RECLAMATION

Managing Water in the West

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

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

November 2006

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Photo on cover is a postcard taken at approximate location of current dam.

Executive Summary

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1. Introduction

The Bureau of Reclamation (Reclamation) is providing technical assistance in the Matilija Dam Ecosystem Restoration Project -- a cost-shared project between the Corps of Engineers (Corps) and Ventura County Flood Control District (District). The Corps requested Reclamation to perform the hydrology, hydraulics, and sedimentation analyses. 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).

Included in this report are results from the hydrology, hydraulic and sediment studies for the Matilija Dam Ecosystem Restoration Project.

1.1. General River and Watershed Description

The Ventura River Watershed is shown in Figure 1.2. 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 1.1. 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 1.3 and Figure 1.3). 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 1.4. There are eight 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.

An Aerial view of Matilija Dam taken from a Helicopter in April of 2004 is given in Figure 1.4 and a Figure 1.5 shows an oblique view of the Upper Ventura River downloaded from Google Earth in November 2006.

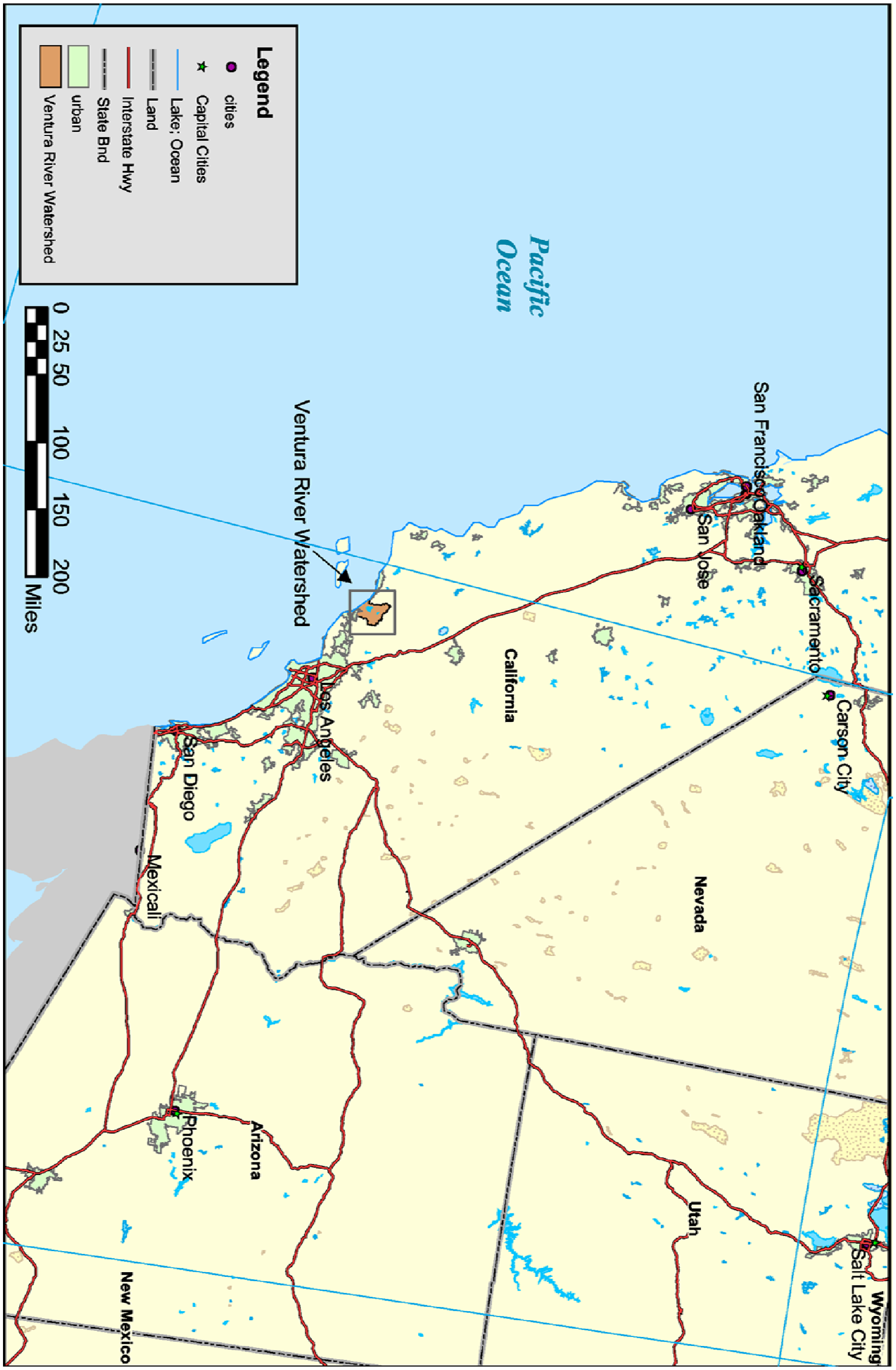


Figure 1.1. Location Map of Ventura Basin.

Figure 1.2. Map of Ventura River Basin.

Table 1.1. 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

Table 1.2. Minor Drainages in the Ventura River Watershed

Map Name	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)
Minor Drainages East of Ventura River						
E1	1st drainage N. of Cozy	0.73	7547	835.45	3608.26	22.1

	Dell Canyon					
E2	Cozy Dell Canyon	1.97	16294	776.15	4351.22	20.2
E3	1st drainage S. of Cozy Dell Canyon	0.24	5059	727.90	1713.96	19.5
E4	MacDonald Canyon	1.12	9968	713.66	2047.84	19.5
E5	Local Drain S. of Meiners Oaks	1.38	11226	644.82	1051.63	19.0
E6	Local Drainage in Mira Monte area	1.30	10952	536.12	846.50	17.6
E7	1st drain S. of Mira Monte	1.35	14270	416.46	908.23	17.6
E8	Oakview area local drainage	0.95	11028	358.57	928.11	17.0
E11	Fresno Canyon	1.26	13268	297.79	1327.06	17.6
E12	Weldon Canyon	2.19	16082	204.90	1159.35	16.9
E13	Manuel Canyon	1.14	12069	185.29	1005.45	16.9
Minor Drainages West of Ventura River						
W1	Kennedy Canyon	1.30	12035	804.91	3210.23	20.2
W2	Rice Canyon	0.73	6839	733.87	1788.03	19.5
W3	Wills Canyon	1.38	12291	642.29	2371.87	19.5
W4	1st drainage S. of Wills Canyon	0.40	4802	617.16	1307.60	19.0
W5	Rancho Matilija area drainage	2.32	14363	433.26	1307.60	17.6
W6	Live Oak drainage from NW	0.26	3448	430.73	960.94	17.0
W7	Cañada de Rodriguez	1.27	9373	184.14	2059.51	16.9
W8	Cañada del Diablo	5.21	22762	78.62	2121.83	16.9

Table 1.3. Major Reaches of Matilija Creek and the Ventura River.

Reach #	River Mile	Reach
8	30 – 17.64	Matilija Creek
7b	17.64 – 16.58	Matilija Delta
7a	16.58 – 16.31	Matilija Reservoir
6b	16.31 – 15.0	Downstream of Matilija Dam to Canyon opening
6a	15.0 – 14.0	From Canyon opening to upstream of Robles Diversion
5	14.0 – 11.11	Near Robles Diversion to Baldwin Road Bridge
4	11.11 – 7.86	Baldwin Road Bridge to San Antonio Creek Confluence
3	7.86 – 5.95	San Antonio Creek Confluence to Foster Park Bridge
2	5.89 – 0.54	Foster Park Bridge to Main St Bridge
1	0.54 – 0.0	Estuary

Table 1.4. Landmarks along River.

Landmark	River Mile
Upstream End of Matilija Reservoir Delta	17.64
Upstream End of Matilija Reservoir	16.58
Matilija Dam	16.31
Matilija Creek confluence with N. Fork Matilija Creek	15.67
Camino Cielo Bridge	15.37
Los Robles Diversion Dam	14.01
Baldwin Road	11.11
End of Live Oak Levee	10.15
Beginning of Live Oak Levee	9.25
Santa Ana Blvd	9.25
Confluence of Ventura River and San Antonio Creek	7.86
End of Casitas Levee	7.67
Beginning of Casitas Levee	6.50
Foster Park Diversion	6.3
Confluence of Ventura River and Coyote Creek	6.2
Casitas Vistas Road (USGS stream gage)	5.89
Ojai Valley Sanitation District Waste Treatment Plant	5.0
Confluence of Ventura River and Cañada Larga	3.56
Shell Road	3.08
End of Ventura River Levee	2.31
Main Street	0.54
Ventura Freeway (Highway 101)	0.39
Southern Pacific Railroad	0.17
Beginning of Ventura River Levee	0
Ventura River Mouth	0

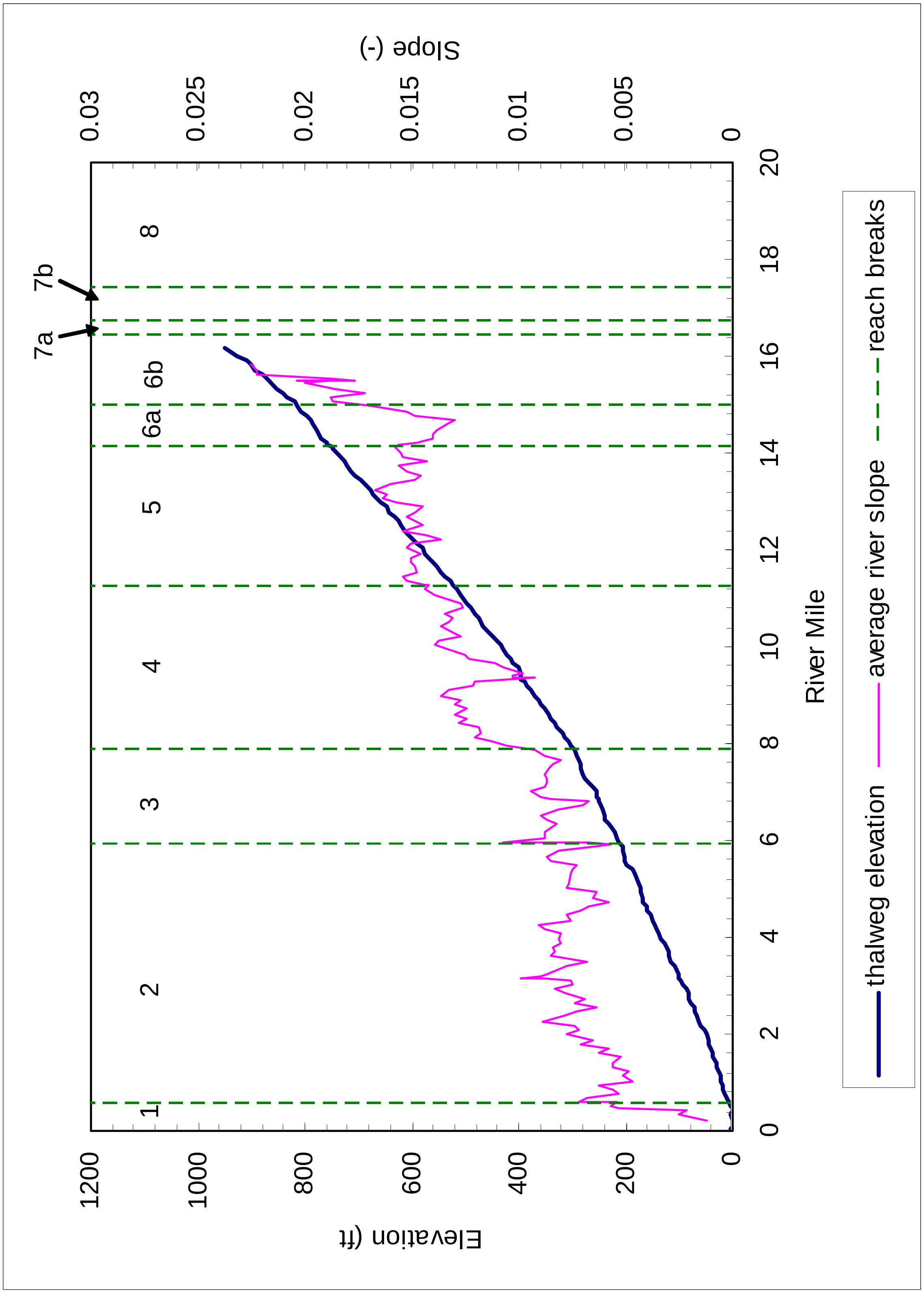


Figure 1.3. Bed Profile and Reach Definitions in the Ventura River.

Figure 1.4. Aerial View of Matilija Dam (taken April 2004).

Figure 1.5. Oblique View of Matilija Dam and Upper Ventura River (From Google Earth, 2006).

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 1.1. 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.

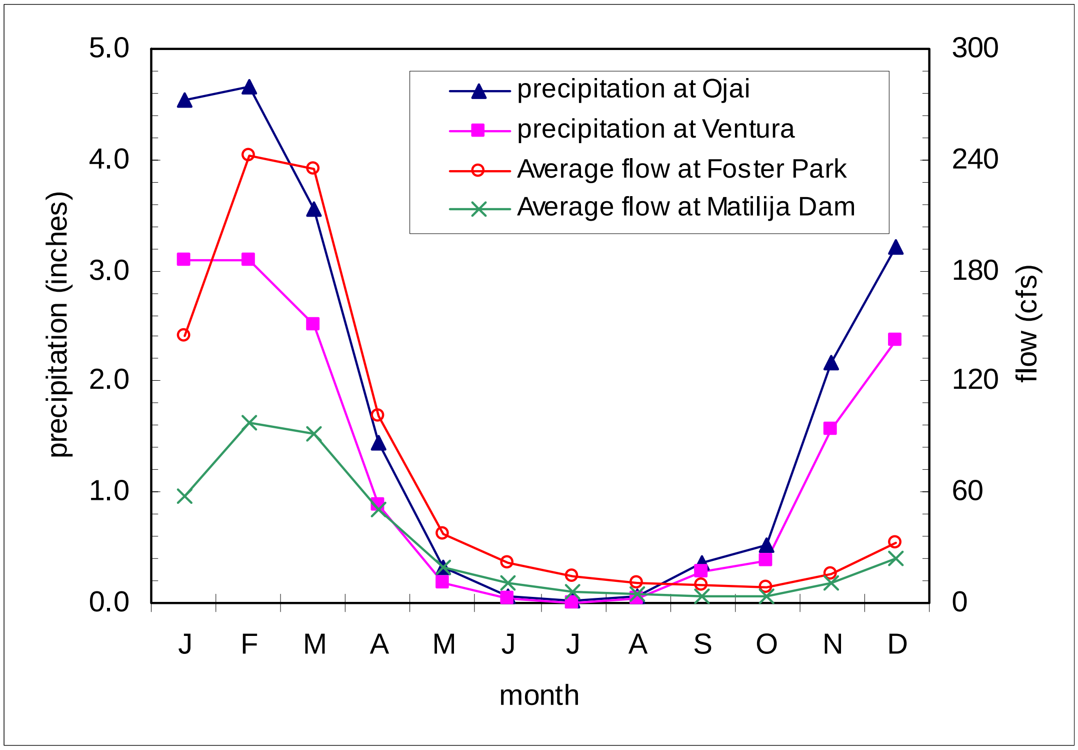


Figure 1.6. Seasonal variation of average rainfall and flow in Ventura River Watershed.

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.

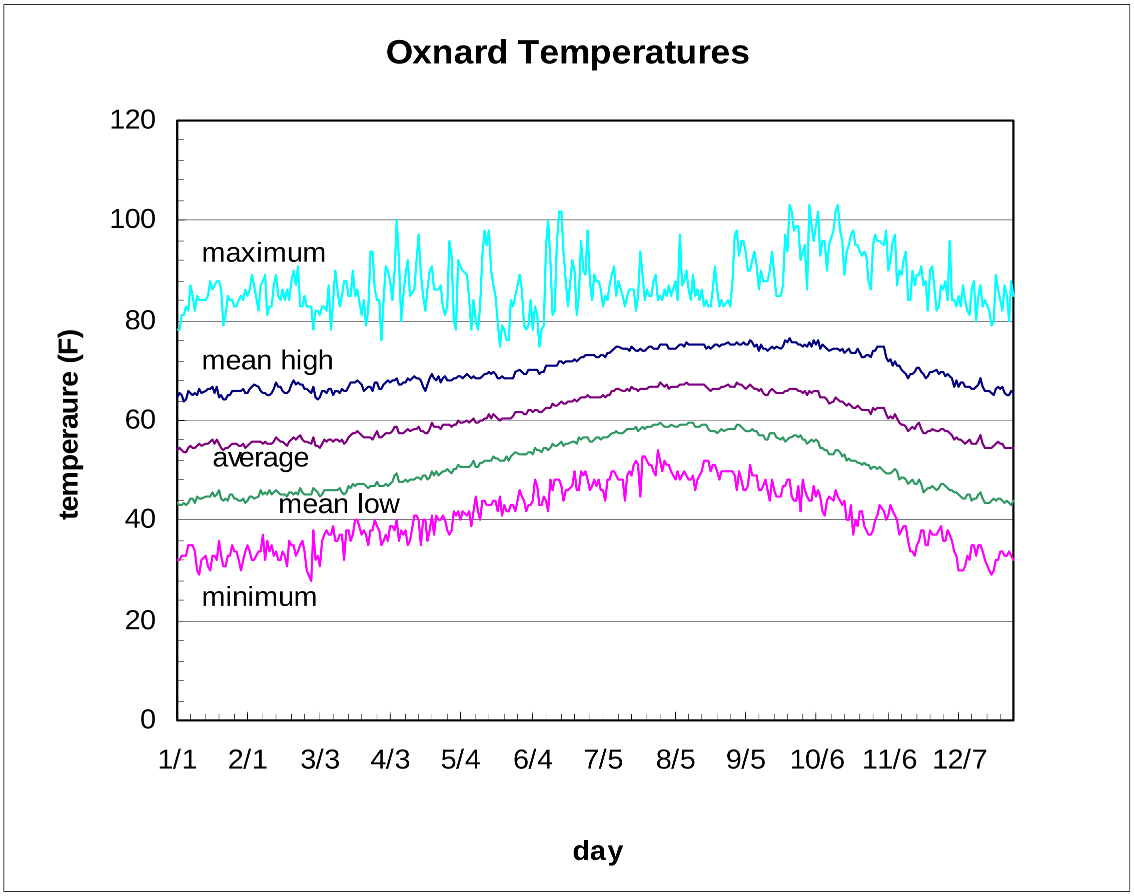


Figure 1.7. Temperatures at Oxnard (34º 11' 00" N 119º 10' 00"W, El. 49 ft) for the period of 1948 to 2000.

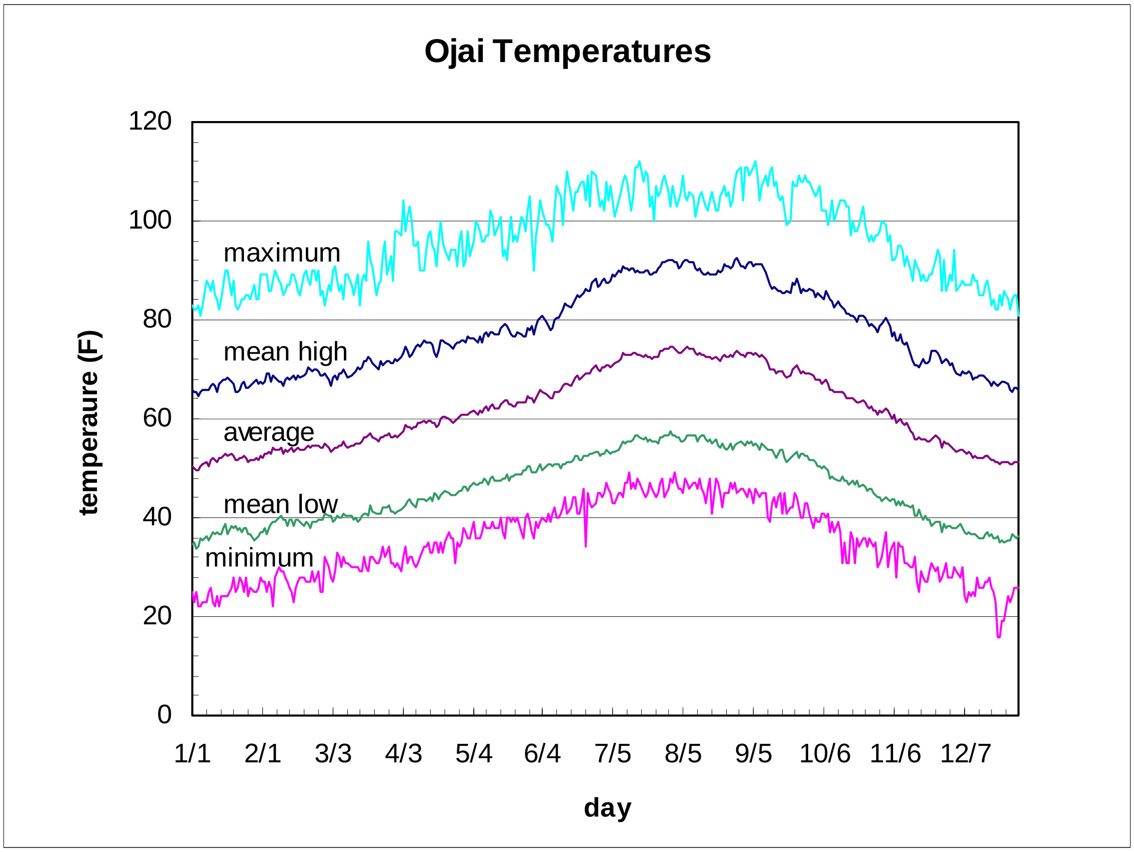


Figure 1.8. Temperatures at Ojai (34º 26' 00"N, 119º 13' 00"W, El. 750 ft) for the period 1948 to 2000.

