ALTERNATIVE 4
DESIGNED JASON WAGNER
CHECKED
DRAWN JASON WAGNER
TECH. APPR.
APPROVED
DENVER, COLORADO
12/16/2003
ALT 4 – 2

Examples of current installations of Obermeyer gates are shown in Figure 21.3 to Figure 21.7. The examples were provided by Obermeyer Hydro Inc. (http://www.obermeyerhydro.com).

1. The use of a flat clamp system as shown in the Japan installation is shown below. This clamp system uses a flush mount clamp system in conjunction with female anchor bolts. The result is a very low profile gate structure in the open position. It should be also noted that a secondary reinforced rubber hinge shield was incorporated to keep rocks and debris out of the gate hinge area. This is also shown in the attached drawing. Most of the projects in Japan pass sediment load in the small to medium category. The attached drawings show a 10-foot high by approximately 70-foot long gate.

Figure 21.3. Installation of Obermeyer Gates in Japan.

2. An installation in Peru passes large boulders along with finer cobbles and sand loads. For fine bed loads, a stainless steel gate panels is recommend for the lower height gates and a reinforced rubber gate mat for higher gates. This particular project utilized a reinforced rubber mat attached to the upstream face of the gate panel. This mat not only protects the upstream gate panel surface from abrasion due to passing fine sediments but also protects the gate from the impact of larger rocks and debris.

Figure 21.4. Installation of Obermeyer Gates in Peru.

Figure 21.5. Installation of Obermeyer Gates in Peru.

3. A third installation shows a typical high head hydroelectric run of river intakes at one of our projects in British Columbia. From the photographs you can see the size of sediment and the protection in the hinge area.

Figure 21.6. Installation of Obermeyer Gates in British Columbia.

Figure 21.7. Installation of Obermeyer Gates in British Columbia.

22. Exhibit J. Conceptual Design of De-silting Basin

A conceptual design of the de-silting basin is shown in Figure 22.1. The basis for the deign was the GSTARS-1D simulation results presented in Section 9.3 and Section 10.3. Chemical floculant is added to the water in Robles Canal before the entrance to the desilting basin. The sediment flocculates and then enters the de-silting basin. The sediment settles to the bottom and cear water exits from the basin and then flows to Lake Casitas.

Three sites were analyzed for their suitability for a de-silting basin. Figure 22.2Figure 22.2 shows an aerial photograph with each site identified. The costs for each site are given in Table 22.1. Based on the current analyses, the preferred site is Site 3. This site is preferred bcause its cost is the least and the site is in a more remote location.

Table 22.1. Cost of alternative sites for desilting basin.

Proposed Site	**Basin Construction** ($ millions)	**Sludge Storage*** ($ millions)	**Chemical Desilting** ($ millions)	**Total** ($ millions)
1 (5 acres)	**1.6**	**1.8**	**0.5**	**3.9**
2 (13.2 acres)	**2.4**	**1.3**	**0.5**	**4.2**
3 (5 acres)	**1.3**	**1.1**	**0.5**	**2.9**

*** Storage site 13 acres- 15 feet high**

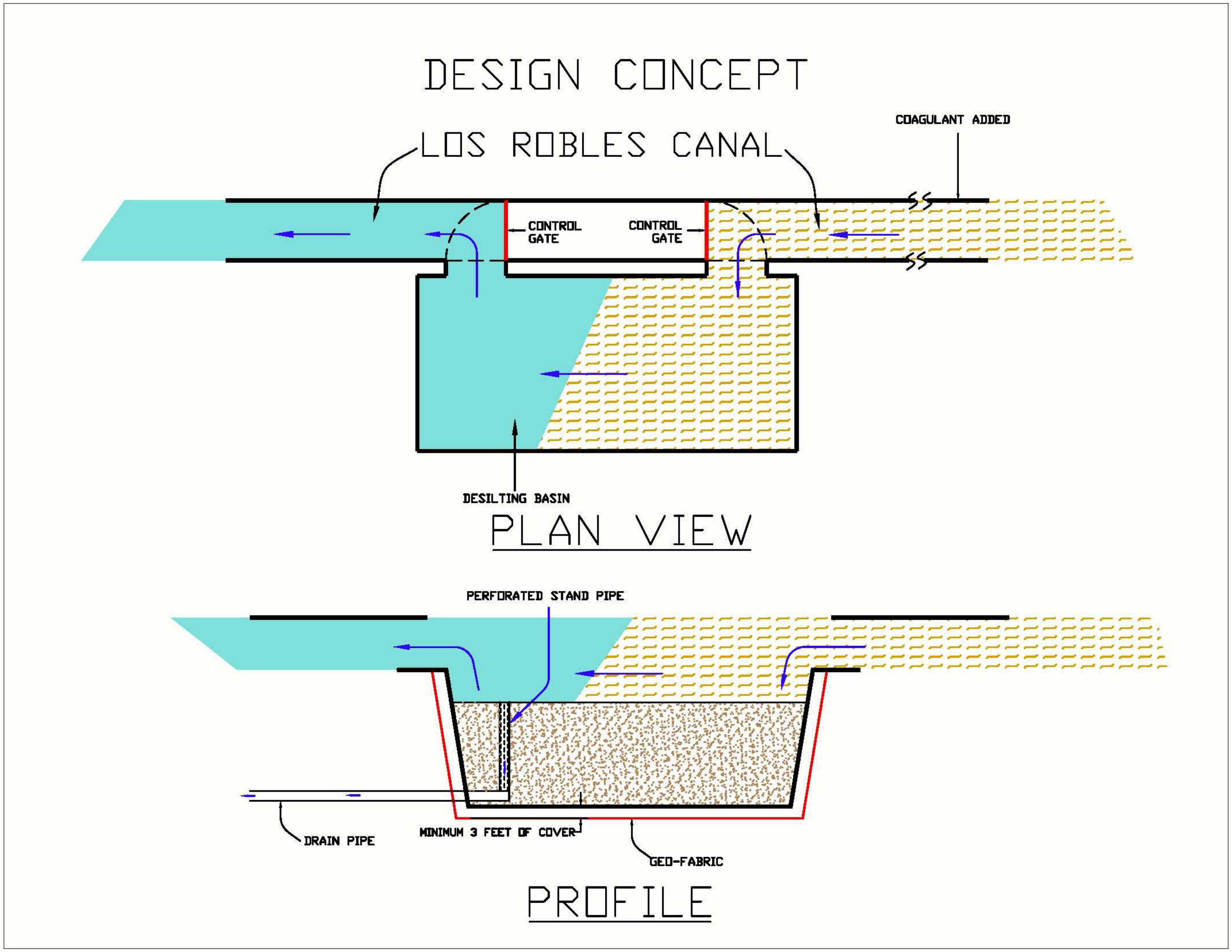


Figure 22.1. Conceptual Design of De-silting Basin.

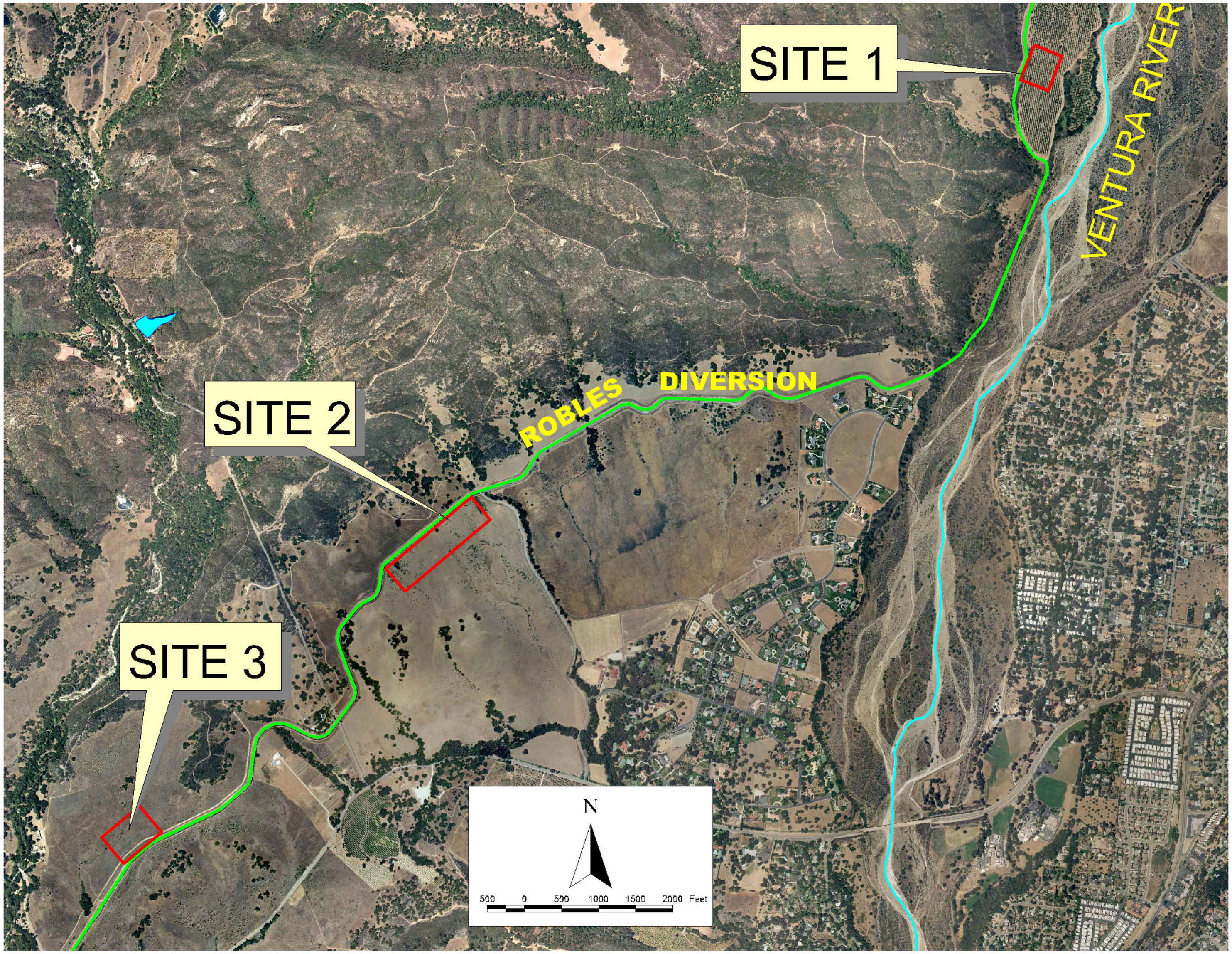


Figure 22.2. Alternative sites for the de-silting basin.

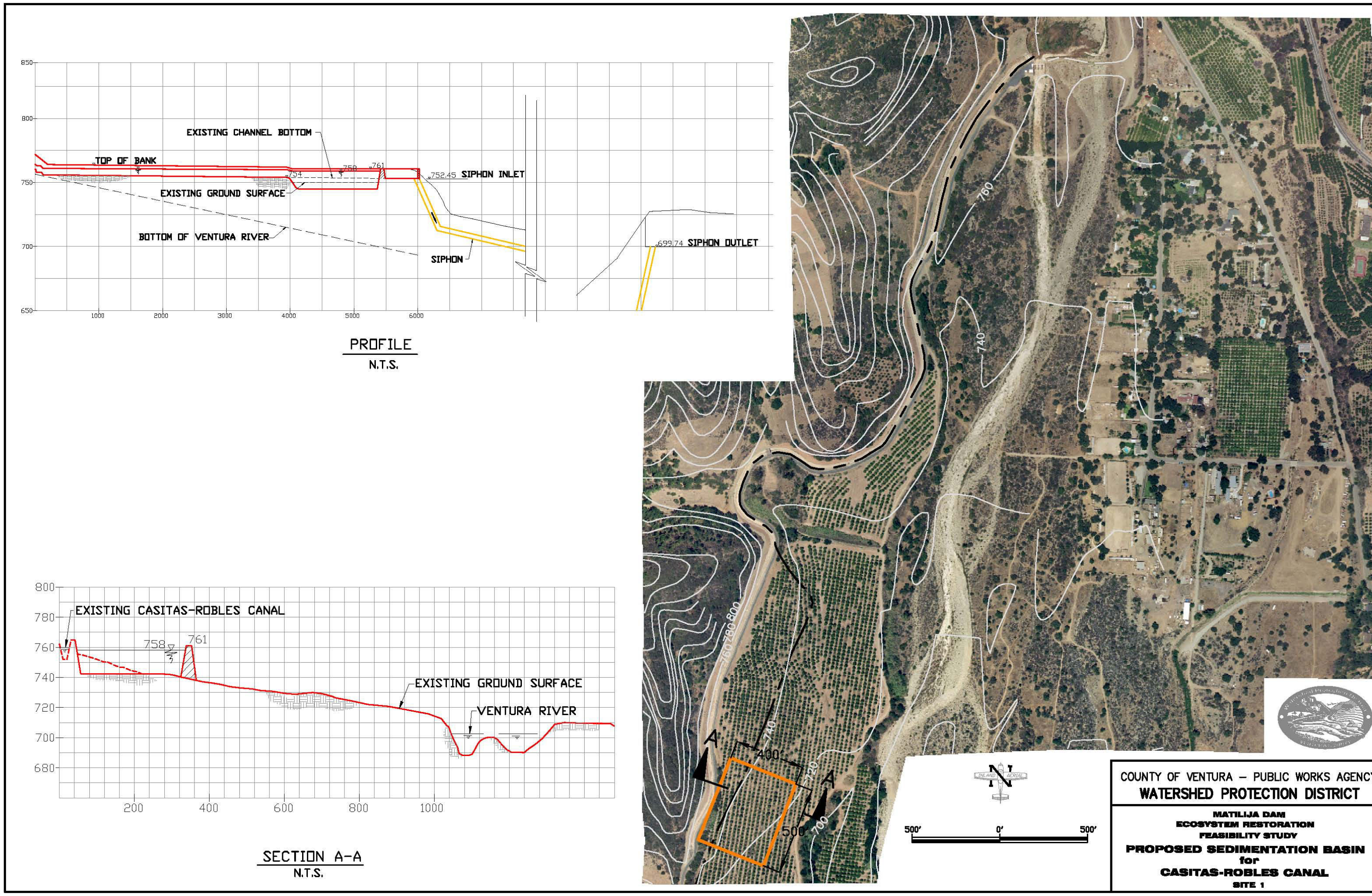


Figure 22.3. Site 1 Drawing.