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 it 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% based upon analysis shown in Section 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.

A report summarizing the safe yield from Casitas Reservoir is found in Entrix (2002). According to that report, 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.

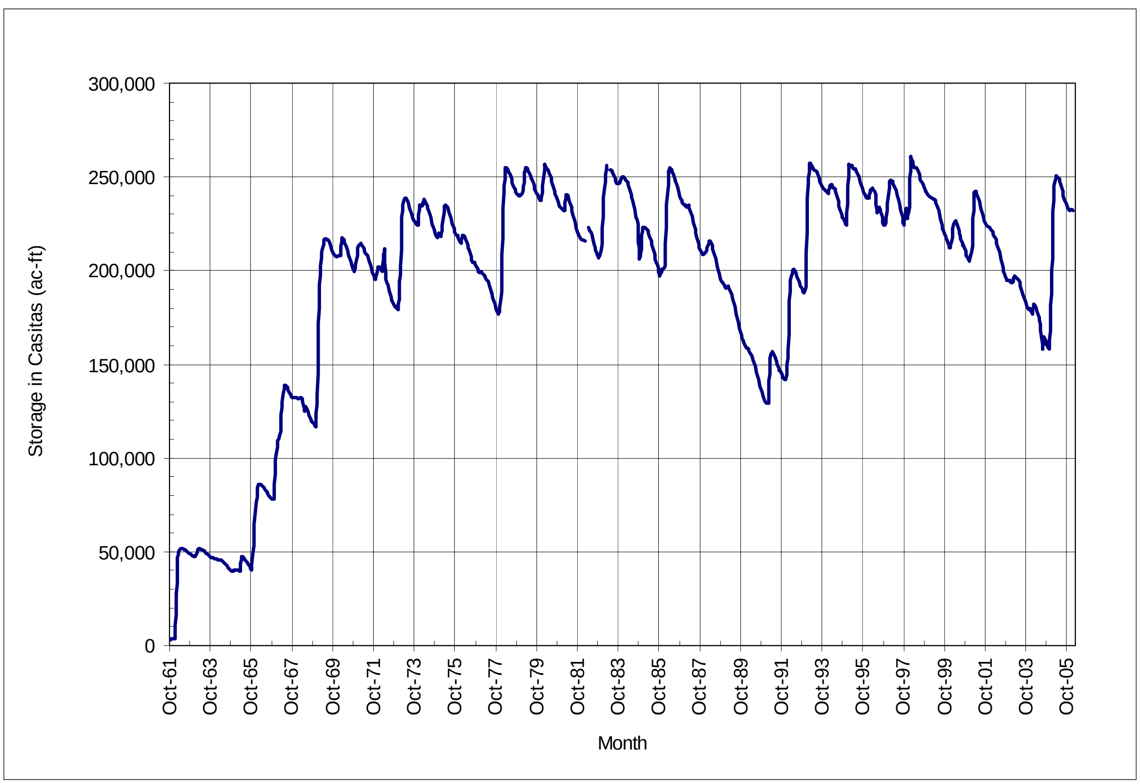


Figure 1.9. Storage in Lake Castitas.

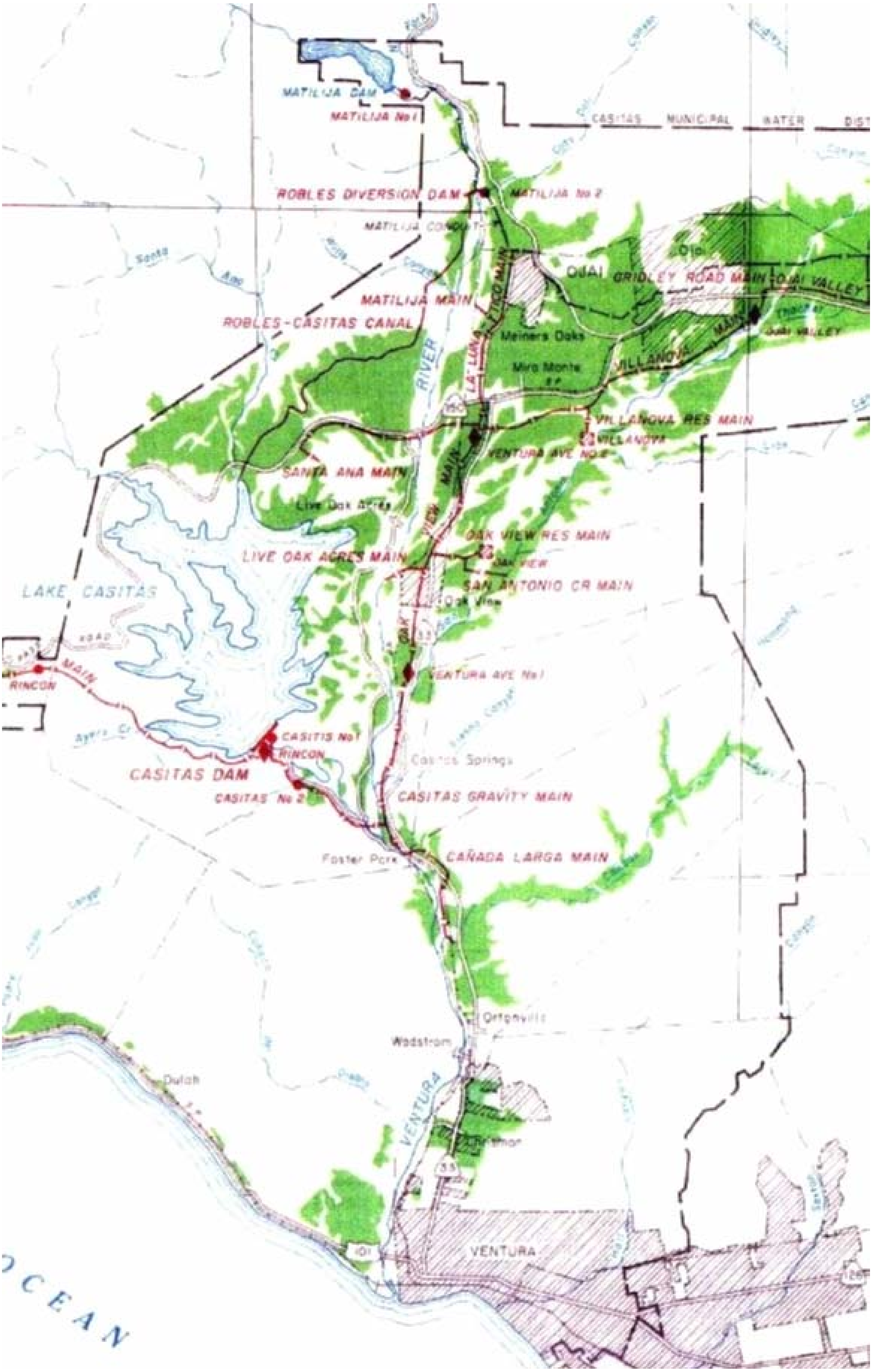


Figure 1.10. Schematic of the Ventura River Project (from Reclamation web site, http://dataweb.usbr.gov/).

Robles Diversion Dam

Robles Diversion Dam was built in 1958 and it 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. 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 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 was completed in the fall of 2005. An aerial view is given in Figure 1.11 showing the ladder under construction. A photograph of the fish screens is shown in Figure 1.12.

Robles Diversion is subject to large 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 1.5. Significant sediment removal is necessary after every major flood. CMWD 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 removal 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 1.5. 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 Fish Passage Facility under Construction in April 2004.

Figure 1.12. Fish Screens and Cleaning Brushes in Robles Canal.

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 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 1.13. The surface diversion has not been used since 2000 because the river shifted and abandoned the channel leading to the surface diversion.

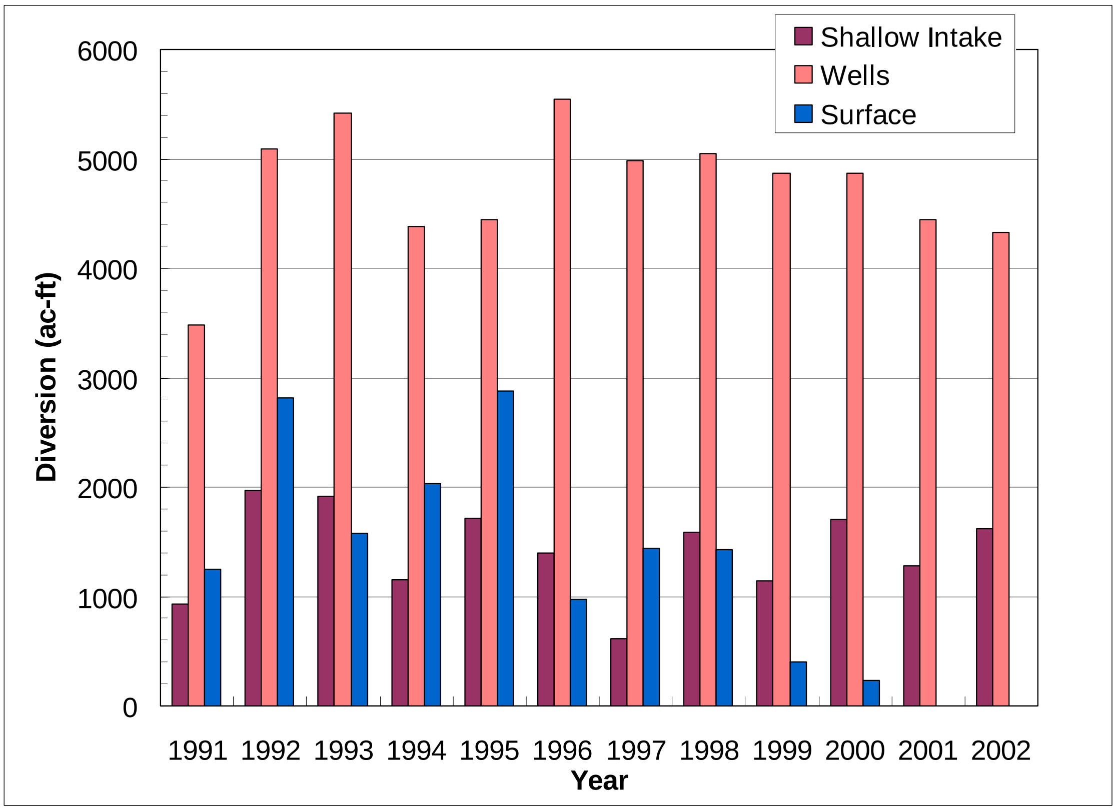


Figure 1.13. Annual Surface Diversions at Foster Park on the Ventura River.

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 1.14.

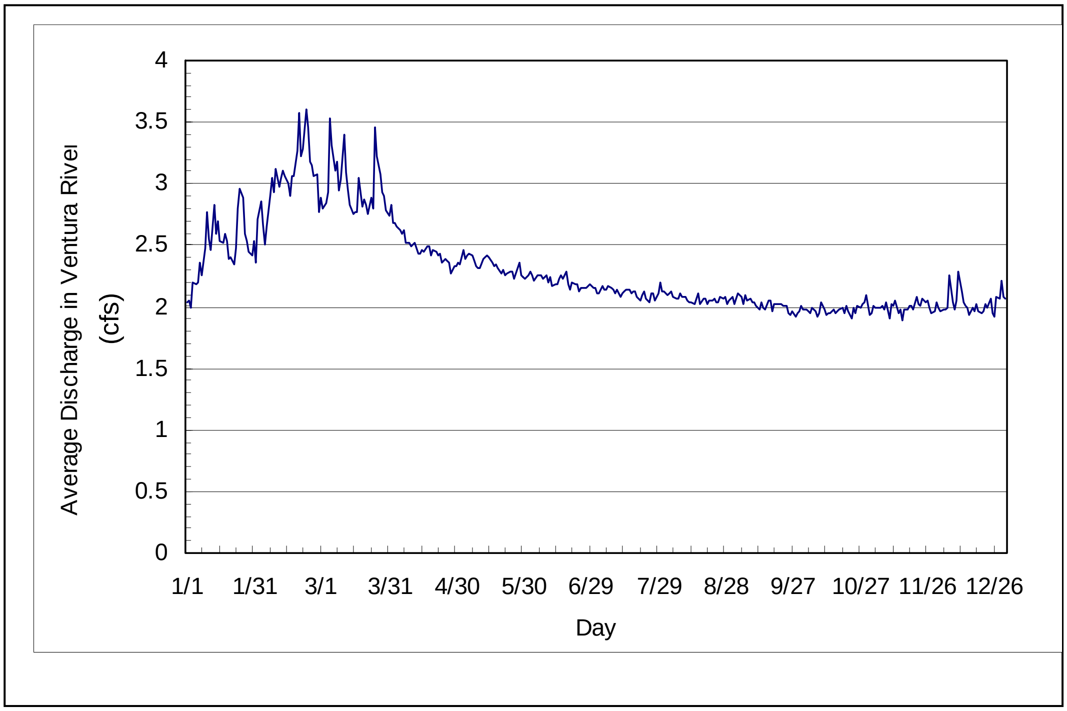


Figure 1.14. Average Discharge of the OVSD Wastewater Treatment Plant for Each Day of the Year.

1.4.3. LEVEES

There are three major levees along the Ventura River and their characteristics are shown in Table 1.6. 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 1.6. Levee Characteristics along the Ventura River.

Levee	**Ventura**	**Casitas Springs**	**Live Oak**
Year Constructed	1947	1978	1995
Downstream River Mile (mi)	0	6.50	9.25
Upstream River Mile (mi)	2.31	7.67	10.15
Downstream Elevation (ft)	14.4	270	412.2
Upstream Elevation (ft)	120.0	310	465.5

1.4.4. DEBRIS BASINS

There are four debris basins in the Ventura River watershed and their properties are listed in Table 1.7. McDonald and Dent Canyons are direct tributaries to the Ventura River, while Stewart Canyon is a tributary to San Antonio Creek.

Table 1.7. 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. Hydrology

This section will discuss the hydrology of the Ventura River Basin. It summarizes the information contained in the hydrology reports of Bullard (2002a, 2002b). 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 #	Drainage Area (mi^2)	Period of 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

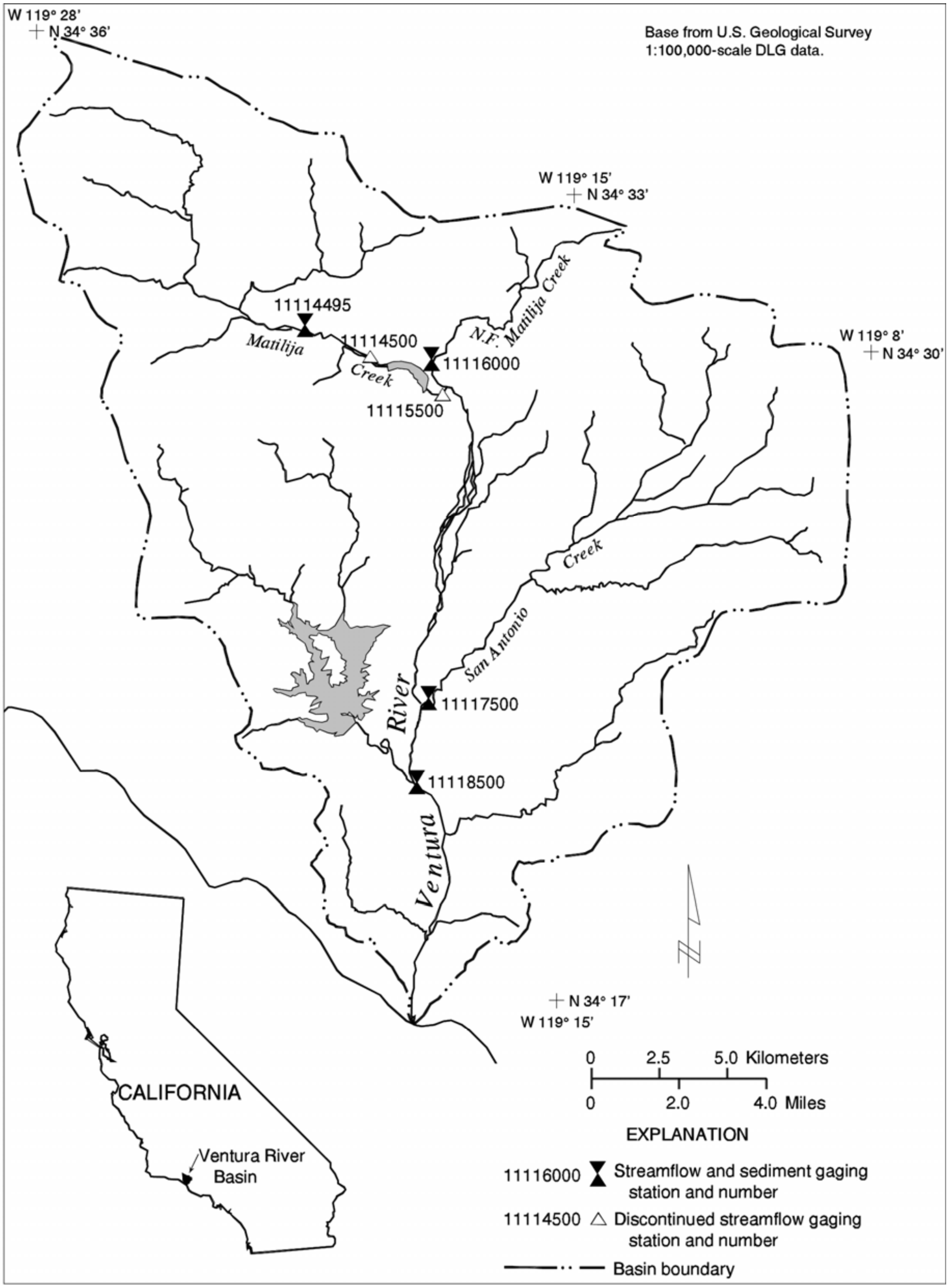


Figure 2.1. Map of Stream Gages in Ventura Watershed.

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, 2002b). 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, 2002b).

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 flow is shown in Figure 2.2 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.

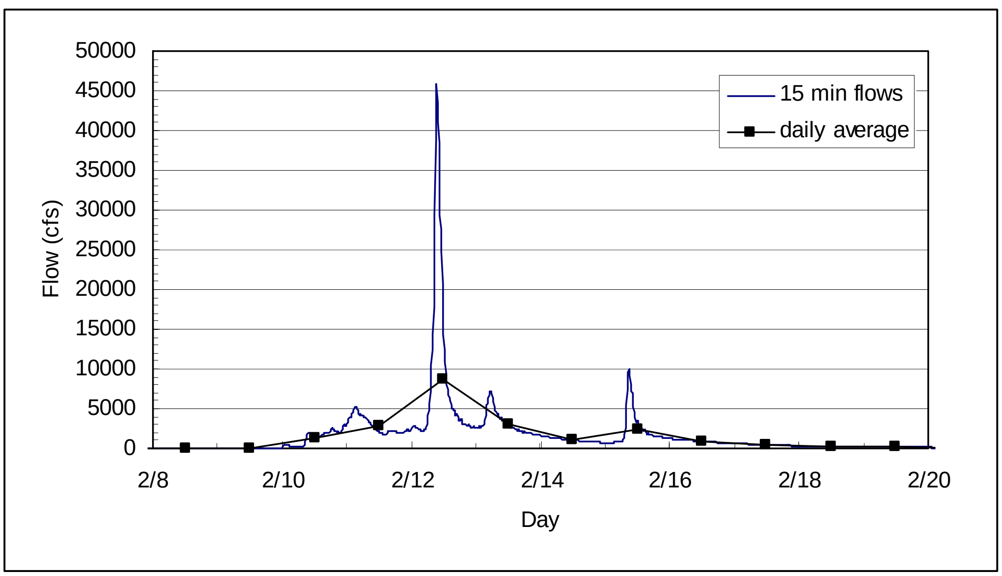


Figure 2.2. Comparison of 15-minute instantaneous hydrographs and daily average hydrographs for the 1992 flood at Foster Park gage on the Ventura River (USGS gage 11118500).

The Ventura River experiences large annual variations in peak flow magnitudes (Figure 2.3 and Figure 2.4). 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.

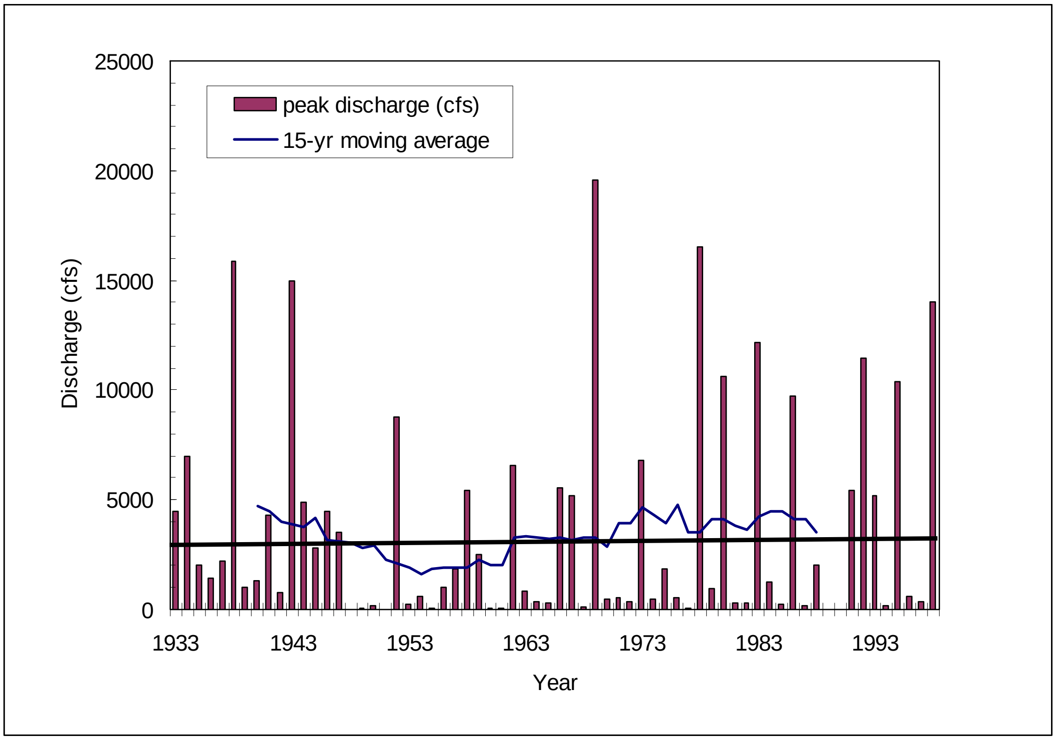


Figure 2.3. Peak Discharge at USGS gage 11115500, downstream of Matilija Dam on Matilija Creek. Flows between Oct 1 1988 and Sept 30 1990 were not available at this gage

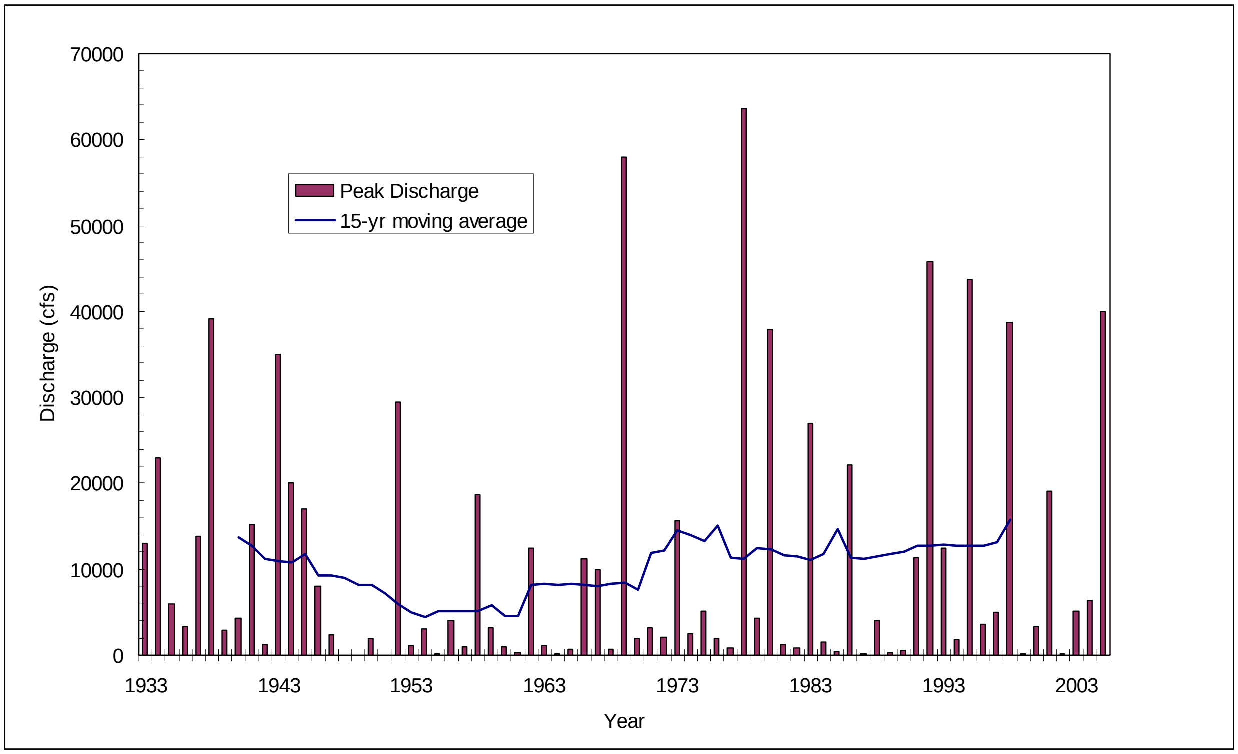


Figure 2.4. Peak Discharge at USGS gage 11118500, near Foster Park on the Ventura River.

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 2.5 and Figure 2.6, respectively. The annual volume of discharge is for the years 1928 to 2000 is shown for the same gages in Figure 2.7 and Figure 2.8. 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 greater or less than the average. A plot of mean average daily discharges is shown in Figure 2.9 and a plot of maximum average daily discharge in Figure 2.10. 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.

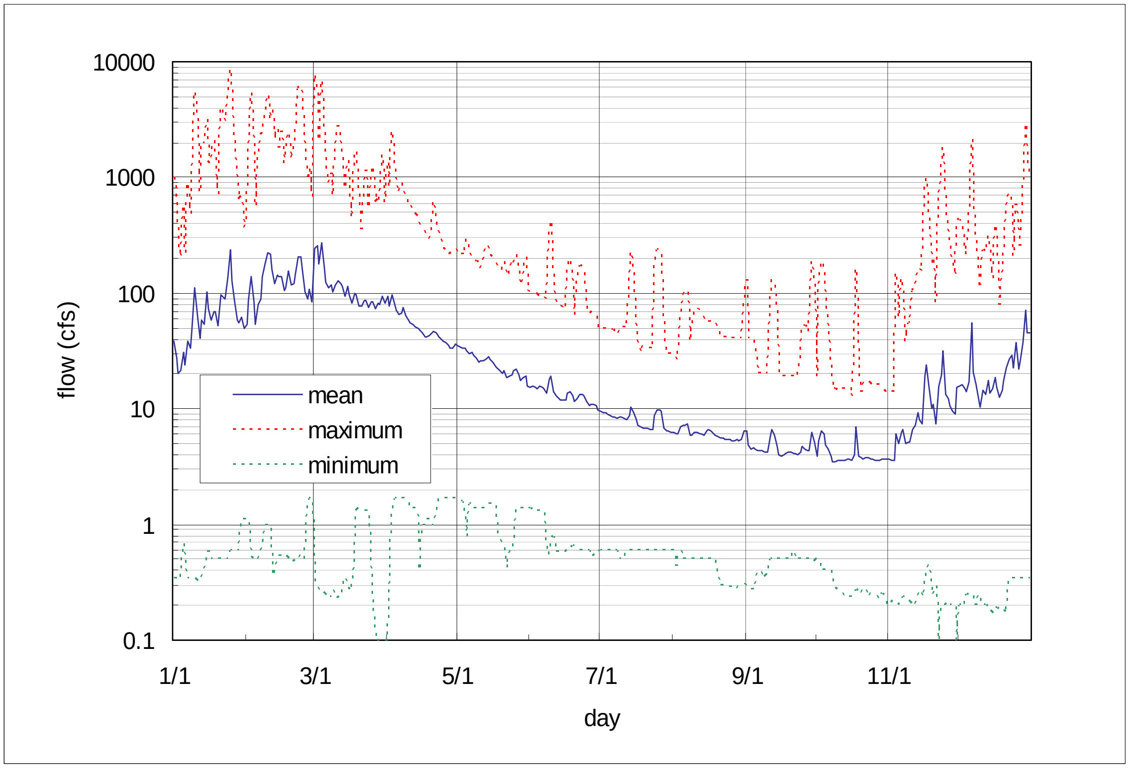


Figure 2.5. Mean, Maximum, and Minimum Daily Average Discharges for Every Day of the Year at USGS gage 11115500, downstream of Matilija Dam.

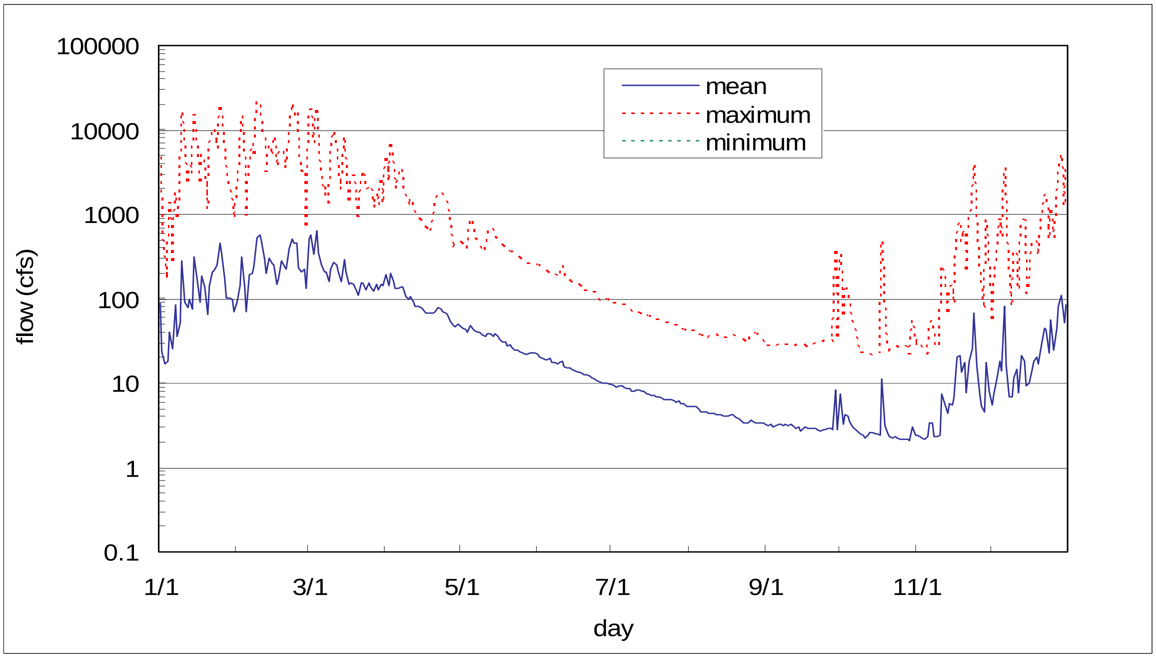


Figure 2.6. Mean, Maximum, and Minimum Daily Average Discharges for Every Day of the Year at USGS gage 11118500, Ventura River at Foster Park.

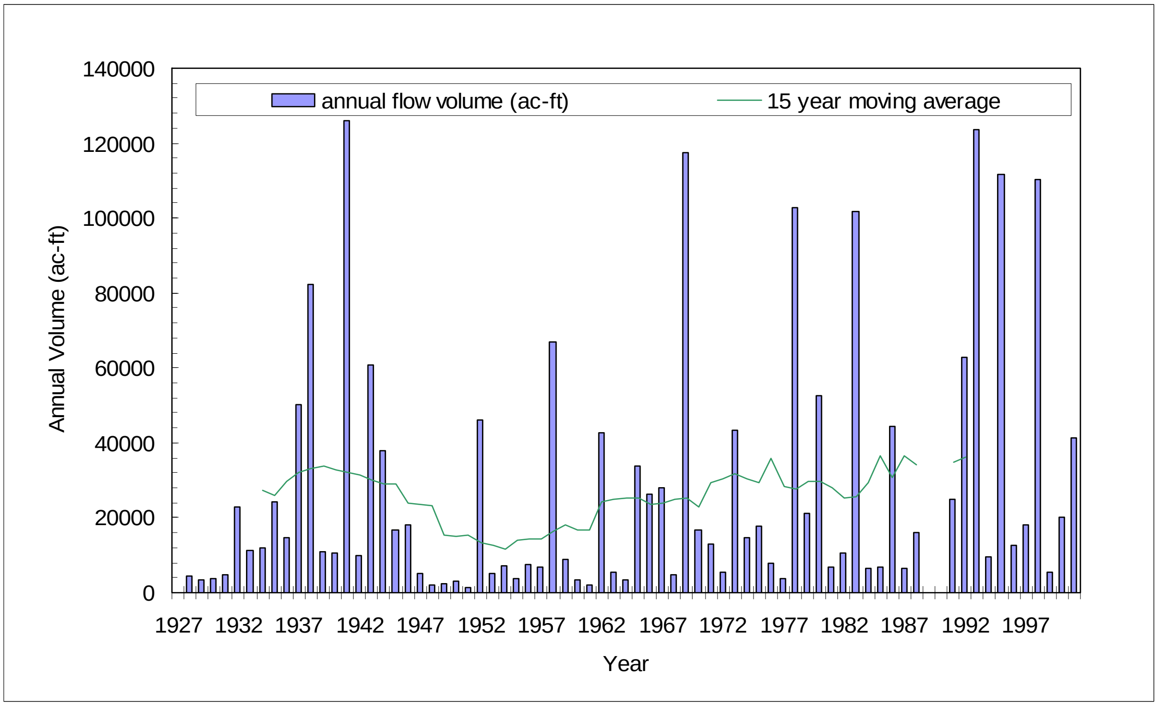


Figure 2.7. Annual flow volume at USGS gage 11115500, downstream of Matilija Dam on Matilija Creek.

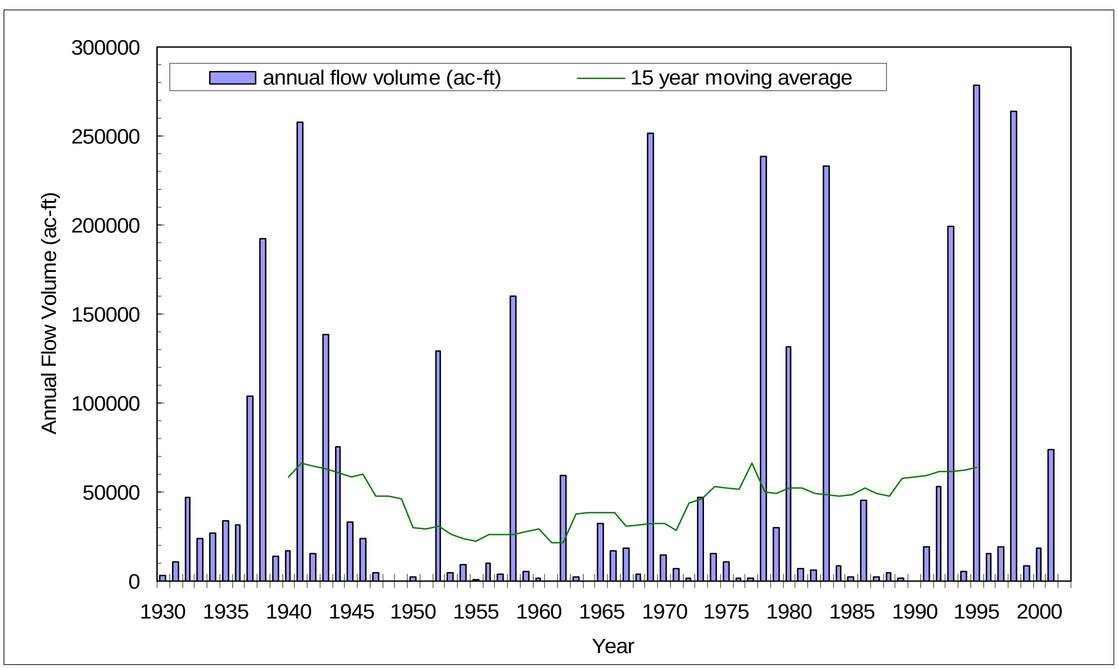


Figure 2.8. Annual flow volume at USGS gage 11118500, near Foster Park on the Ventura River.

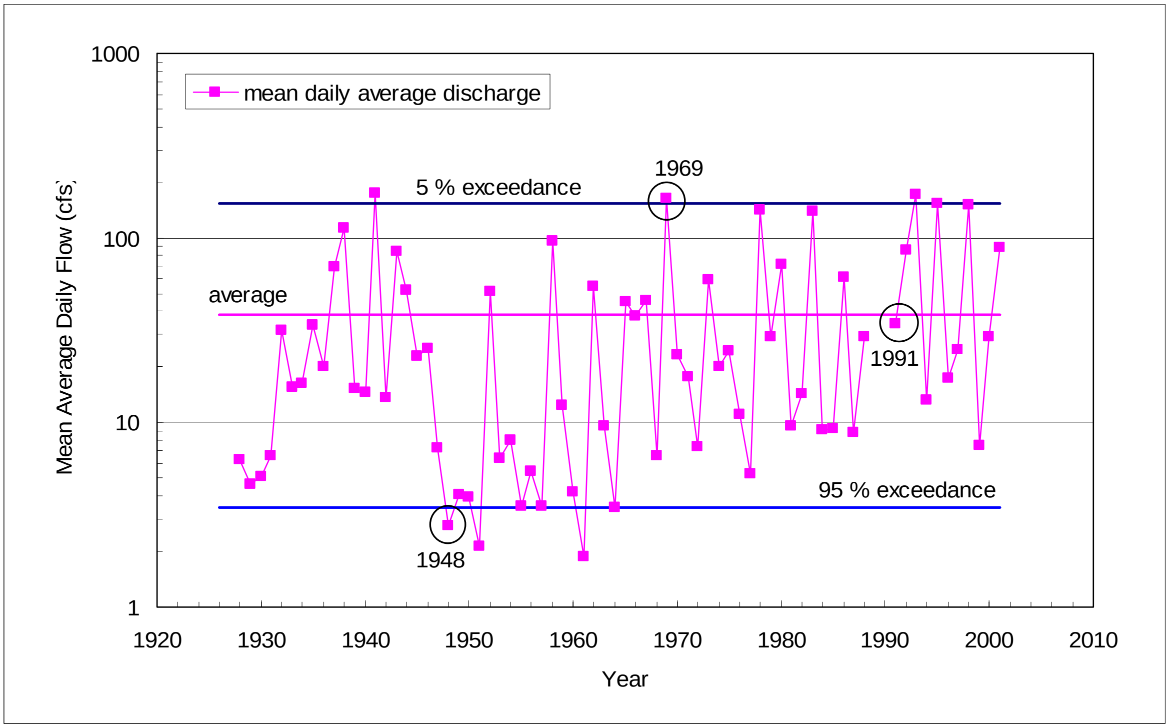


Figure 2.9. Analysis of mean daily average flows at USGS gage 11115500, downstream of Matilija Dam on Matilija Creek.