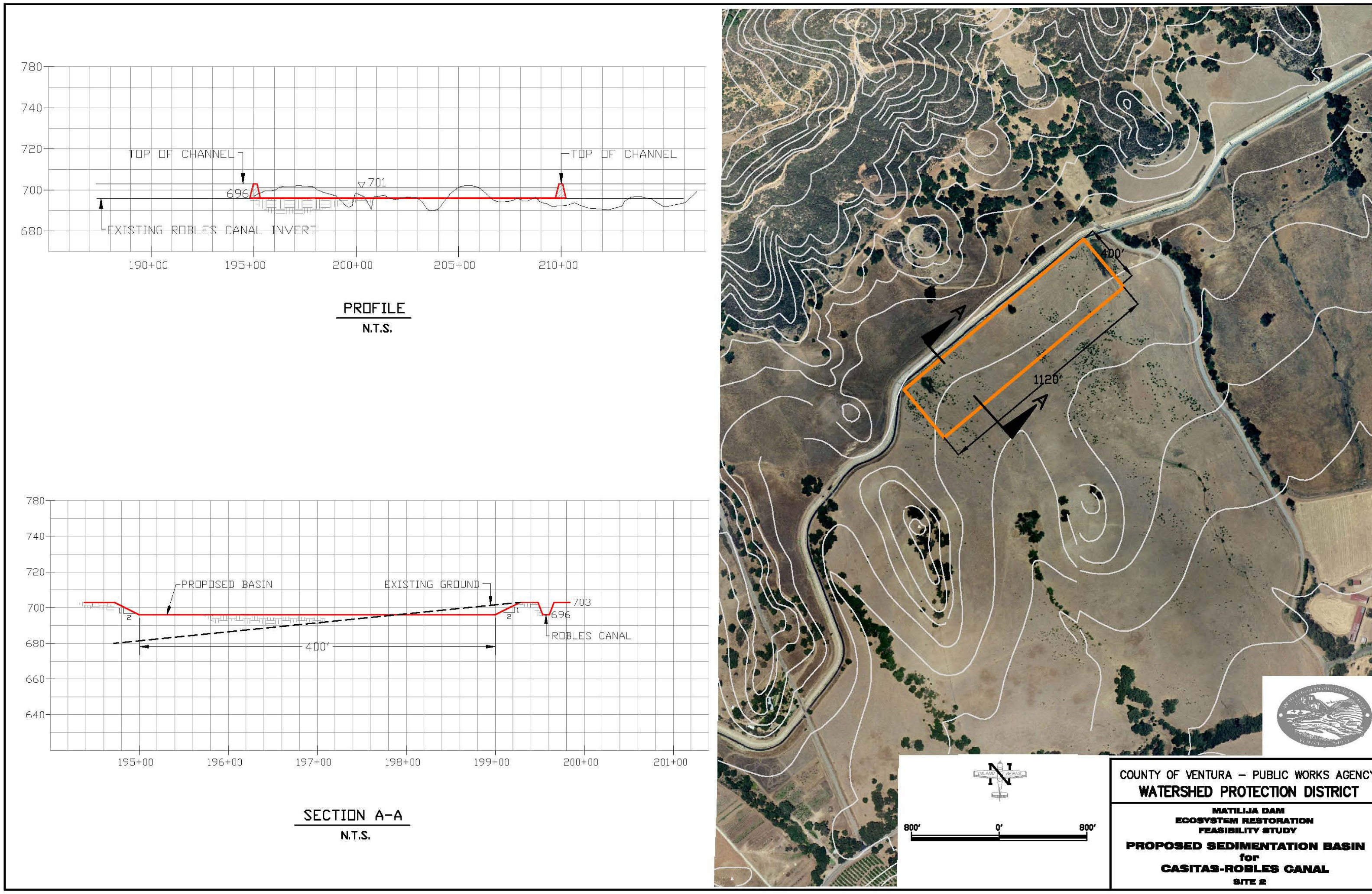


Figure 22.4. Site 2 Drawing.

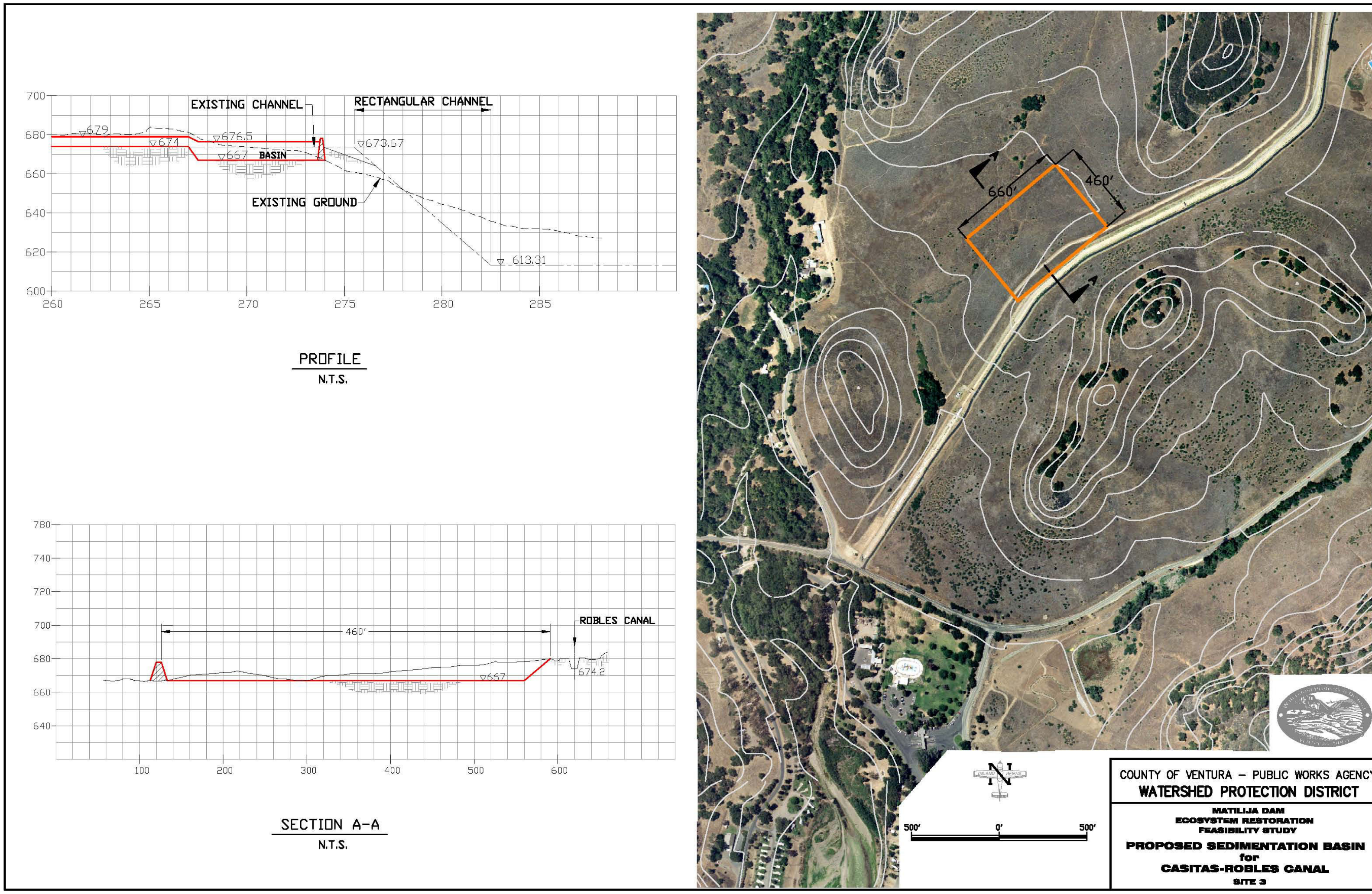


Figure 22.5. Site 3 Drawing.

23. Exhibit K. Conceptual Design and Benefit of Increasing Robles Canal Capacity

If the capacity of Robles Canal is increased, the risk to CMWD's water supply is reduced. As an example, the increase in water supply was calculated assuming a canal capacity of 750 cfs. The longest drought period on record was the period from 1944 to 1964. To calculate the increase in water supply, it was assumed that additional diversion would occur any time the flow in the Ventura River was greater than 500 cfs. The increase in water supply was approximately 20,000 ac-ft during the period 1944 to 1964. This equates to approximately 1,000 ac-ft per year. Therefore, it is estimated that the safe yield could be increased almost 1,000 ac-ft per year if the capacity of Robles Canal is increased to 750 cfs. The increase in canal capacity would not significantly affect the sediment transport within the Ventura River. Most of the sediment transport occurs at flow much larger than 750 cfs.

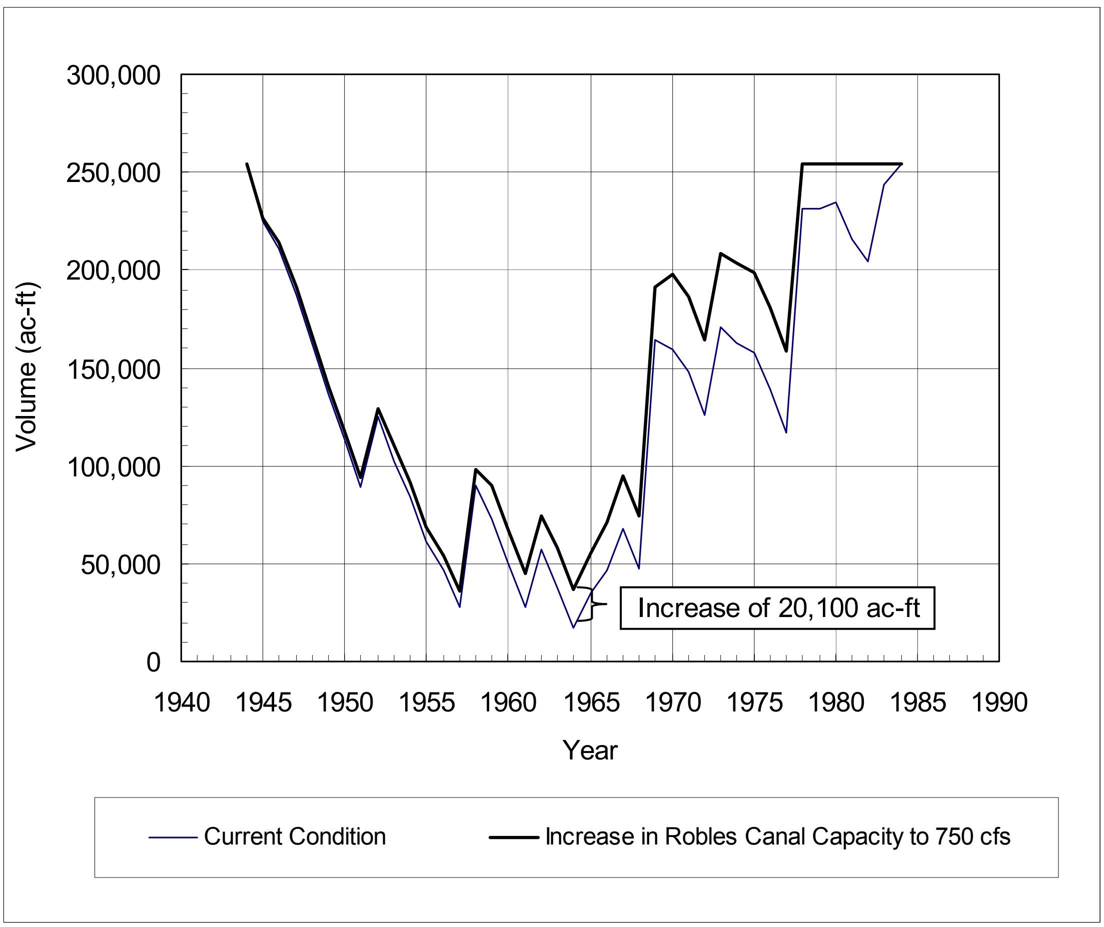


Figure 23.1. Estimated increased in water supply due to an increase in the capacity of Robles Canal to 750 cfs.

24. Exhibit L. Sensitivity of Alternative 2a Impacts to Changes in Numerical Model

Several more model runs were performed to determine the sensitivity of the results to changes in these model parameters. The 100-yr flood was used as the hydrology input. The duration of the flood was 200 hours. Results would be shown for two 100-yr floods in succession. The sensitivity analysis was done only for Alternative 2a.

24.1. Sensitivity to Transport Formula

The results using two different transport formulas were compared. One formula was described in Section 8.3.4 “Non-Cohesive Sediment Transport Parameters”. This is a combination of the Wilcock bed-load function and the Englund-Hansen Total Load function. The other is a combination of Meyer-Peter-Muller (MPM) and Englund-Hansen. The Meyer-Peter-Muller formula was used to compute the bed-load for particles larger than 2 mm and the Englund-Hansen formula was used for particles smaller than 2 mm.

The results for the erosion from the reservoir are shown in Figure 24.1 using the Wilcock Formulation and Figure 24.2 using the MPM formulation. For Alternative 2a, the total volume of sediment remaining after mechanic removal of the reservoir fine sediment is estimated to be 3.9 million yd^3. The MPM formulation predicts approximately 2.0 million yd^3 of sediment removed during the first flood and an additional 1.0 million yd^3 during the second. The Wilcock formulation predicts approximately 1.9 million yd^3 of sediment removed during the first flood and an additional 0.8 million yd^3 during the second. Most of the difference is due to the sand erosion, because the MPM formulation does not take armoring effects into account, it allows more sand to erode.

The deposition after two 100-yr floods in succession is shown in Figure 24.3. The MPM formula gives much more deposition in the upper reaches. The maximum deposition reached 9 feet using the MPM formula, versus 5 feet for the Wilcock formula. This is most likely a result from the fact that more sediment is eroded using the MPM formula. As a comparison, the maximum deposition predicted by the analytical method of Section 7 titled “Analytical Modeling of the ” is also plotted in Figure 24.3. The analytical method generally predicts less aggradation upstream of RM 14 and more aggradation downstream. However, the calculated deposition is generally within the upper and lower estimates of deposition as predicted by the analytical model.

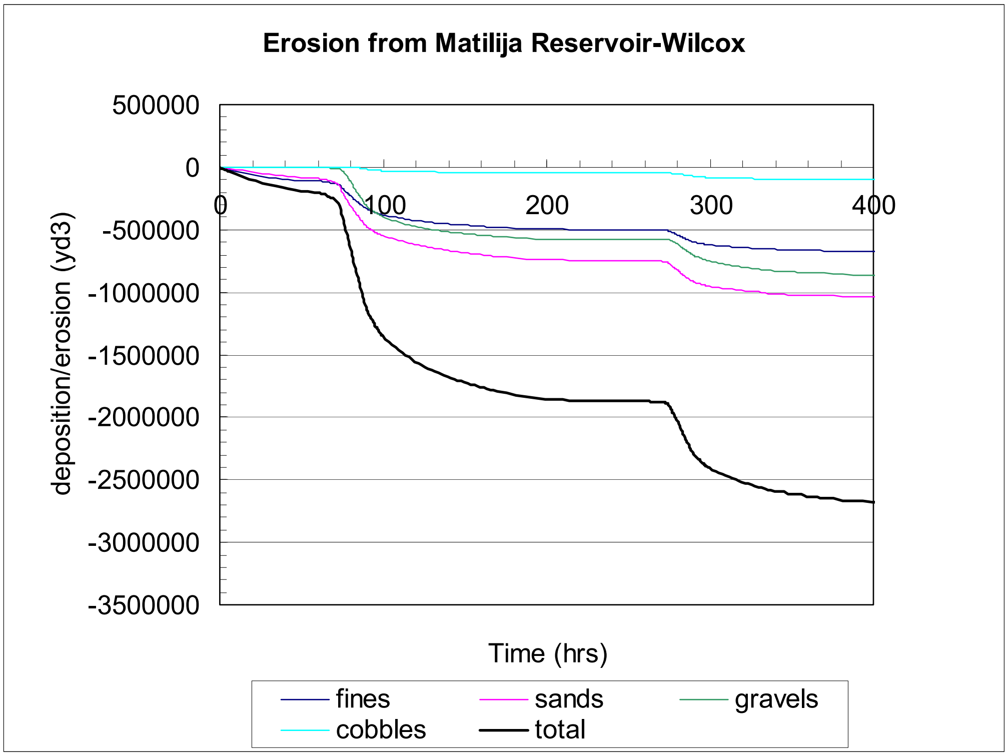


Figure 24.1. Results for the erosion of Matilija Sediment using Wilcock formula.