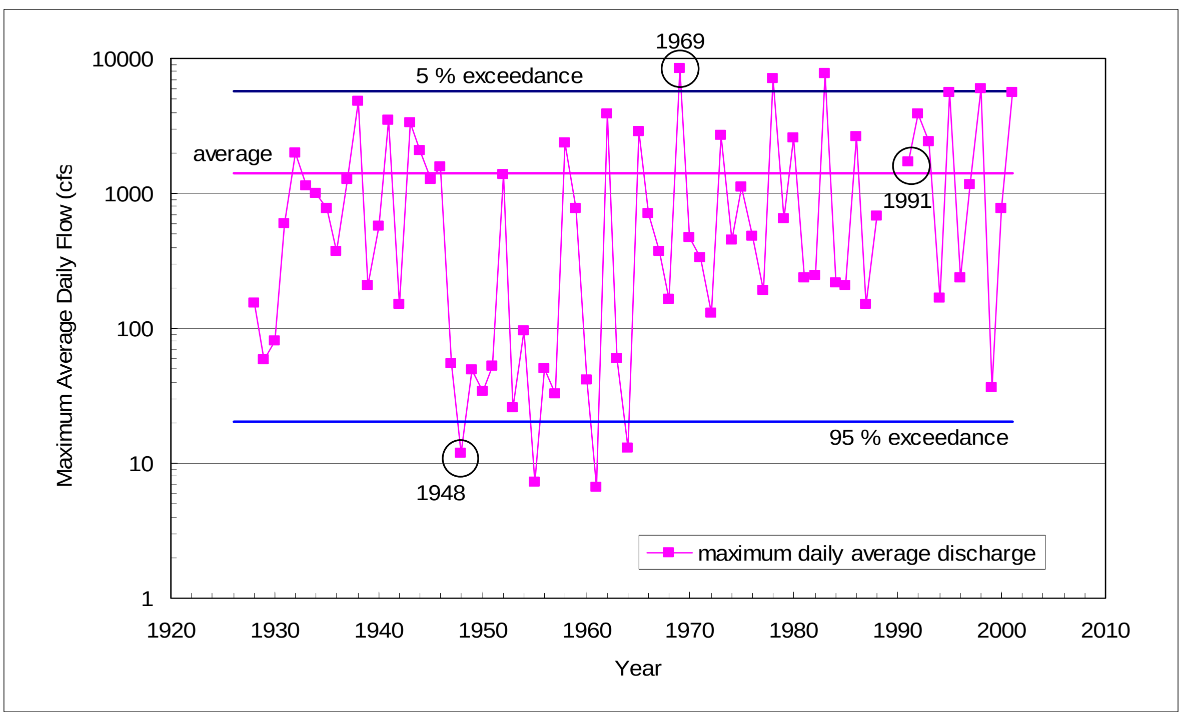


Figure 2.10. Analysis of maximum daily average flows at USGS gage 11115500, downstream of Matilija Dam on Matilija Creek.

2.2.1. FLOW DURATION CURVES

Flow duration curves were developed for the stream gages shown in Table 2.4 and Table 2.5. 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. Section 16 titled “Exhibit B. Flow Duration Curves by Month” contains flow duration curves for each month.

Table 2.4. 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 2.5. 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.

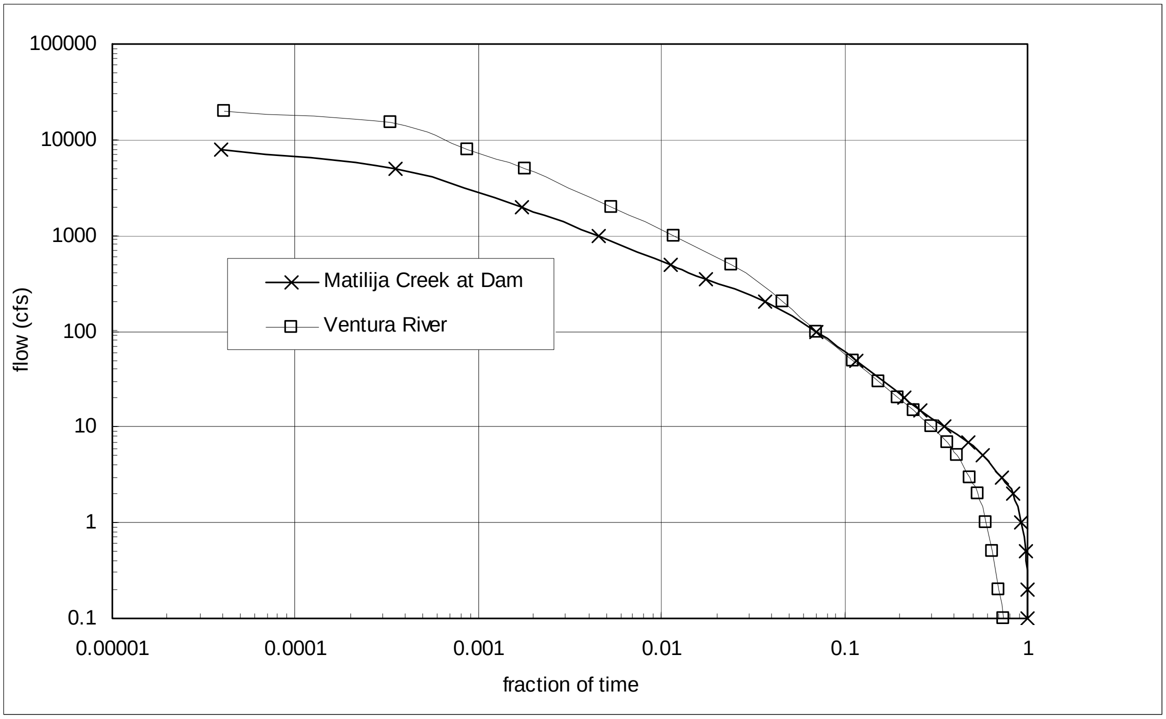


Figure 2.11. Plot of flow duration curves for USGS gage 11115500 at Matilija Dam and USGS gage 11118500, near Foster Park on the Ventura River.

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 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 2.12). 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.

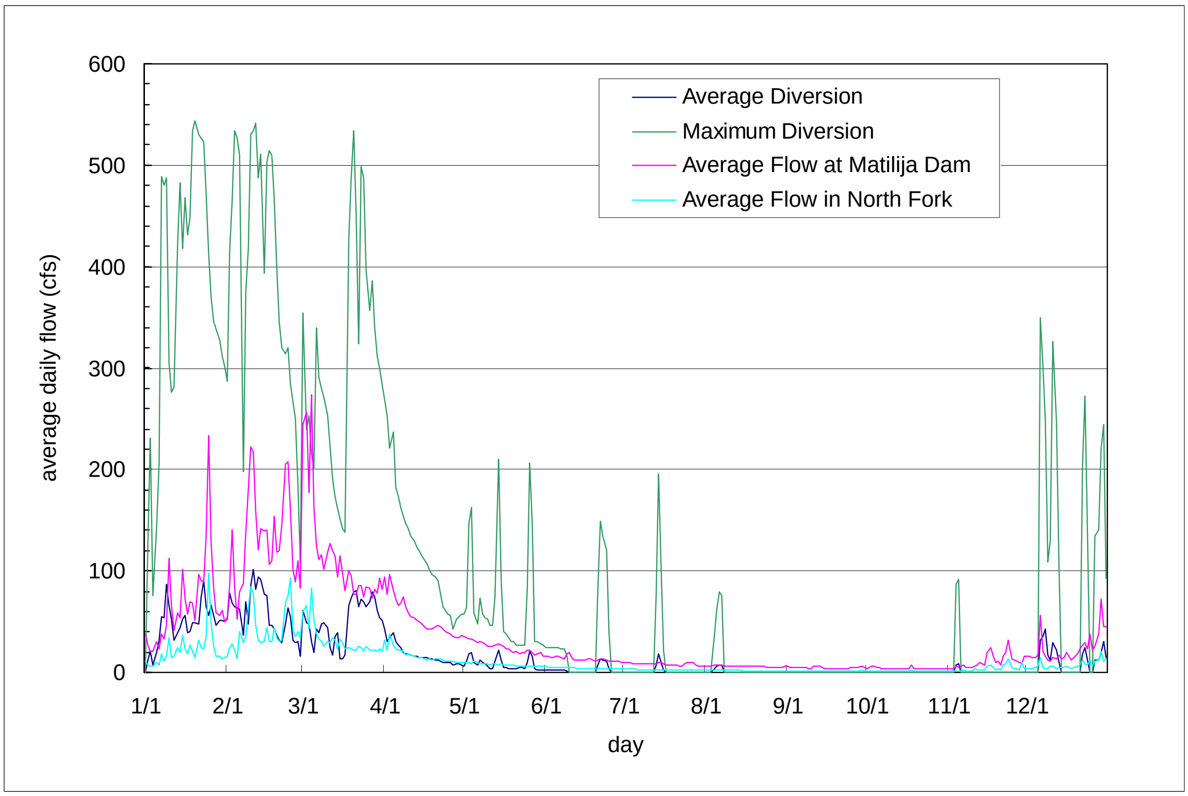


Figure 2.12. Average flow in Matilija Creek, North Fork Matilija Creek, and Robles Diversion.

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.

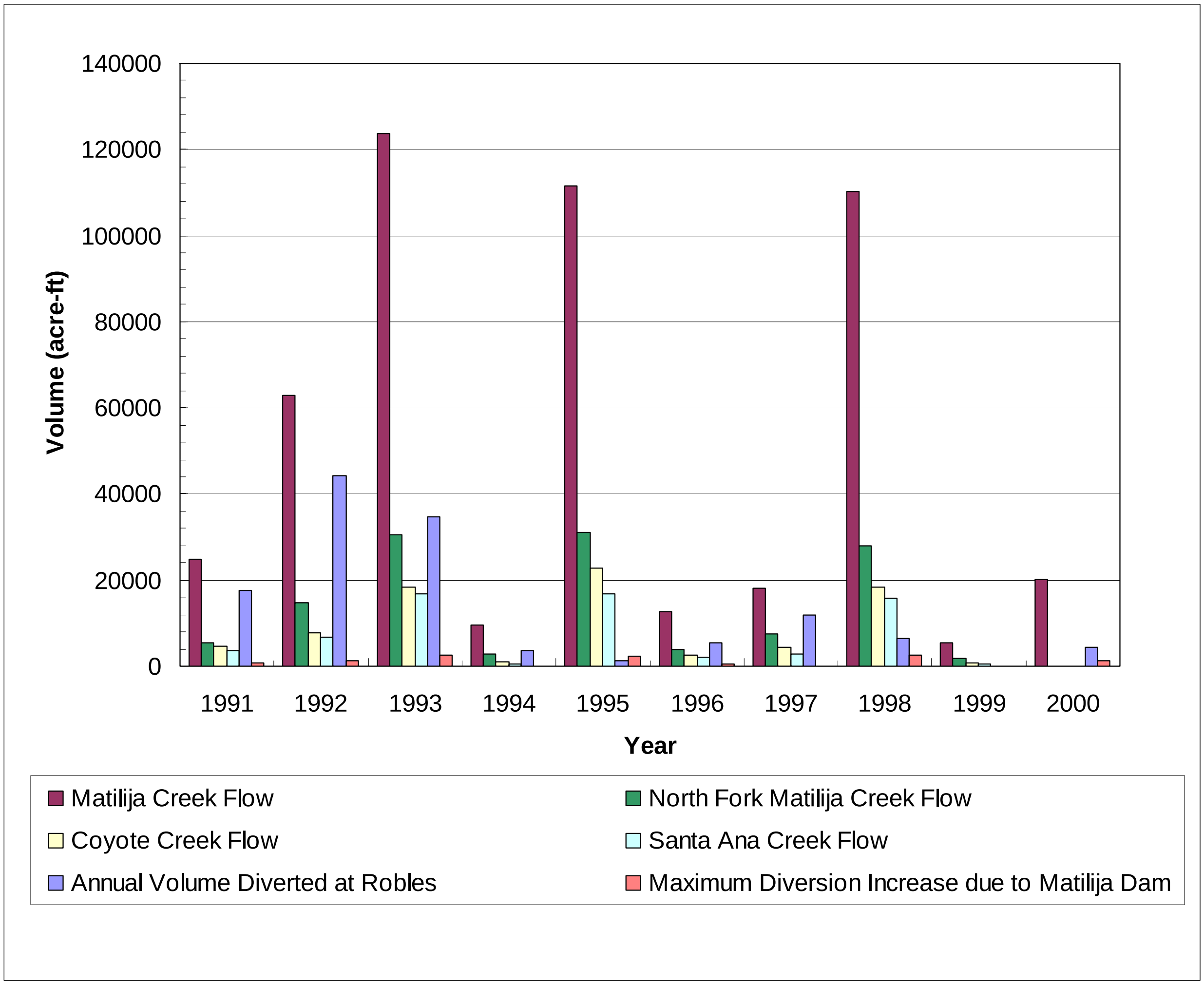
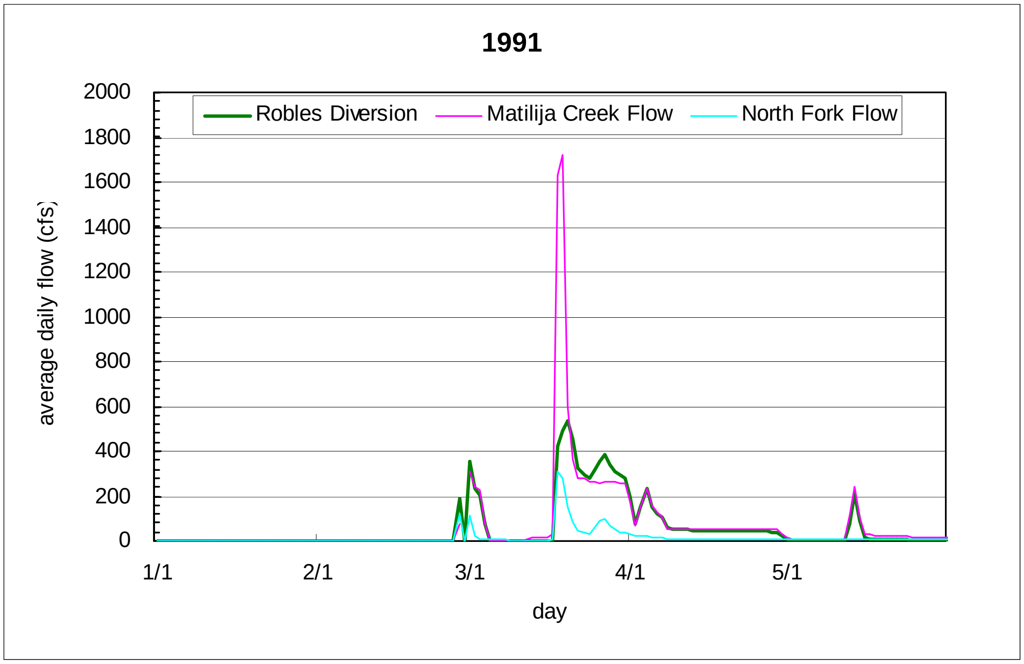
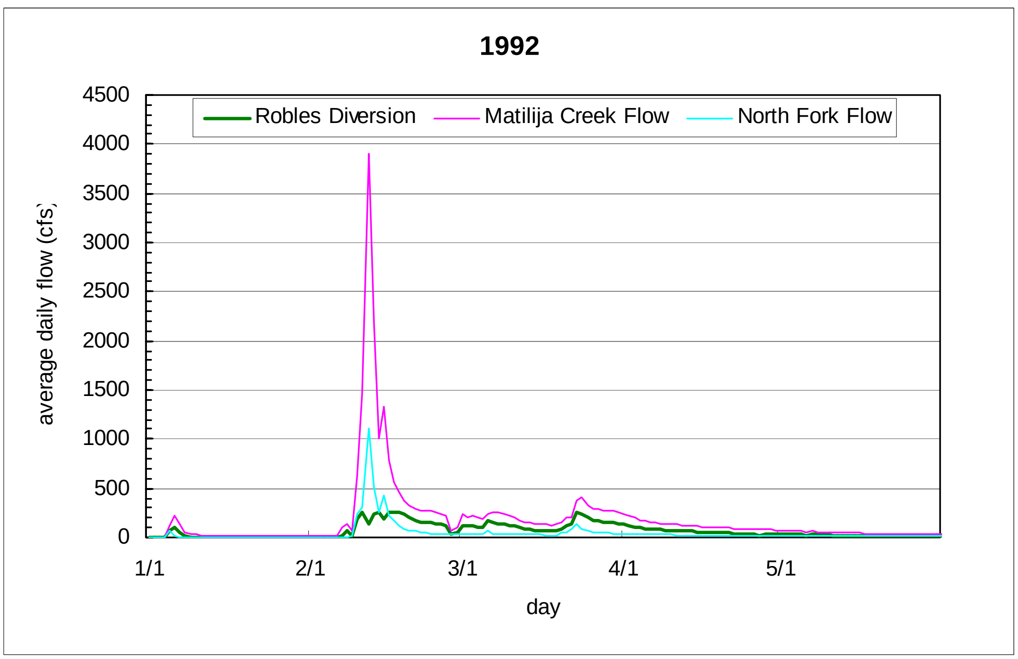
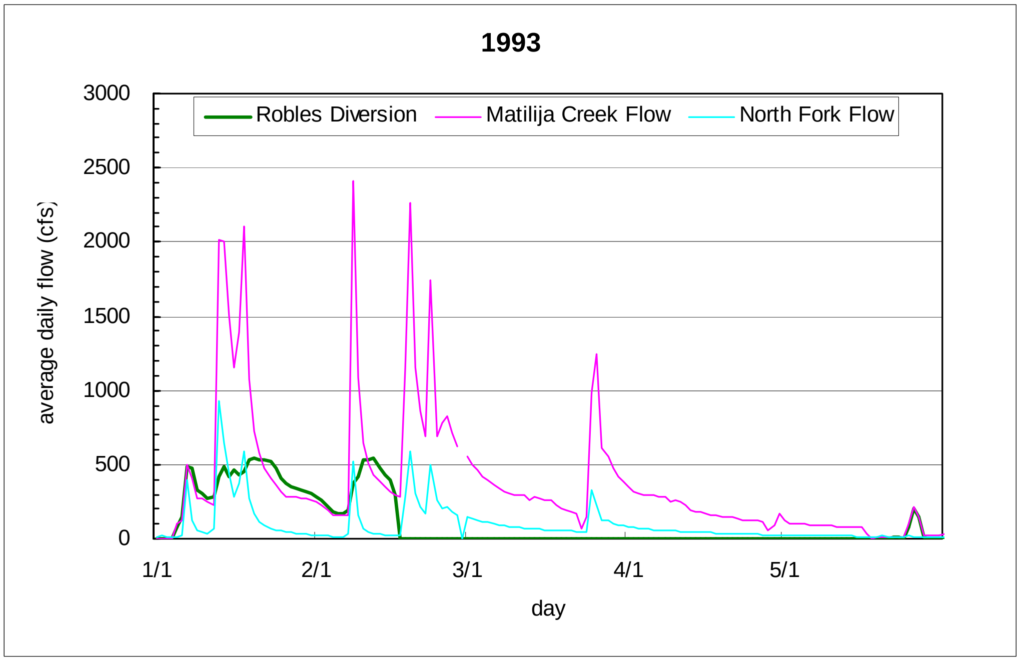


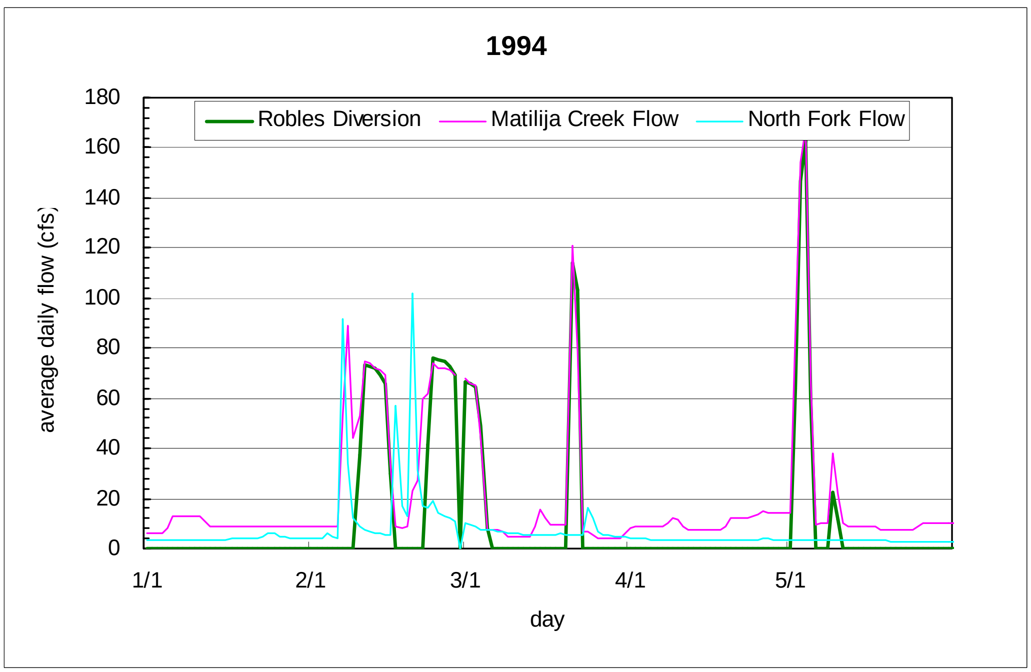
Figure 2.13. Annual Flow and Diversion Volumes for Period 1991 to 2000.