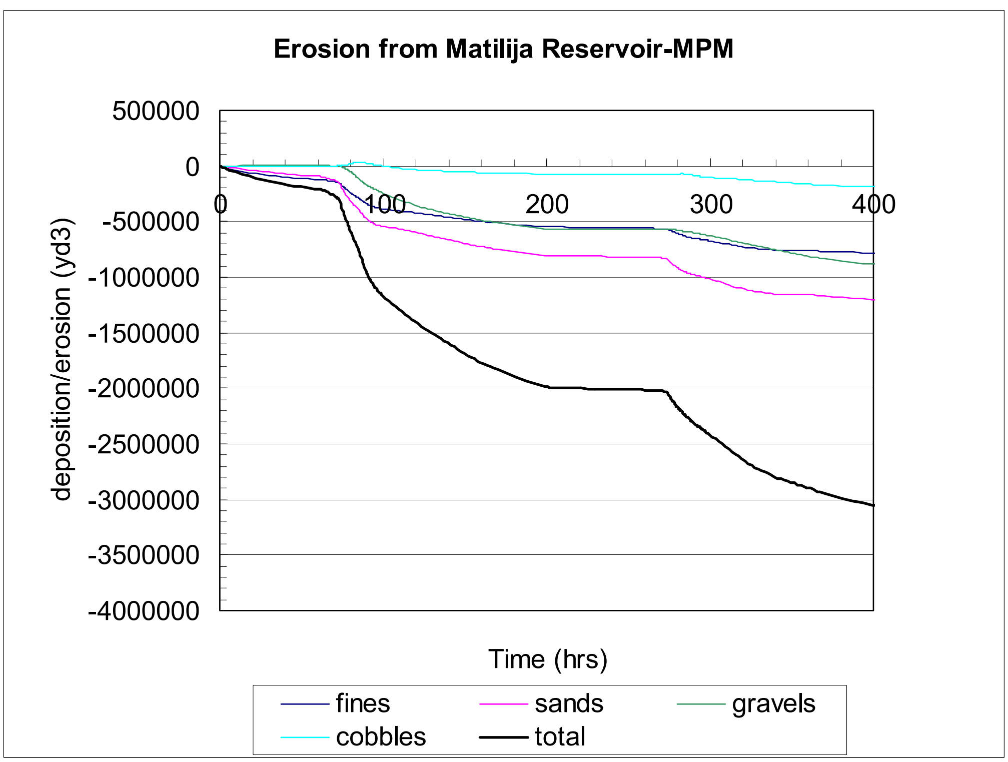


Figure 24.2. Results for the erosion of Matilija Sediment using MPM formula.

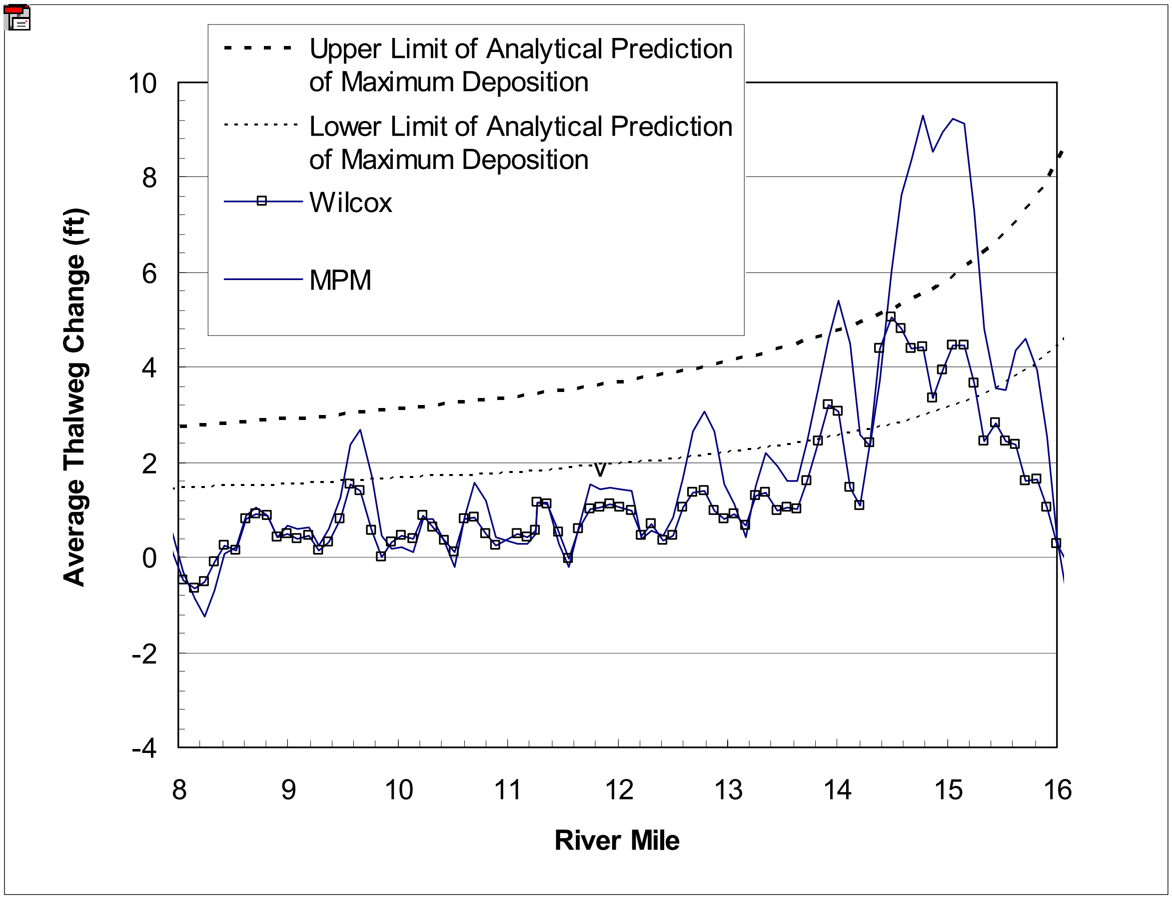


Figure 24.3. Comparison of deposition results between the Wilcock formula and the MPM formula after two 100-yr floods in succession.

24.2. Sensitivity to Manning's Roughness Coefficient

The sensitivity to the Manning's Roughness Coefficient was also studied. The roughness coefficient was increased by 20 % and decreased by 20%. The model results show that the deposition reaches immediately downstream of Matilija Dam is sensitive to the changes in the Manning's n value. However, beyond RM 12, the results were not sensitive to the Manning Coefficient. Greater roughness generally decreases the transport capacity of the channel, and that is the reason that deposition increases when the roughness coefficient increases. The deposition downstream of RM 12 is unaffected because the sediment that is going to deposit, deposits upstream.