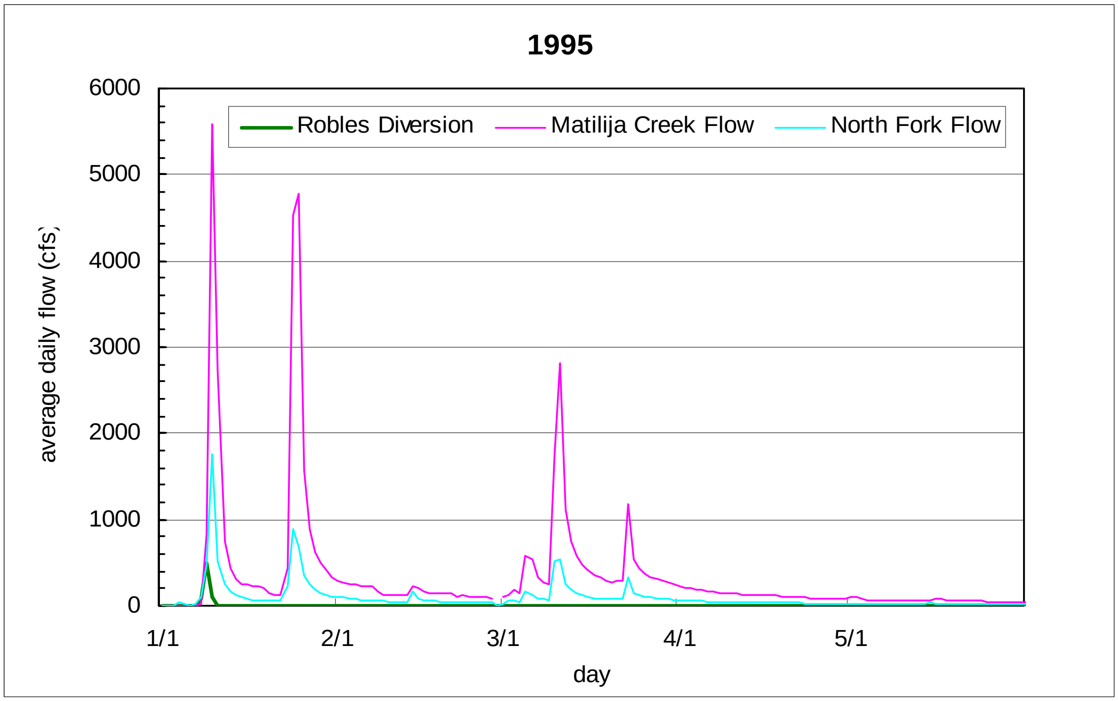
The actual daily average flow diversions for the period of 1991 to 1998 are shown on the following pages (Figure 2.14). It should be noted that there are large variations in the flow for a given year and as a result, there are large variations in the diversion at Robles. For example, in 1997, 47% of the flow of North Fork and Matilija Creek was diverted into Robles Canal.

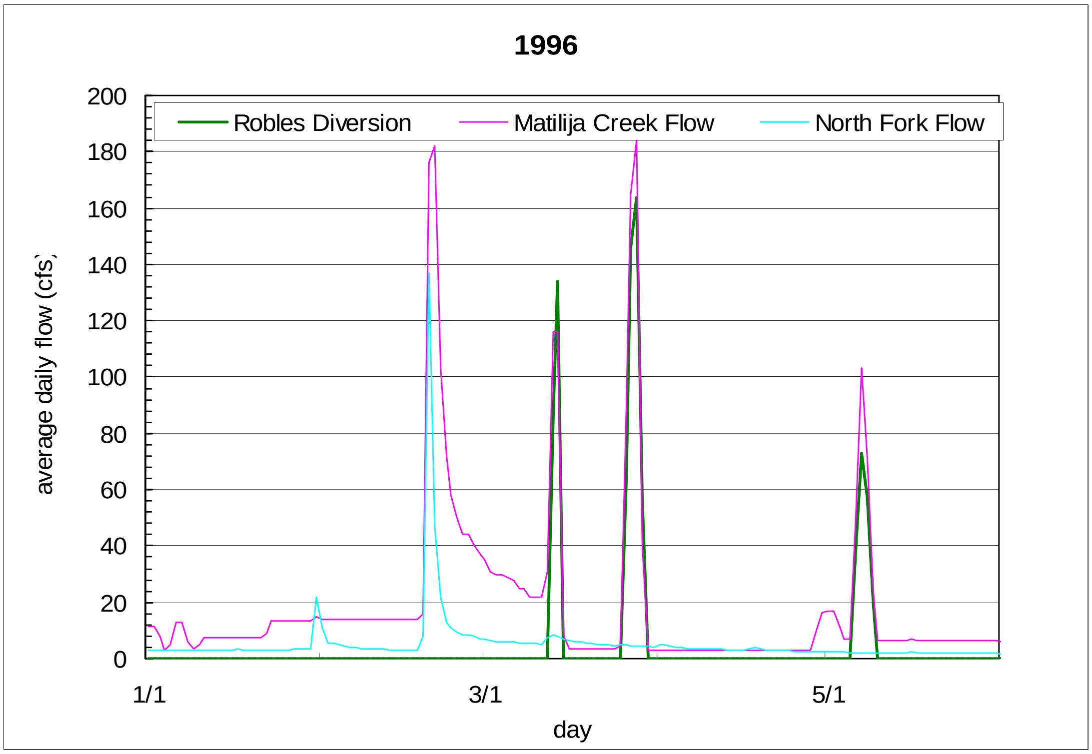

1991
Robles Diversion
Matilija Creek Flow
North Fork Flow
average daily flow (cfs)
2000
1800
1600
1400
1200
1000
800
600
400
200
0
1/1
2/1
3/1
4/1
5/1
day

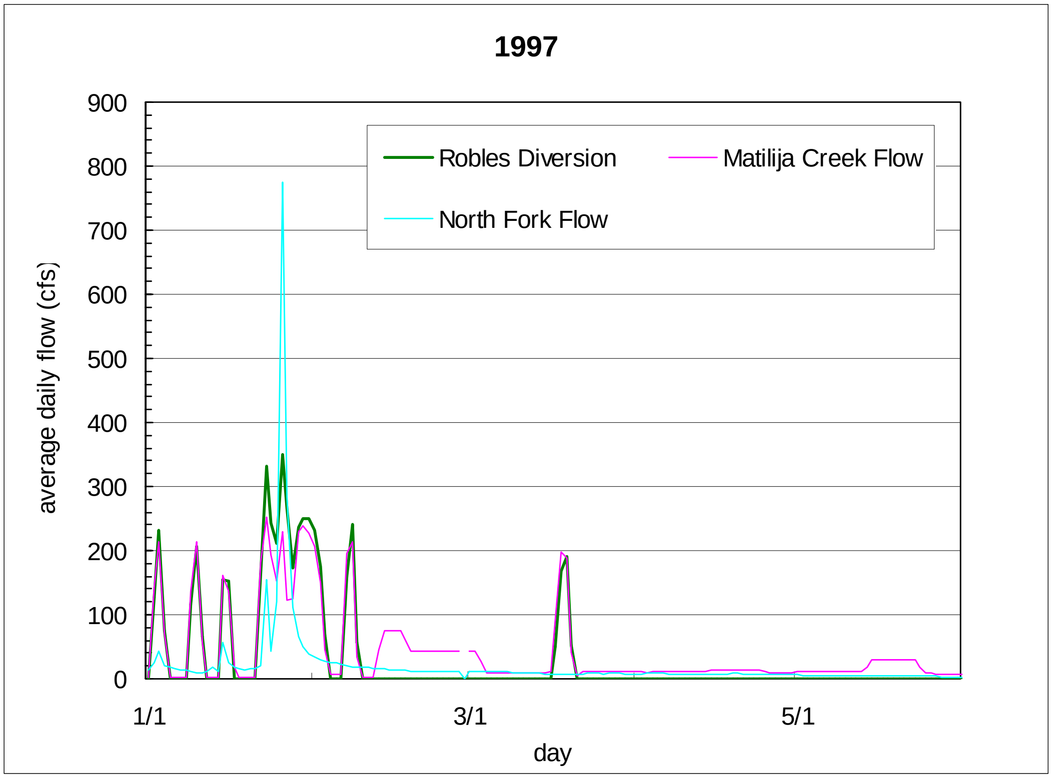

1992
Robles Diversion
Matilija Creek Flow
North Fork Flow
average daily flow (cfs)
4500
4000
3500
3000
2500
2000
1500
1000
500
0
1/1
2/1
3/1
4/1
5/1
day


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


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


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


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



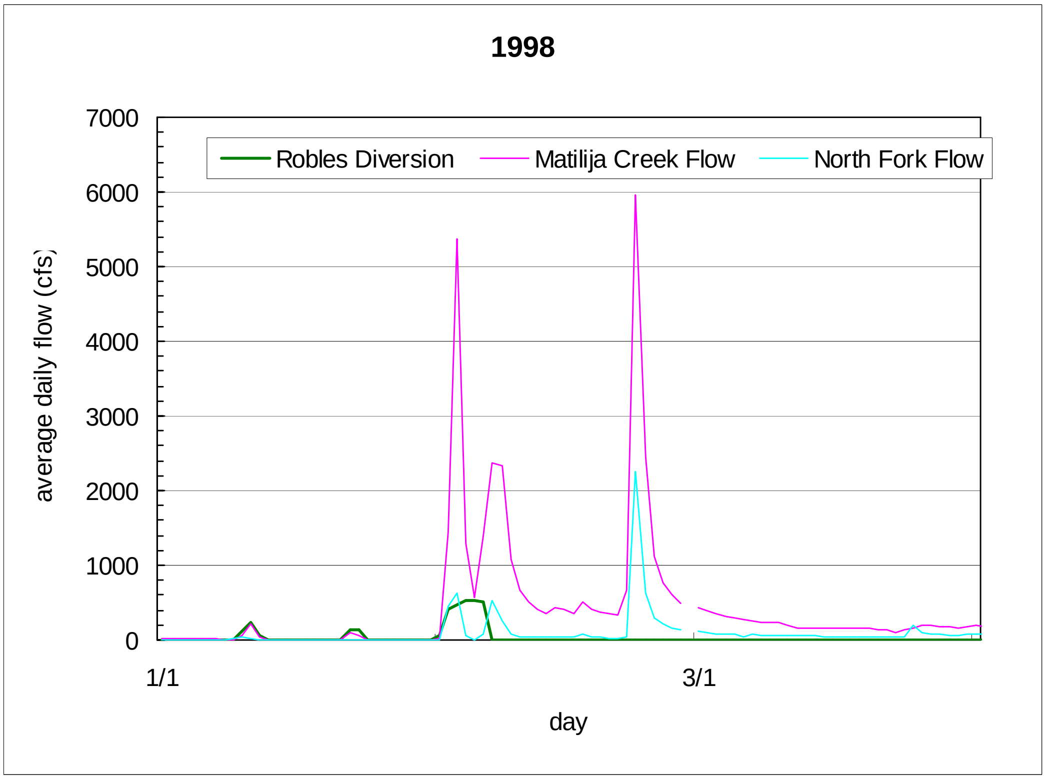


Figure 2.14. Daily average flows at Matilija Creek, North Fork Matilija Creek, and Robles Diversion for the period 1991 to 1998.

2.3.1. BENEFIT OF MATILIJA DAM

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 the contribution of Matilija Dam to the safe annual yield of Lake Casitas 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. Because the capacity of Matilija reservoir is less than 500 ac-ft, it is expected that Matilija's contribution to the safe yield of Casitas Reservoir is now much less than 420 ac-ft/yr.

The benefit was recalculated using all available hydrologic data not just the hydrology during the dry years used to determine the safe yield. It was assumed the current storage capacity is 470 ac-ft. Therefore, the benefit computed will be higher than the contribution to the safe yield. This was done so that an upper bound of the benefit of Matilija Dam to Robles Diversions could be calculated. In addition, the operations of Matilija Reservoir and Robles Diversions were 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 470 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 470 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 470 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.

The flows records listed in Table 2.6 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 560 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 560 ac-ft/yr represents 4.3 percent of the average annual Robles diversion volume of 13,000 ac-ft.

Table 2.6. 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

3. 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 riverbed. 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)			

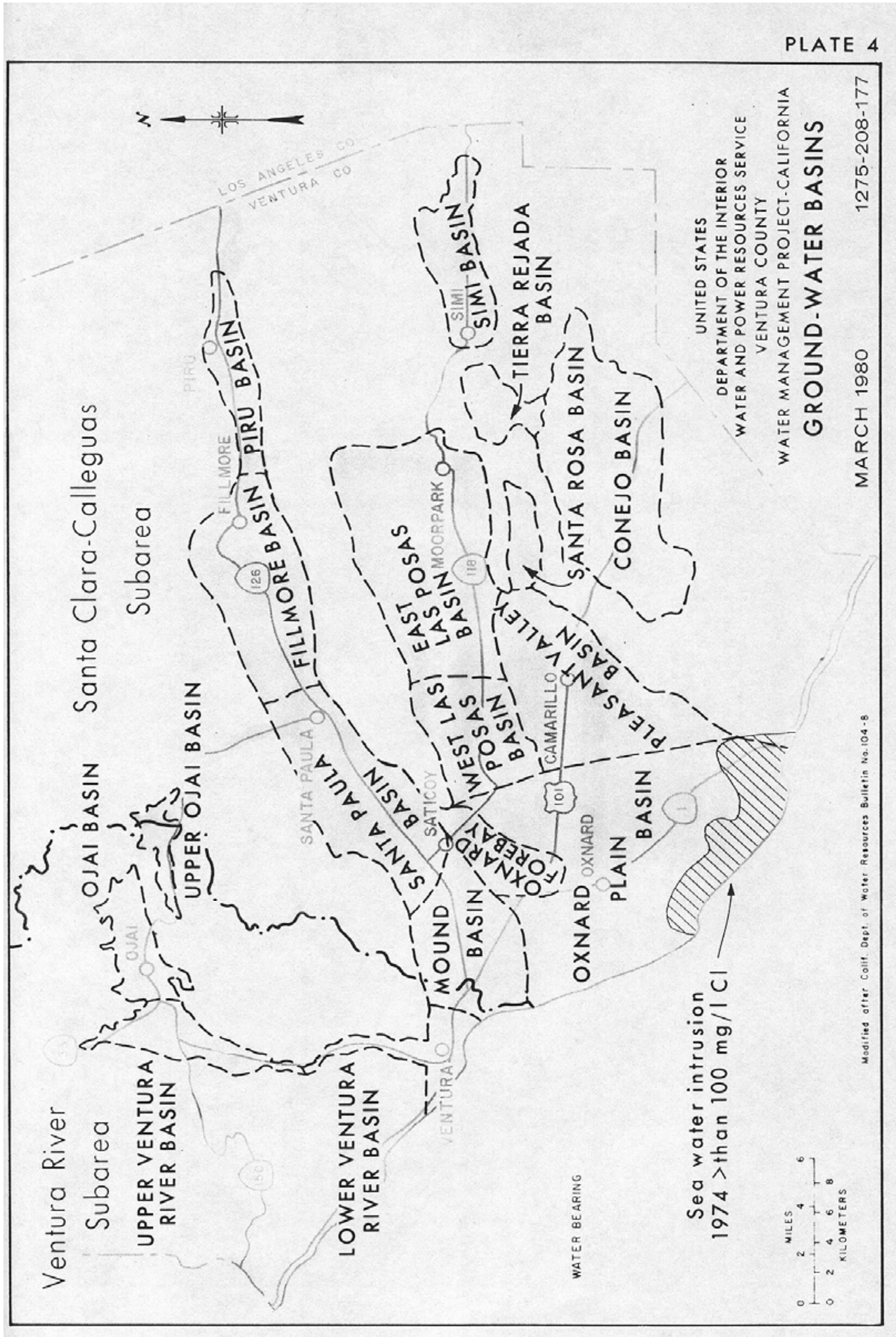


Figure 3.1. Map of groundwater basins in Ventura County. From Reclamation (1981).

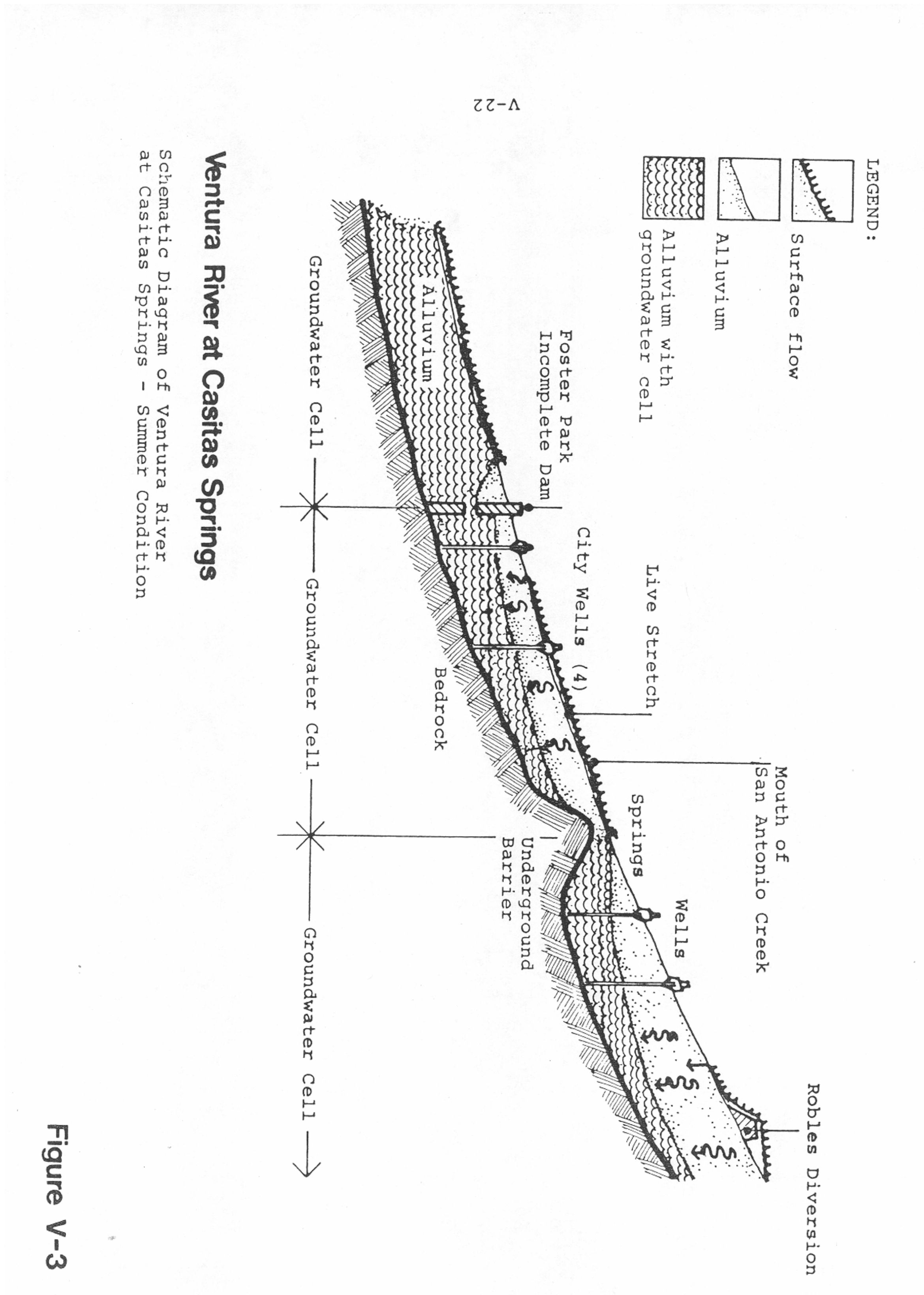


Figure 3.2. Schematic of groundwater basins below Ventura River (Turner, 1971).

4. Hydraulics

This section documents the estimation of water surface elevations and flood boundaries for the 10-year, 20-year, 50-year, 100-year, and 500-year flood peaks (Table 2.2).

4.1. Cross Section and Bridge Geometry

A LiDAR aerial survey performed by Airborne1 in March of 2005 was used to as the base survey. Airborne1 provided raw point data in ASCII format. They provided data with two different levels of processing. One level of processing was termed the “bare earth” data set and other was termed “model key points” data set. The model key points contained approximately 1/3 the number of points as the bare earth data set. The LiDAR data was found to have a discontinuity near the where the Ventura River crosses the Oil Fields at approximately RM 4. It appeared that the points to the south of this point were given in a vertical datum of NGVD29 rather than NAVD88. More specifically, the tiles numbered greater than 1100 were reported in a vertical datum of NGVD29 while the other tiles were reported in NAVD88. The correction in this area between the two was 2.487 feet at RM 4 and 2.438 feet near the ocean. Therefore, an average correction of 2.46 feet was applied to all tiles with a number greater than 1100.

A Triangular Irregular Network (TIN) was generated from both the bare earth and model key point’s data set. The bare earth TIN filled approximately 1.4 GB of hard disk space while the model key point TIN was one-third that size. ARC-GIS running in the Windows XP was would encounter memory limitations when the bare earth TIN was used to generate other data sets, such as cross sections. Therefore, the TIN generated from “model key points” served as the surface from which HEC-GeoRAS Ver 4.1 generated cross sections. The cross sections were generated every 500 feet. Then these cross sections were manipulated so that they did not cross each other, were approximately perpendicular to flow lines, and captured important hydraulic controls, including bridges and structures. The cross section locations are given in “Exhibit C. Cross Sections used in Study”.

The 2005 LiDAR survey was compared against the 2001 Photogrammetry survey. In general, the comparison between the surveys found little inexplicable change. Major differences between the surveys could generally be explained by changes in river plan form, bank erosion or differences in the methods used to eliminate vegetation from the aerial survey. However, one important difference in the two surveys was found in the Hot Springs reach (RM 16). The 2005 survey showed the bed approximately 3 to 4 feet higher than the 2001 survey in this reach. Two example cross sections are shown in Figure 4.1 and Figure 4.2. However, subsequent ground surveys confirmed that the 2001 survey was more accurate in this reach and therefore the 2005 cross sections were corrected within HEC-RAS before the hydraulic computations were performed.

Bridge pier and deck information were taken from the previous survey performed in 2001 as documented in Reclamation (2004). Santa Ana Bridge is at a 30-degree angle to the line perpendicular to the river flow. Therefore, a 30-degree skew angle was used for the bridge deck and the bridge cross sections in HEC-RAS to compute the bridge hydraulics.

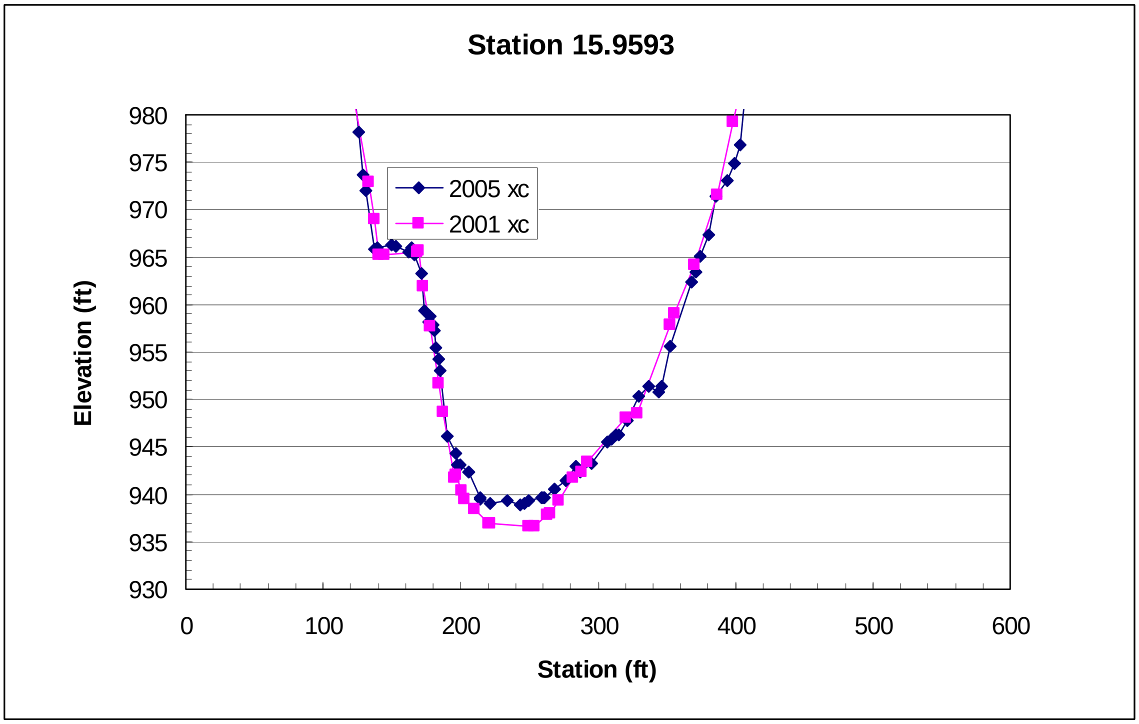


Figure 4.1. Comparison between 2001 and 2005 aerial surveys at Station 15.9593.

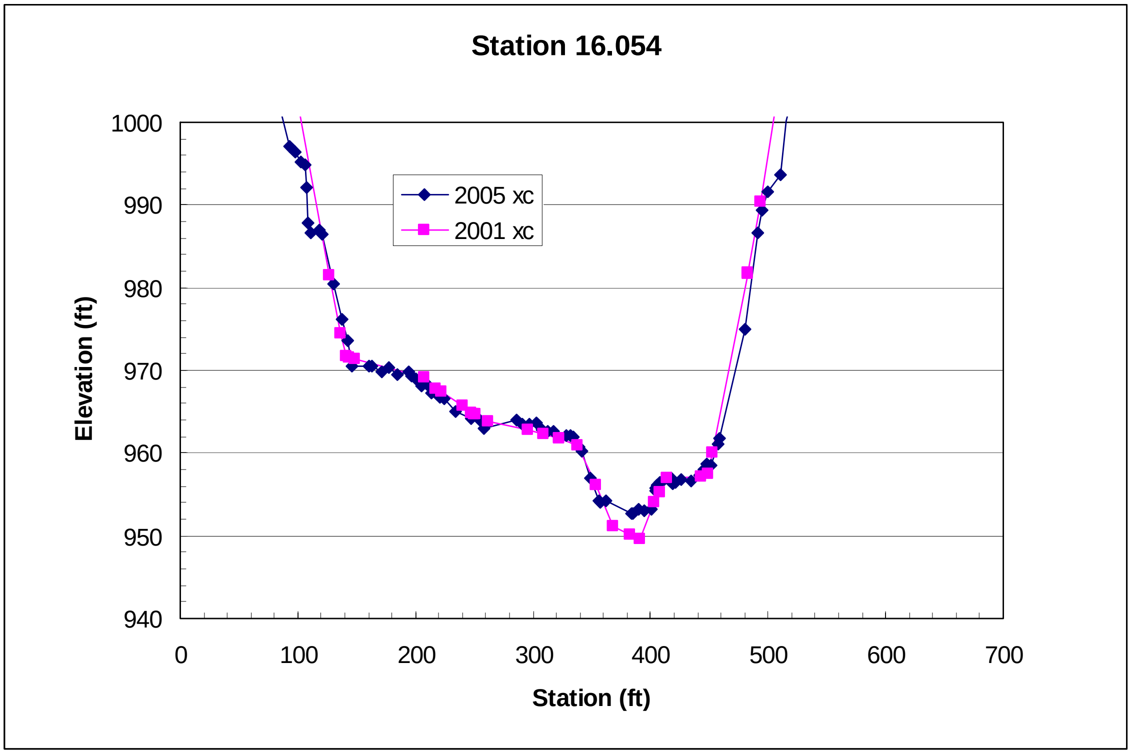


Figure 4.2. Comparison between 2001 and 2005 aerial surveys at Station 16.054.

4.2. Hydraulic Roughness

The hydraulic roughness values (Manning's n) were calibrated using high water marks surveyed following the 2005 WY. High water marks were surveyed at four locations along the Casitas levee, two locations along the Live Oak Levee, and two locations just downstream of Robles Diversion Dam. The peak flows occurred on January 10, 2005. The peak flow at Foster Park was measured by USGS. The peak flow at Matilija Hot Springs was measured by CMWD as well as at dam crest. The flow used in this analysis was computed from an average of the two measurements. The peak flow at Santa Ana Bridge was measured by the Ventura County ALERT system. The peak flows at North Fork and San Antonio Creek were measured by the County at the stream gage locations. The estimated peak flow at these locations is summarized in Table 4.1.

The best-fit Manning's n value was 0.04 for the entire river below Robles Diversion Dam. The comparison between all measured water surface elevations (WSE) and computed WSE is shown in Figure 4.3, Figure 4.4, and Figure 4.5. All computed WSE matched the observed to within 0.5 feet except for the observed water surface near RM 13.59 (Figure 4.5). The measured water surface at RM 13.59 was approximately 1 foot lower than computed. It is uncertain why there is this discrepancy and future field surveys will investigate this site. Excluding the RM 13.59 point, the average difference showed computed WSE 0.1 feet lower than the measured WSE. The channel roughness upstream of Robles, in Matilija

Canyon was increased to 0.06 because of the large boulders in this area. The roughness of the section between Robles and Matilija Canyon was set to 0.05 so that the roughness change is more gradual.

The computed WSE using GSTAR-1D is also shown on Figure 4.3, Figure 4.4, and Figure 4.5. There is little difference between the water surface elevations computed by GSTAR-1D and HEC-RAS. If all cross sections not affected by bridges are eliminated, the average difference between the HEC-RAS computed WSE and the GSTAR-1D WSE for this calibration flow is less than 0.1 feet.

Table 4.1. Peak Flows Measured on January 10, 2005.

Stream Gage	**River**	**Drainage Area (mi^2)**	**Flow (cfs)**	**Time**
Foster Park	Ventura River	188	41000	0800
Santa Ana Alert Gage	Ventura River	89.8	20000	unknown
Matilija Hot Springs	Matilija Creek	54.7	7400	unknown
San Antonio	San Antonio Creek	51.2	25000	0800
North Fork	North Fork Matilija Creek	15.6	5000	0810

Table 4.2. Hydraulic Roughness used in HEC-RAS model.

RM	**Floodplain Roughness**	**Channel Roughness**
16.3 to 14.9	0.08	0.06
14.9 to 14.25	0.08	0.05
14.25 to 0	0.08	0.04

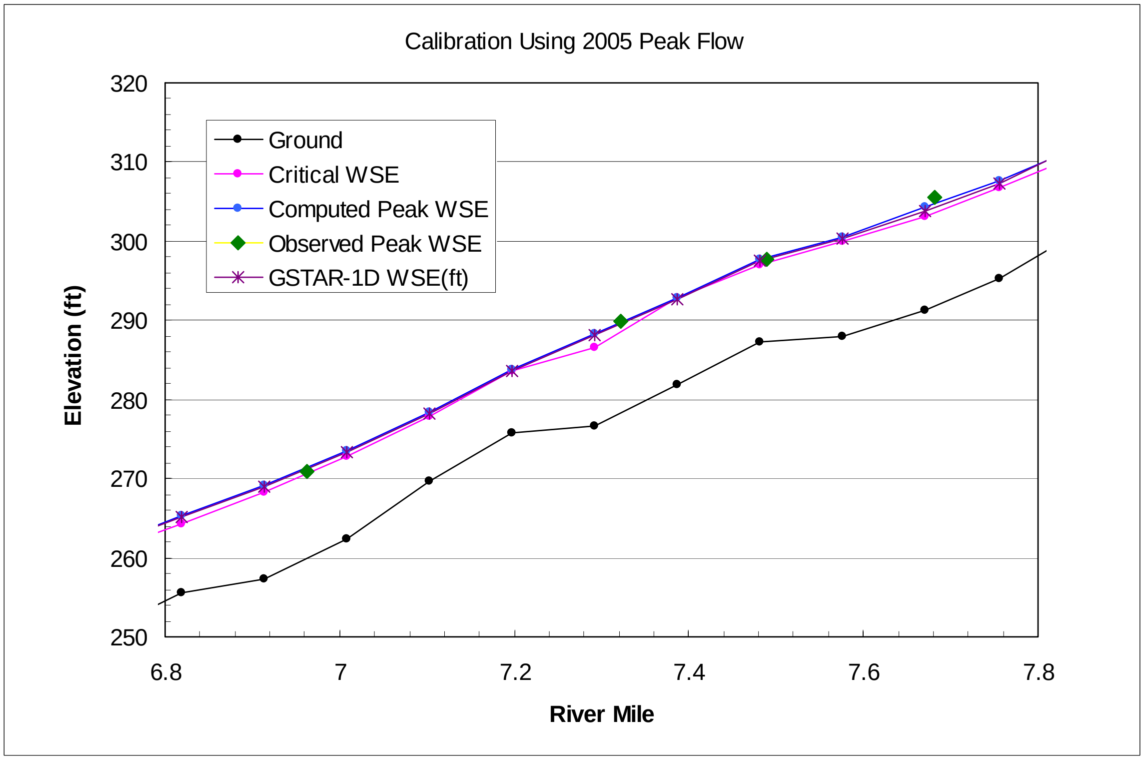


Figure 4.3. Plot of observed and computed peak water surface elevations at the Levee near Casitas Springs.

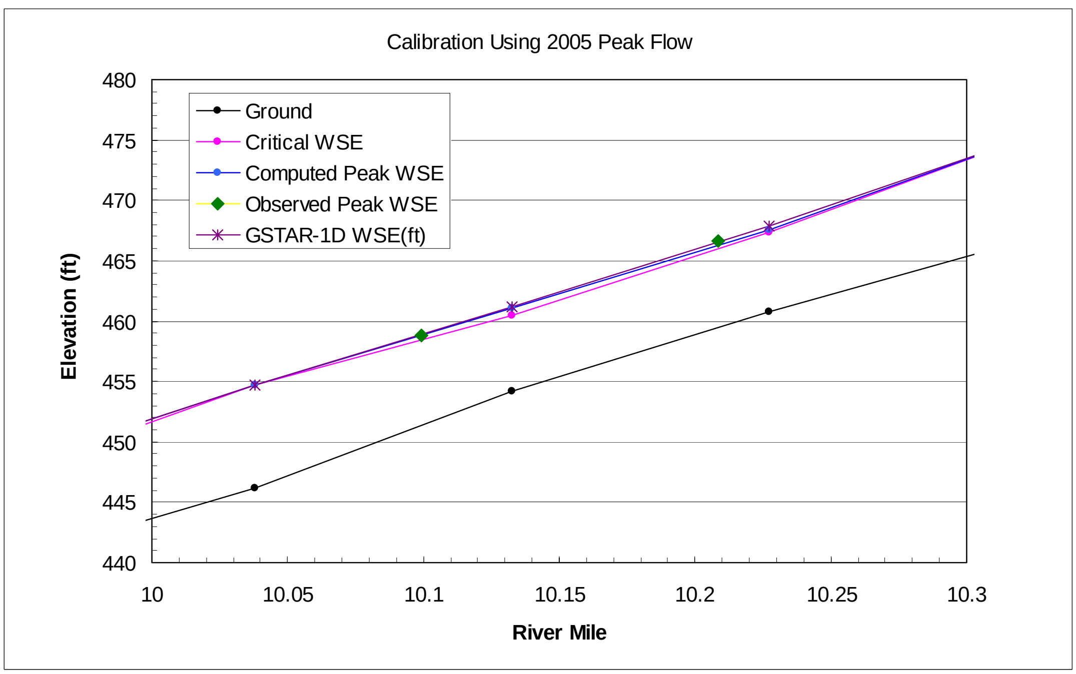


Figure 4.4. Plot of observed and computed peak water surface elevations at the Levee near Live Oak.

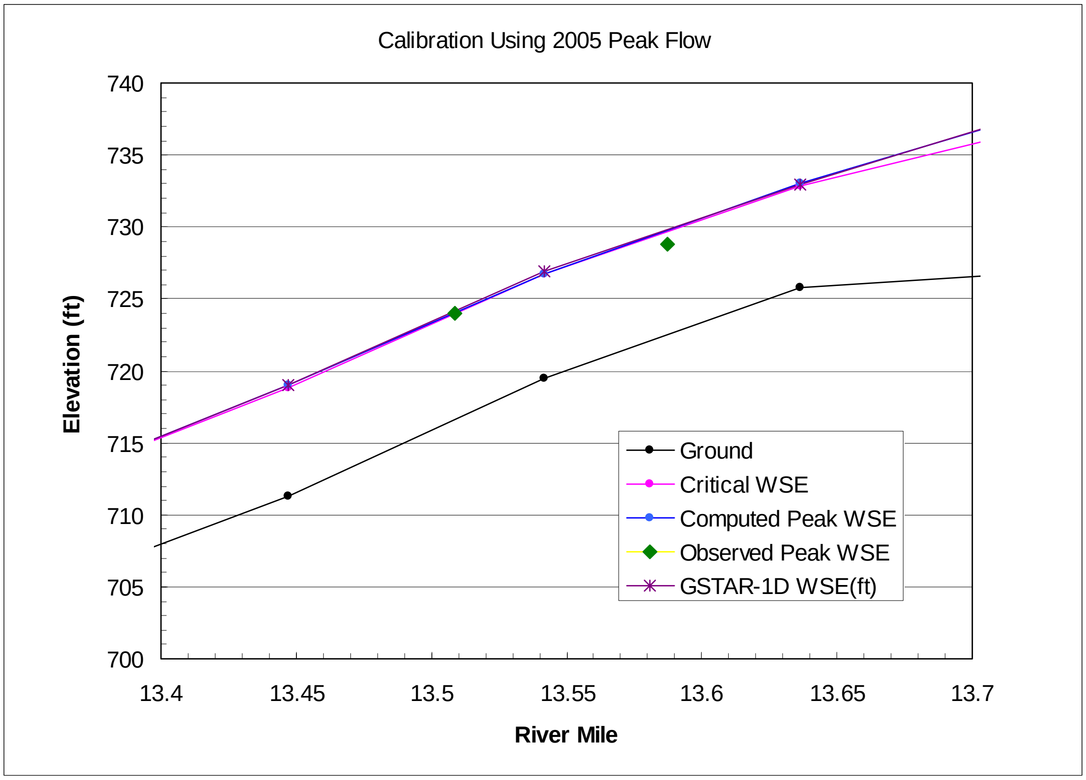


Figure 4.5. Plot of observed and computed peak water surface elevations just downstream of Robles Diversion.

The roughness was increased to 0.06 in Matilija Canyon to reflect the large boulders present and vegetated banks. The roughness was based on Manning's n-values reported by the 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 0.03 to a high estimate of 0.05. Table 4.3 shows the results. All simulations used a floodplain roughness coefficient of 0.08.

Table 4.3. Results of Manning's n Sensitivity Analysis.

Reach	**Δ WSEL n = 0.03**	**Δ WSEL n = 0.05**
Santa Ana Bridge to Robles	-0.17	0.54
Casitas Vista Bridge to Santa Ana Bridge	-0.54	0.78
Ocean to Casitas Vista Bridge	-0.59	0.92

The analysis indicated that the Manning's coefficient has only a small effect on water surface elevations. Much of the river flowed near a Froude number of 1. Critical depth controls the water surface more than the roughness coefficients. Interpolating additional cross sections may improve the accuracy of the flow modeling. In some cases, interpolating additional sections may decrease the

Froude number and shift more water surface control to the roughness coefficient. Additional cross sections may increase the water surface elevation.

4.3. 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 F. Flood Mapping" and show the inundated areas along the Ventura River for the study reach. The inundation mapping assumed engineered 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 for flows equal to or greater than the 50-yr flow discharge. The floodplain was expanded 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.

Overflow mapping does not include the flooding induced by rivers other than Matilija Creek and the Ventura River. For example, the flooding caused by Cozy Dell Canyon, San Antonio Creek and Fresno Canyon is not considered as part of this study.