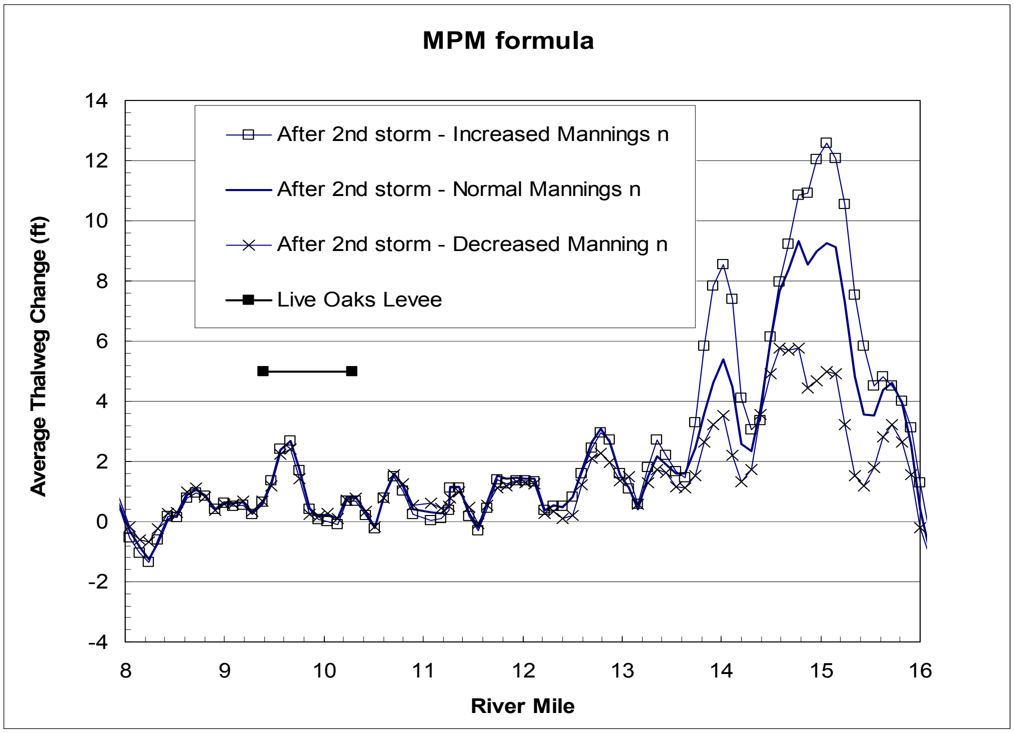


Figure 24.4. Sensitivity of depositions to changes in the Manning Roughness Coefficient using the MPM formula.

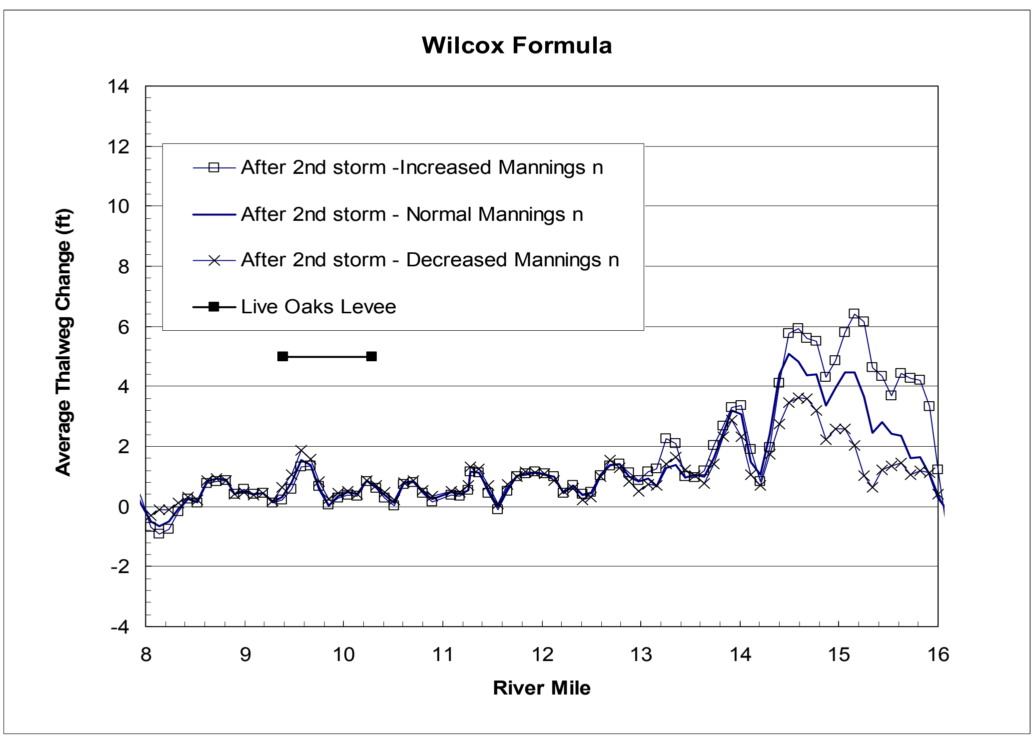
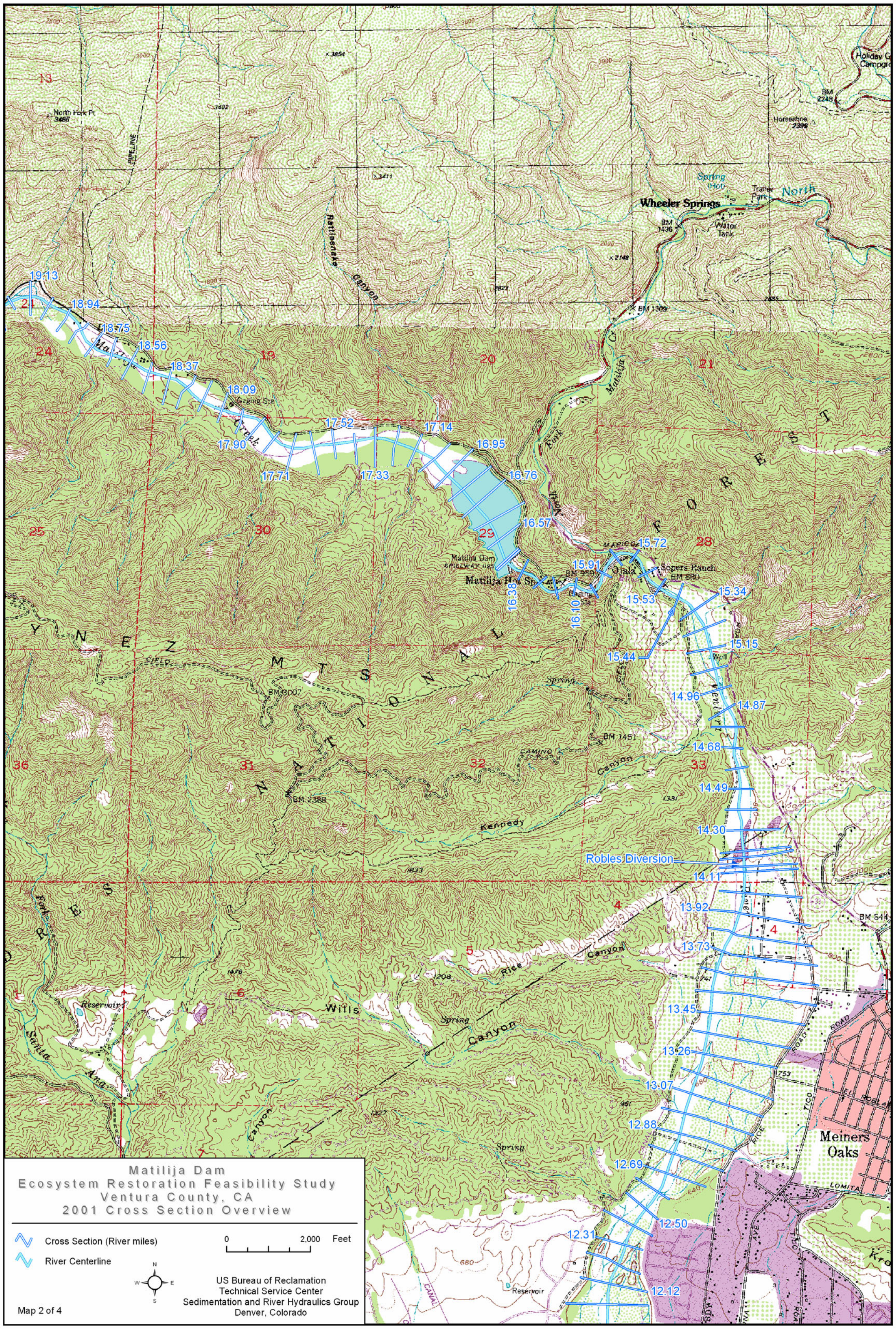
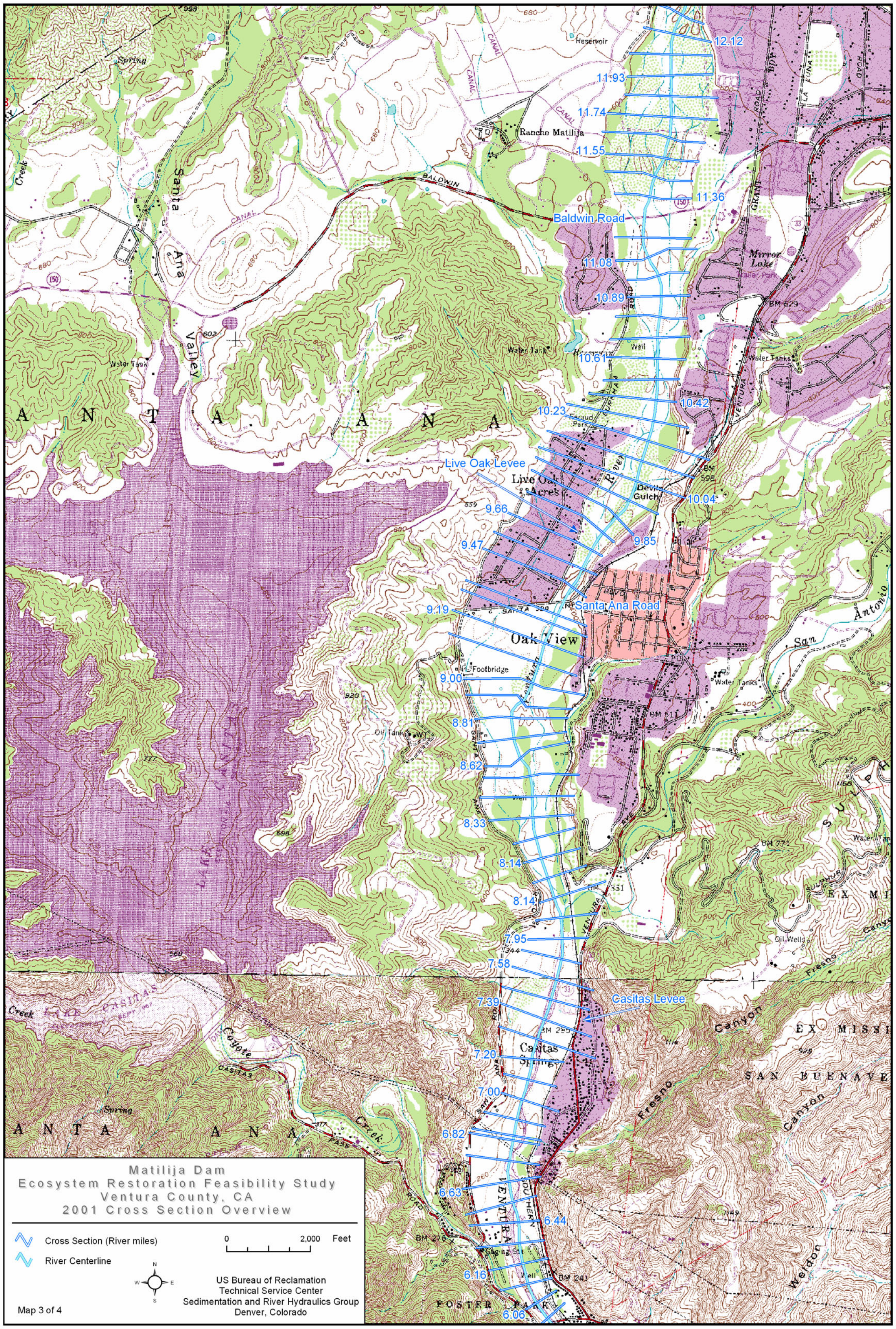