4.4. Flood Risk Assessment

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

Reach 6b – RM 16.4-15.0

Reach 6b begins immediately downstream of Matilija Dam and extends downstream to the canyon mouth. There is development at the "Matilija Hot Springs" facility and around the Camino Cielo Bridge.

<u>Matilija Hot Springs</u>: This reach contains little development except the "Matilija Hot Springs" facility. The 50-year event and above inundate the lower grounds and the 500-yr flood will inundate the Hot Spring's pool and surrounding structures.

<u>Camino Cielo</u>: Camino Cielo Bridge has three openings each approximately 11 foot wide and 5 ½ feet high. The original design estimated the capacity of these openings to be approximately 1000 cfs. While the original design estimated the streambed to be at an elevation of 867 feet, the LiDAR survey estimated the bed upstream of the bridge to be approximately 870 feet. It is likely that there was deposition following the January 2005 flood on the upstream side of the bridge.

This deposition decreased the opening height to less than 3 feet and the Camino Cielo Bridge is overtopped at approximately 500 cfs (Figure 4.6).

There are several structures on the south side of the river and upstream of the Camino Cielo Bridge from RM 15.62 to 15.45 that are located near but slightly above the 100-yr and 500-yr floodplains. On the south side of the river downstream of the bridge, there is in one residence at cross section (XC) 15.1515 that is below the 10-yr flood elevation. The next property downstream is between XC 15.1515 and 15.0568 and is surrounded by the 10-yr floodplain, and the structure is only 1 to 2 feet above the 100-yr floodplain.

North of the river and upstream of the Camino Cielo Bridge between XC 15.3675 and 15.3873, one structure is located in the 10-yr floodplain. The base of this structure will be below 3 feet of water at the 100-yr flood. Downstream of the Camino Cielo Bridge, between XC 15.3409 and 15.2917 the property is encircled by the 100-yr flood. However, the structures on this property are 5 to 6 feet above the 100-yr flood elevations. There is an orchard on the Northeast side of the river from XC 15.2462 to 14.9621. Some parts of the upstream section of this orchard are inundated at the 10-yr flood. The 100-yr flood inundates 1/4 to 1/3 of the orchard.

Figure 4.6. Downstream side of Camino Cielo.

Reach 6a – RM 15-14.0

Reach 6a begins at the canyon mouth and extends downstream to Robles Diversion Dam.

Meiners Oaks Area: There are approximately 20 structures located along Oso Road and North Rice Road between RM 14.4 and 14.1 within Reach 6a. (There are additional structures within this community downstream of 14.1, but located in Reach 5.) All of these structures are constructed at grade. There is no functional levee, but most all of these structures are around 40 feet above the 100-year floodplain. There may be some flood risk caused by the flows originating from Cozy Dell Canyon, but these are not considered as part of this study.

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.

Floods can cause a variety of problems at Robles Diversion. One such problem is associated with the fish screens that were placed in the canal as part of the fish passage facility constructed in 2004. These fish screens can become plugged with sediment and debris and prevent flows from passing down the canal. The screen cleaning equipment in current use is not adequate to keep the screens clean during floods (see Figure 4.7).

Figure 4.7. Picture Taken During January 2005 Flood Showing Fish Screens Removed in Robles Canal.

Reach 5 – RM 14.0 – 11.10

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

Continuation of Meiners Oaks Area: There are several residences located to the east of the river between RM 14 and 13.4. All of these structures are constructed at grade, with no significant first floor elevation above the floodplain and there is no engineered levee. If the 100-yr does not cause any lateral migration to the east, these residences would be protected by a berm made of river deposits that extends from Robles Dam to approximately RM 13.2. Downstream of RM 13.8, however, the channel has shown evidence of large migration rates and this natural berm could be eroded by large flows. Therefore, below RM 13.8, the berm was not considered to function as a levee for the floods equal to or greater than the 50-yr flood. To estimate the amount of bank erosion during large floods the channel migration zone was digitized from the 1970 and 1978 aerial ortho-rectified photos. If the current 50-yr floodplain did not cover the maximum extents of the channel migration zone, the floodplain was extended to the extents of the zone.

The berm was assumed to act as a functional levee for the 10-yr and 20-yr floods. It should also be noted that the Cozy Dell drainage passes through this community and this drainage can cause substantial flooding. The flows from Cozy Dell were not considered in this study but the project will have not effect on this flood impacts associated with the flows from Cozy Dell. However, a complete flood risk analysis should consider the flows from Cozy Dell.

Burn Dump: The area called the “Burn Dump” is located on the East side of the river between RM 11.3 and Baldwin Road. There is grouted riprap that provides erosion protection, but it does not provide flood protection because it is not tied into the bridge abutments on the downstream end and the upstream end is not tied into the surrounding topography. On the upstream end of the grouted riprap, a levee was constructed of riverbed material but the river has already eroded through it in several locations. A picture of this levee is shown in Figure 4.9.

There are also OVSD sewer lines that are along the east side of the river from RM 12.12 to Baldwin Road Bridge. Much of the sewer line is within the 100-yr floodplain. Some of the sewer line just upstream of the Burn Dump is also within the historical channel migration zone. One OVSD pipeline is buried beneath the Ventura River upstream of Baldwin Road.

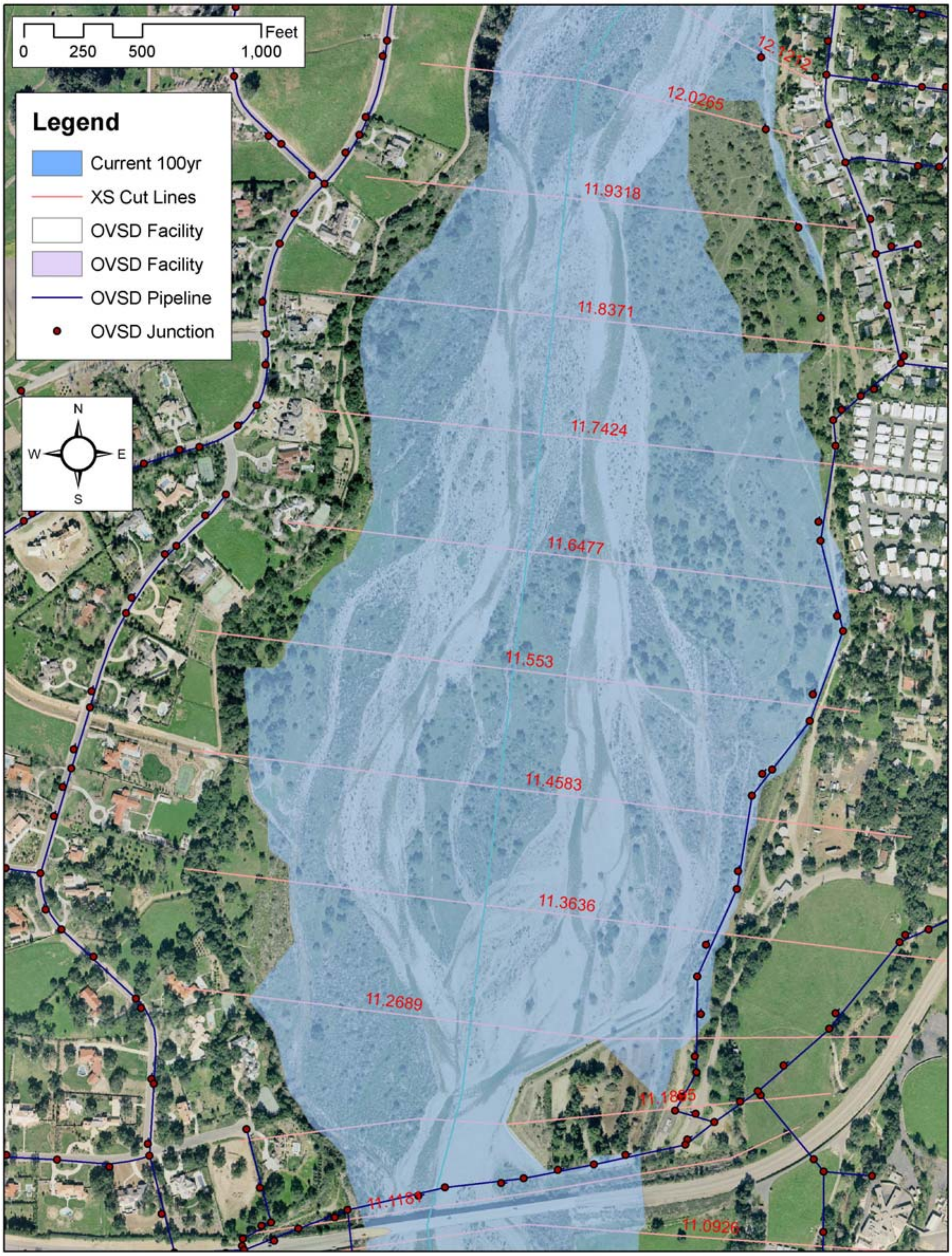


Figure 4.8. 100-yr floodplain from RM 12 to RM 11. Showing location of OVSD pipelines

Figure 4.9. Levee located upstream of "Burn Dump" on east side of Ventura River at approximately RM 11.54.

Reach 4 – RM 11.10 – 7.93

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

Between Baldwin Road and Live Oak Acres: Many residences along the west side of the river are near the 100-yr floodplain. One property at RM 10.7 on the west side of the river is located within the 10-yr floodplain. A levee is high enough to protect this property from the 100-yr flood, but the levee is constructed of unknown material and the river may erode it. The 2005 flood moved the river channel very near this levee.

There is an OVSD sewer line located along the west side of the Ventura River downstream of Baldwin Road within the 100-yr floodplain (Figure 4.10).

Live Oak Drain enters the Ventura River from the west side just upstream of Live Oak Acres at approximately RM 10.15 (Figure 4.11). Live Oak Drain has a bottom elevation of approximately 457.5 feet where it crosses under Burnham Road. It was designed to carry the 100-yr flood of approximately 890 cfs at a flow depth of approximately 5 feet at a slope of 0.0009. However, the designed assumed a elevation of 456.5 feet at the drain exit. Since that time, the drain exit has aggraded to 458 feet. Therefore, there is now essentially no slope to the drain from Burnham Road to the Ventura River, a distance of approixmatley 860 feet. The 100-yr flood elevation of the Ventura River at this location is approximately 462 feet. The levee elevation along the drain is approximately 469.5 feet. Because there is no slope to the drain, it is likely that it will continue to experience aggradation.

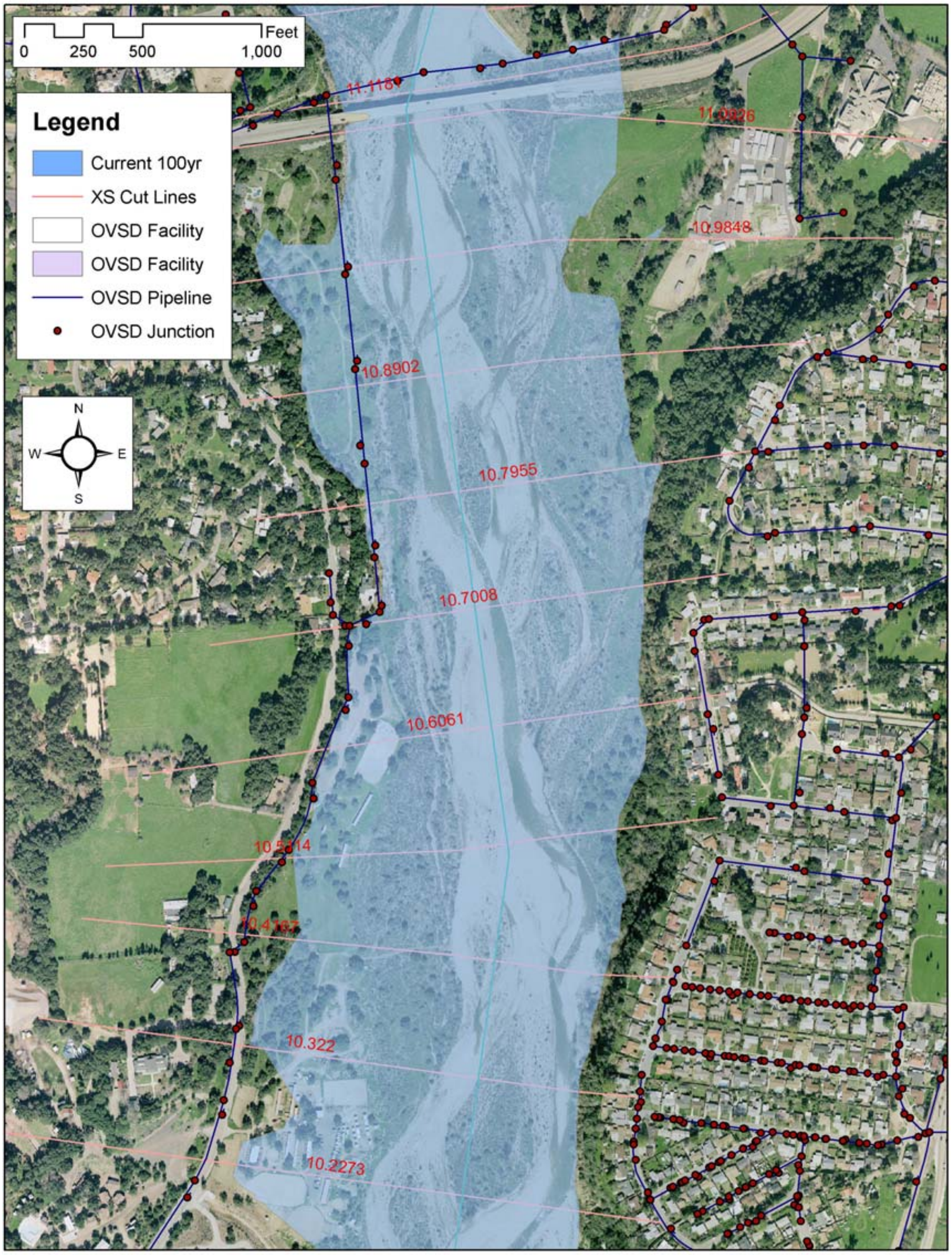


Figure 4.10. 100-yr floodplain from RM 11 to RM 10 Showing location of OVSD pipelines.

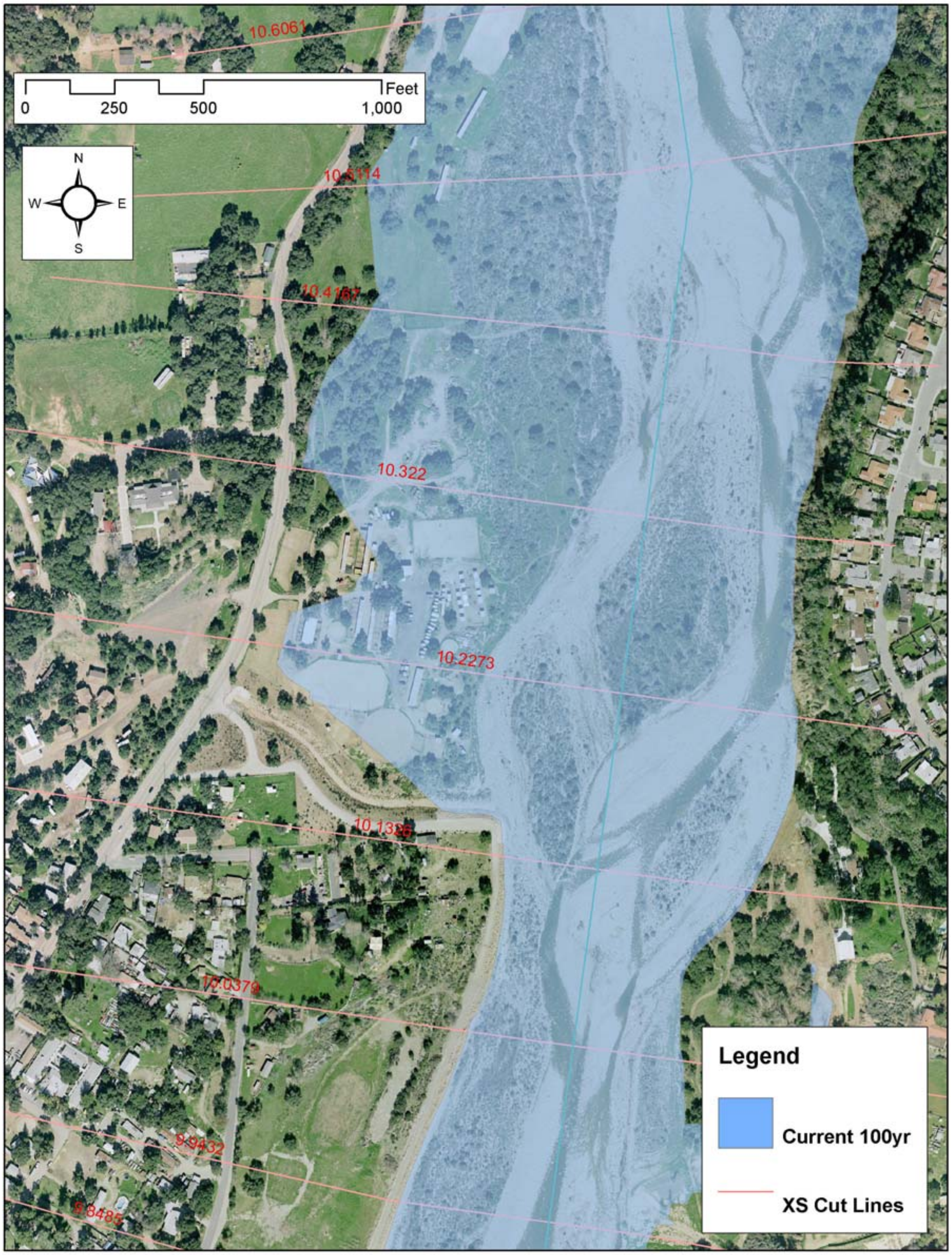


Figure 4.11. 100-yr Floodplain in Vicinity of Live Oak Drain.

Live Oak Acres: The Live Oak Levee is on the west bank of the Ventura River and extends from RM 9.25 to RM 10.15. It protects the populated area of Live Oak. 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, but the downstream portion is exceeded by the 500-yr flood because of the backwater caused by Santa Ana Bridge.

The Live Oak levee may be subject to erosion as evidence by the damage caused by the Jan 2005 flood shown in Figure 4.12 at approximately RM 9.4. There is backhoe shown in the photo repairing the damage caused by the Ventura River eroding the riprap protection along the West bank. The riprap placed along the Live Oak Levee is approximately ½ ton based upon the County records. From the Santa Ana Bridge to RM 9.5, it is estimated that rock weighing at least 0.8 ton would be required to prevent erosion up to the 100-yr flood. The levee will continue to erode at flood flows with a return period of 20-yr and greater. Smaller flows may also erode the levee if the river is directed at the levee as occurred in 2005.

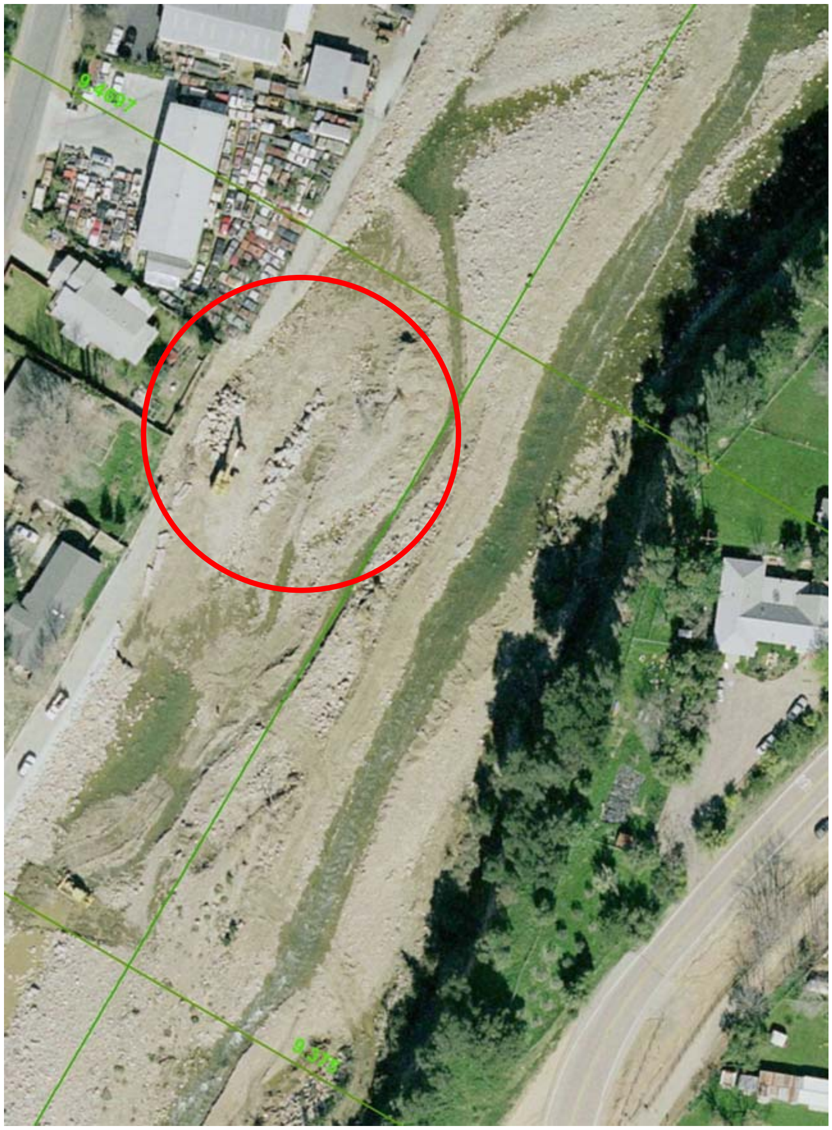


Figure 4.12. Aerial photo taken in February 2005 showing erosion of Live Oak Levee erosion between XC 9.375 and 9.4697.