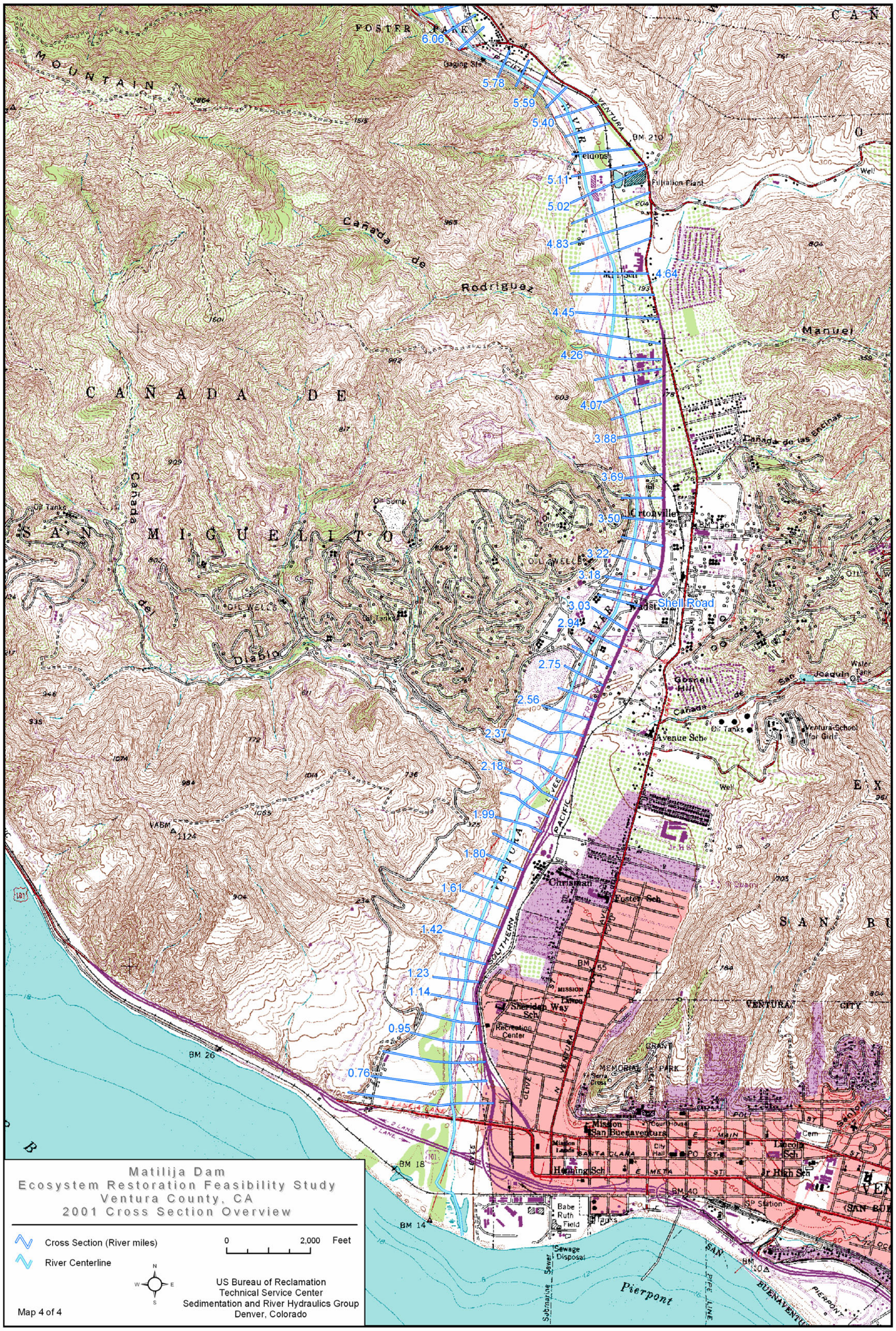
Figure 24.5. Sensitivity of depositions to changes in the Manning Roughness Coefficient using the Wilcock formula.

25. Exhibit M. Location of Cross Section Used in Study


Matilija Dam
Ecosystem Restoration Feasibility Study
Ventura County, CA
2001 Cross Section Overview
Cross Section (River miles)
River Centerline
0
2,000
Feet
US Bureau of Reclamation
Technical Service Center
Sedimentation and River Hydraulics Group
Denver, Colorado
Map 1 of 4
22.26
22.06
21.78
21.59
21.40
21.21
21.02
20.83
20.64
20.45
20.27
20.08
19.89
19.70
19.51
19.33
19.13
18.94
Middle
Matilija Campsite
Matilija
Campsite
North Fork Pt
Upper
North
Fork
Creek
Matilija
Canyon
White Ledge
Peak
S A N T A Y N
A I N S
West Fk Santa Ana Cr


Matilija Dam
Ecosystem Restoration Feasibility Study
Ventura County, CA
2001 Cross Section Overview
Cross Section (River miles)
River Centerline
0
2,000
Feet
US Bureau of Reclamation
Technical Service Center
Sedimentation and River Hydraulics Group
Denver, Colorado
Map 2 of 4
Wheeler Springs
Robles Diversion
Meiners Oaks
19.13
18.94
18.75
18.56
18.37
18.09
17.90
17.71
17.52
17.33
17.14
16.95
16.76
16.57
16.38
16.10
15.91
15.72
15.53
15.44
15.34
15.15
14.96
14.87
14.68
14.49
14.30
14.11
13.92
13.73
13.45
13.26
13.07
12.88
12.69
12.50
12.31
12.12


Matilija Dam
Ecosystem Restoration Feasibility Study
Ventura County, CA
2001 Cross Section Overview
Cross Section (River miles)
River Centerline
0
2,000
Feet
US Bureau of Reclamation
Technical Service Center
Sedimentation and River Hydraulics Group
Denver, Colorado
Map 3 of 4
12.12
11.93
11.74
11.55
11.36
Baldwin Road
11.08
10.89
10.61
10.42
10.23
Live Oak Levee
10.04
9.85
9.66
9.47
Santa Ana Road
9.19
9.00
8.81
8.62
8.33
8.14
8.14
7.95
7.58
Casitas Levee
7.39
7.20
7.00
6.82
6.63
6.44
6.16
6.06
Rancho Matilija
Oak View
Casitas Springs
Mirror Lake
Live Oak Acres
Devils Gulch


Matilija Dam
Ecosystem Restoration Feasibility Study
Ventura County, CA
2001 Cross Section Overview
Cross Section (River miles)
River Centerline
0
2,000
Feet
US Bureau of Reclamation
Technical Service Center
Sedimentation and River Hydraulics Group
Denver, Colorado
Map 4 of 4
6.06
5.78
5.59
5.40
5.11
5.02
4.83
4.64
4.45
4.26
4.07
3.88
3.69
3.50
3.22
3.18
3.03
2.94
2.75
2.56
2.37
2.18
1.99
1.80
1.61
1.42
1.23
1.14
0.95
0.76
Shell Road

26. Exhibit N. Description of Historical Channel Morphology Data

In addition to the location of active-channel sections and segments for the Ventura River in 1947, 1970, and 2001, a number of descriptive or measured attributes were compiled in the GIS for each section. Compilation was done in an attributes table linked to the SectionLoc1947, SectionLoc1970, and SectionLoc2001 GIS layers. The attributes in each of the three tables are:

Section Name (Sect_Name): The letter designation of the section. Beginning at the estuary, the sections are labeled in an upstream direction A through Z, then Aa through Zz, Aaa through Zzz, and Aaaa through Yyyy (table 2 [Excel file "GIS_writeup_Table2"]).

River Mile (Riv_Mile): The location of the section, in number of miles upstream from the mouth of the Ventura River, as measured along the 2001 thalwag.

Segment Name (Seg_Name): For section that contain more than one active channel, each active-channel segment of the section was assigned a name by attaching a sequential number to the section name, starting with the segment at the right bank (for example, section Gg, sections Gg1, Gg2, and Gg3).

Bank-to-Bank Width (Bank_Bank): Total width, in feet, between the two ends of a section line.

Active Width (Active_Wth): The width, in feet, of the active channel. If the section contains more than one active channel (segment), the active width is the sum of the widths of the segments.

Segment Width (Seg_Width): The width, in feet, of the individual segment named in the "Seg_Name" field.

Channel Form (Chan_Form): General geomorphic descriptive phrase for the section as a whole (not for individual segments). (Examples: Straight channel; Channels and bars; Channels, bars, islands.)

Right Bank Material (R_Bank): Brief description of the right-bank material (and (or) vegetation) as interpreted from the aerial photograph. If the section contains more than one active channel (segment), the description pertains to the right bank of the individual segment named in the "Seg_Name" field.

Left Bank Material (L_Bank): Brief description of the left-bank material (and (or) vegetation) as interpreted from the aerial photograph. If the section contains more than one active channel (segment), the description pertains to the left bank of the individual segment named in the "Seg_Name" field.

Confidence in Active Channel Determination (Confids): A scale of 1 to 3, with 1 being best, representing a subjective rating of the confidence in determining the boundaries of the active channel for the individual section, based on interpretation of the aerial photograph.

Remarks (Remarks): Comments regarding uncertainties in identifying the boundaries of the active channel, or more detailed description of one or more of the attributes.

Table 26.1. Table Describing Select Available Photography of Ventura River.

Flood Year/ Peak Flow (cfs)	Year	Photo Dates	Remarks
1938 Mar 2/ 39,200	1939	**1939 Jan 17**	Full set of photos
1943 Jan 22/ 35,000		**1945 Oct, Nov**	Additional peaks 20,000 (1944); 17,000 (2/2/1945) Matilija Dam built 1947
	1947	(1947 Sep 13)	Full set of photos
1952 Jan 15/ 29,500	1953	(1953 Jan 05)	(Matilija Dam); (also coast and N. Fork Matilija) No intervening peak flows over 18,700 (1958)
		1965 Jun 09	Matilija Dam and Reservoir, downstream to Casitas
1969 Jan 25/ 58,000	1969	1969 Jan 29	Jan. 29-30 combined = coast to Matilija Reservoir
		1969 Jan 30	Jan. 29-30 combined = coast to Matilija Reservoir
		1969 Feb 16	Ventura R., Ventura Mission to Matilija Hot Springs
1969 Feb 25/ 40,000	1969	**1969 Feb 26**	Full set of photos (minus dam & S. end of Matilija Creek)
		1970 Jan 30	Ventura R., from coast to Matilija Cr confluence
1978 Feb 10/ 63,600	1978	1978 Feb 14	Full set of photos
		1978 Mar 06	Full set of photos
1980 Feb 16/ 37,900	1980	1980 Feb 24	First bridge upstrm of Hwy 101 to Matilija Reservoir Lower priority. Does not include estuary.
1983 Mar 01/ 27,000	1983	1983 Mar 04	Hwy 101 to upper end of Matilija Reservoir Does not include estuary.
1987 Mar 06/ 22,100	1987		Matilija Creek watershed fire, July 1985 (Have July and September 1985 Matilija Creek photos.) No photo sets between 1983 and 1992.
1992 Feb 12/ 45,800	1992	**1992 Mar 18**	N. of estuary to Matilija Cr upstream to half of reservoir
		1994	USGS digital orthophotos
1995 Jan 10/ 43,700	1995	**1995 Jan 15**	Ventura R., from coast to Matilija Cr
	1998	1998 Feb 12	Ventura R., from coast to Matilija Cr (after 1st peak)
1998 Feb 23/ 38,800	1998	**1998 Mar 10**	Ventura R., from coast to Matilija Cr (after last peak)
	2001	**2001 Sep 09**	Ventura R estuary to start of Matilija Cr Upper N Fork