Along the East Bank of the Ventura River opposite the Live Oak Levee, there are properties that are located at the top of a high terrace (Figure 4.13). This terrace is steep and appears to be primarily composed of old alluvial deposits (Figure 4.14). Therefore, the base of this terrace may be subject to erosion during high flows and the top of the terrace may erode from surface runoff. Most residences appear to be built 25 feet or more away from the edge of the terrace, but fences, utility poles, gazeboes, etc… are within a few feet of the edge. There was evidence of recent bank failure at RM 9.6 along this terrace. These locations are shown in Figure 4.15 .

The County installed protective groins along this bank in the summer of 2005 to prevent any further erosion of the base of the terrace. There are five groins beginning approximately 1200 feet upstream of Santa Ana Bridge and extending approximately 1300 feet further upstream. The protective groins are sufficient to prevent erosion at this location in the immediate future.

Figure 4.13. Picture of Looking at East Bank of Ventura River at Taken at Approximately RM 9.8. Notice houses, utility lines, and Properties Located near Terrace.

Figure 4.14. East Bank of Ventura River Located at RM 9.6 Showing Alluvial Material of Terrace and Some Recent Bank Failure.

Figure 4.15. East Bank of Ventura River Located at RM 9.6 Showing Some Recent Bank Failure,

The Santa Ana Bridge is on the downstream end of the levee and this bridge passes the 100-yr flood, but the flood elevation is only about 1 foot below the bridge soffit. There is historically deposition on the upstream side of this bridge and the County has a program to excavate the riverbed at the Santa Ana Bridge to maintain flow capacity. The bridge is a constriction on the river, and one repercussion of the bridge constriction is that the scour around the bridge abutments is increased, as evidenced in the photo taken after the 1998 flood (see Figure 4.16).

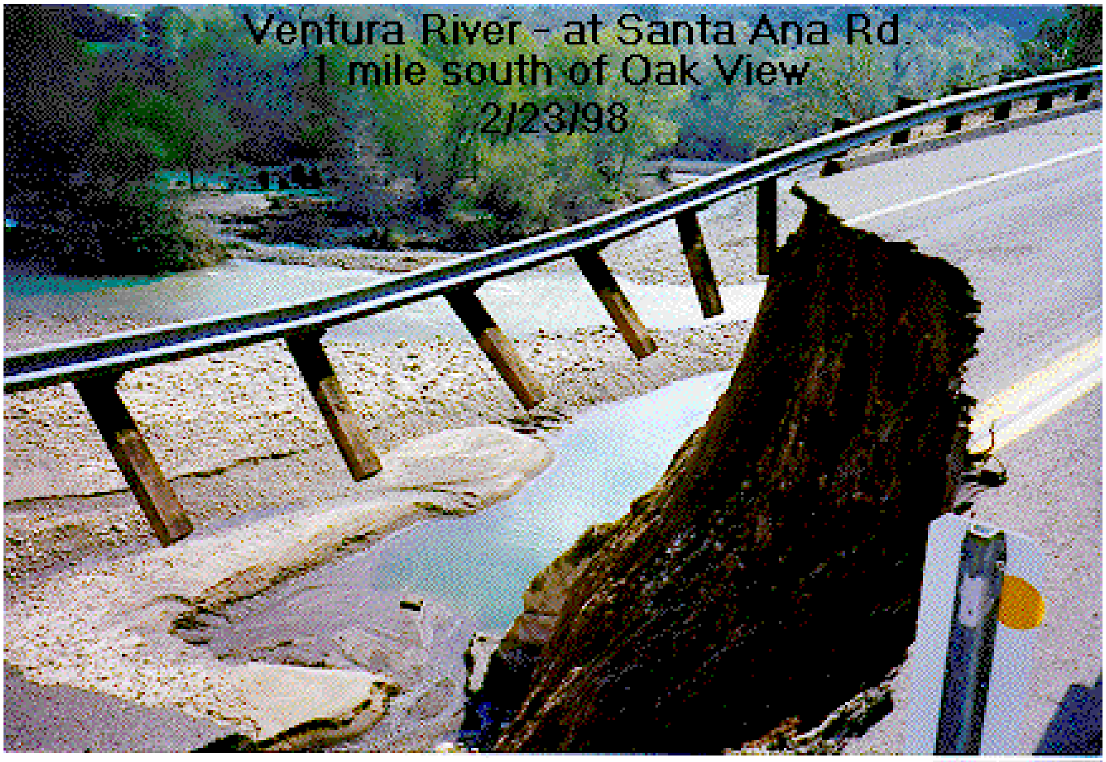


Figure 4.16. Picture of the looking downstream at the Ventura River from the East Abutment of Santa Ana Bridge. Picture was taken after the 1998 flood on 2-23-1998.

Downstream of Santa Ana Bridge: There are at least three residences located between RM 9.2 and 8.9 on the west side of the river. A levee is high enough to protect these residences from the 100-yr flood, but it is constructed of riverbed material that may easily erode at higher flows. The levee is not assumed to protect the residences downstream of the Santa Ana Bridge.

Reach 3 – RM 7.93-5.95

Casitas Springs: The Casitas Springs Levee runs from RM 6.50 to 7.67 along the east side of the river and protects the town of Casitas Springs. The levee elevations were increased in 2005 so that it now contains the 500-yr flood. Previously, upstream sections of this levee were overtopped by the 50-yr flood. Figure 4.17 shows the potential flood risk at Casitas Levee before the levee improvement. The photo was taken near peak flood stage during the 1998 flood event estimate to have a return period less than 20 years and the water surface is very near the top of the levee.

An additional flood risk is caused by the Fresno Drain that passes through Casitas Springs and through the Casitas Levee. The drain is open and near the elevation of the riverbed rendering the Casitas Levee ineffective below RM 6.8. The 10-yr flood can inundate residences on the east side of the river below RM 6.8. One resident located along the Fresno Drain estimated depths of 2 to 3 feet high on her home during the 2005 flood, which was between a 10-yr and 20-yr flood. Another structured along the Fresno Drain has been damaged beyond repair and is shown in Figure 4.18.

There are several OVSD pipelines in the 100-yr floodplain in this reach (Figure 4.19).

Figure 4.17. 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 4.18. Structure just North of Fresno Drain Damaged Beyond Repair by Repeated Flooding of Fresno Drain.

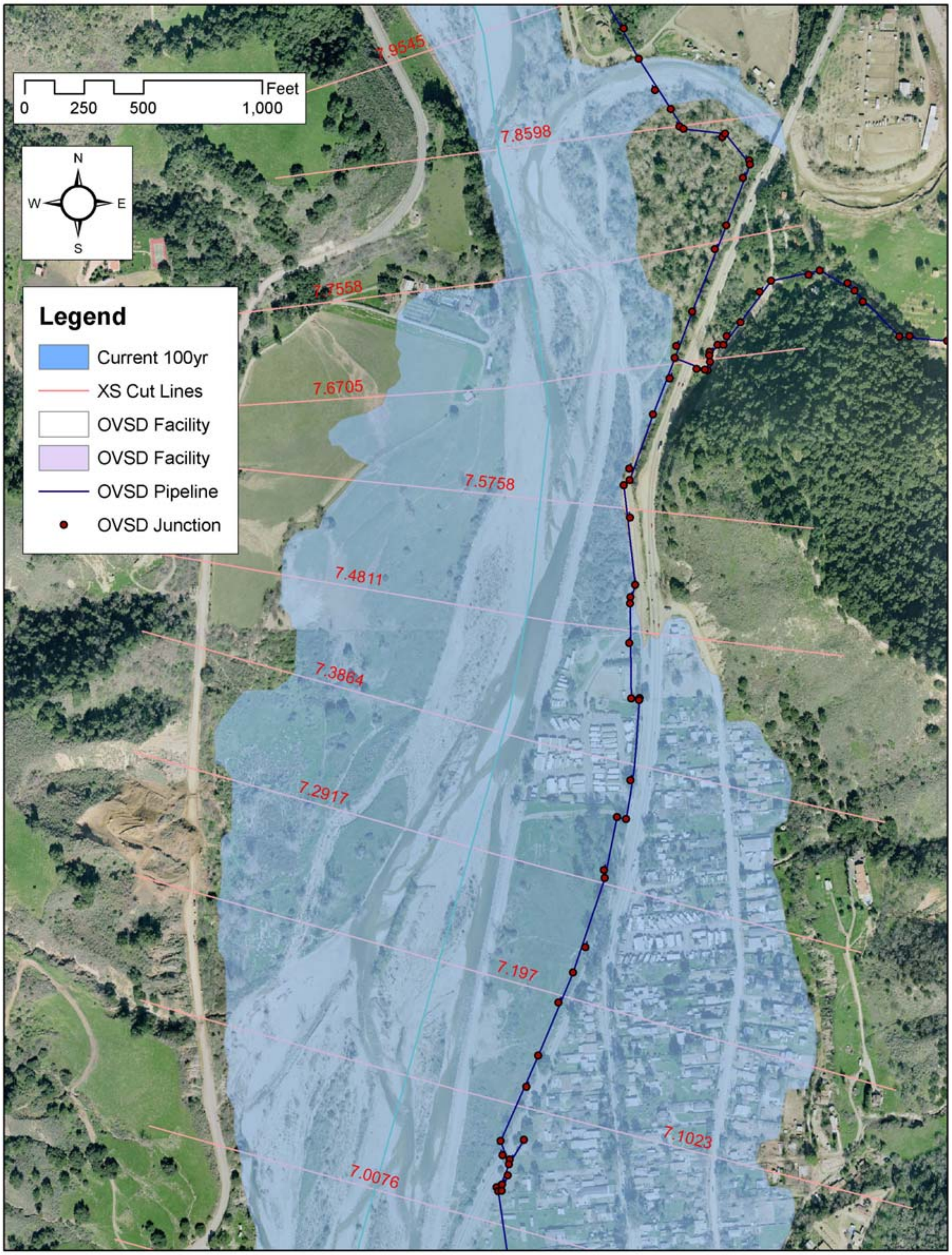


Figure 4.19. 100-yr floodplain from RM 8 to RM 7. Locations of OVSD Pipelines are shown.

<u>Foster Park Area</u>: There is a community upstream of Casitas Vista Bridge located on the west side of the river opposite Foster Park from RM 6.4 to 6.3. A levee surrounds this community and is constructed of riverbed material. It is highly vegetated and it will likely protect the community from the 10-yr flood. There was no evidence of flooding from the 2005 flood. However, the river is beginning to erode the top end of this levee and it is likely that flows larger than the 2005 flood would erode more of this levee and potentially flood the entire community. The flood mapping shows this community flooded for the 50-yr flood and larger.

Foster Park is located on the east side of the river upstream of Casitas Vista Bridge and is within the 100-year floodplain. In the summer of 2005, the County installed groins to protect the park from bank erosion (Figure 4.20).

Figure 4.20. Groins installed after 2005 flood along east side of river at Foster Park.

Reach 2 – RM 5.95-0.6

There are at least three residences located in the 50-yr floodplain on the East bank of the river just downstream of Casitas Vista Bridge (~ RM 5.8).

There is an OVSD pipeline located along the margins of the 100-yr flood plain from RM 6.3 to 5.5.

The OVSD waste treatment facility is located between RM 5.10 and RM 4.88. The pools where sludge is temporarily stored are inundated by the 500-yr flood and the 100-yr flood elevation is within 1/2 foot of inundating the upstream ponds. The buildings where the waste treatment processing occurs are also inundated by the 500-yr flood and the 100-yr flood is within 2 feet of inundating these buildings.

The Brooks Institute is located on the east side of the river between RM 4.64 and 4.55. It is above the 100-yr flood elevation but inundated by the 500-yr flood.

An oil refinery is located from RM 4.3 to 4.0 and is inundated by the 500-yr flood. An assumed operational levee protects it from the 100-yr flood. However, no field inspection of this levee was performed.

The Ventura Levee extends along the east side of the river 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 flood are confined by the Ventura Levee.

There are several agricultural fields located on the west side of the river from RM 1.0 to the Main Street Bridge. Most of these are potentially inundated by the 10-yr flood and all are inundated by the 100-yr flood.

A recreational vehicle park is located on the west side of the river between Main Street Bridge and the Highway 101 Bridge. It is inundated by the 10-yr flood and it showed signs of being inundated in the ortho-photos taken after the January 2005 flood.

5. 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, 1987a, 1987b; 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 that 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 have occurred 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 (1978) only studied watersheds smaller than 10 mi^2 and therefore their conclusions may not necessarily scale up to the larger watersheds.

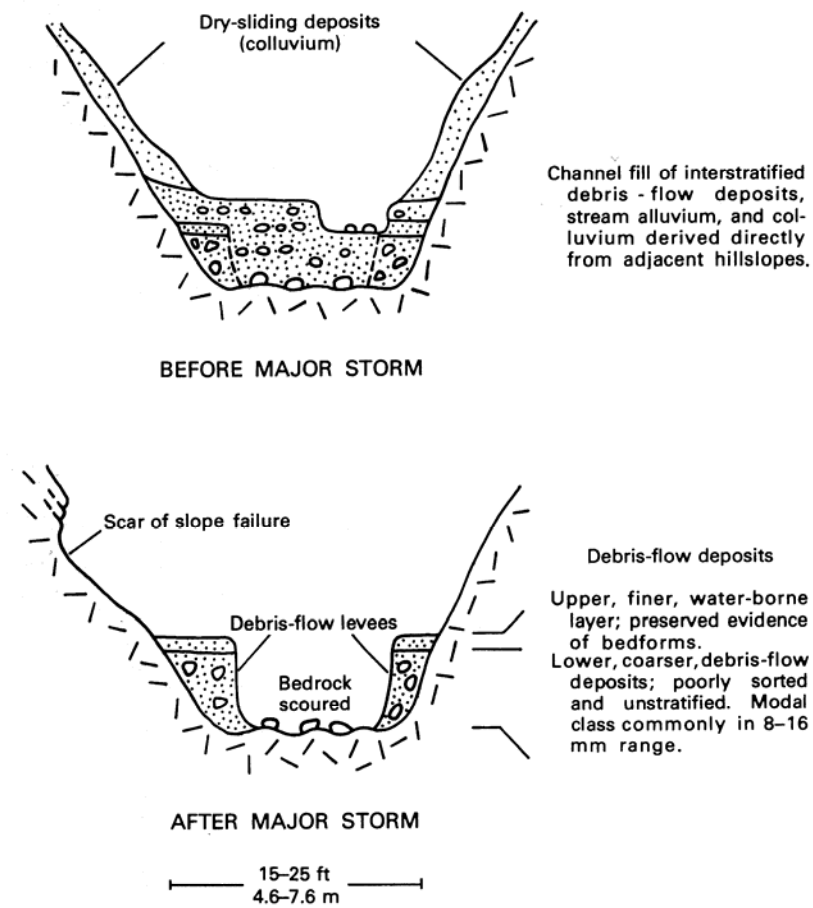


FIGURE 7.—Diagrammatic cross sections of typical bedrock-channel deposits in an area of high erosion rates in the Transverse Ranges.

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 (1988) 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 (1988) 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 5.2 where the years corresponding to the five floods were the only years to show significant sediment transport.

Hill and McConaughy (1988) determined that over 98% of the total sediment load in the Ventura River and San Antonio Creek is suspended. Approximately 96 % of the 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.

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 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 1.1. The results of applying Scott and Williams (1978) 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 (1978) for sediment production could significantly underestimate the amount of fine sediment.

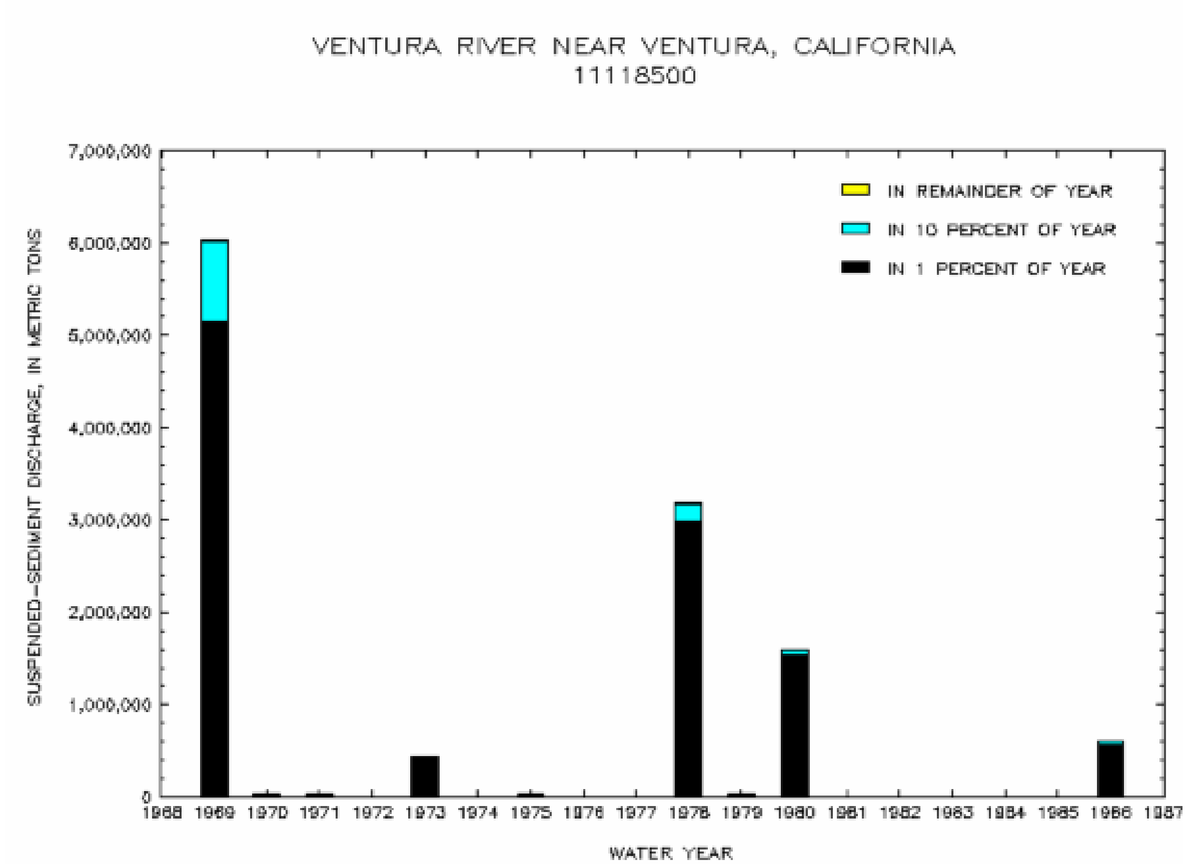


Figure 5.2. Suspended Sediment Loads in Ventura River. There was no data recorded from 10/1/73 to 9/30/74 and from 10/1/82 to 9/30/85 (figure from USGS http://webserver.cr.usgs.gov/sediment/). The year 1983 had substantial flow and sediment transport.

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 surface 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 5.3. 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.

In addition to the surface bed material data, six sub-surface bed material samples were also collected. These were collected by carefully removing the surface layer from a two foot by two foot square by hand, assuming that the surface layer is approximately the thickness of the largest size present. A sample weighing approximately 20 to 30 lbs was then collected with a hand shovel and the sample was sent to the lab for particle size analysis.

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

Surface 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. A Softball and Tape Measure are shown in Figure for Scale.

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.

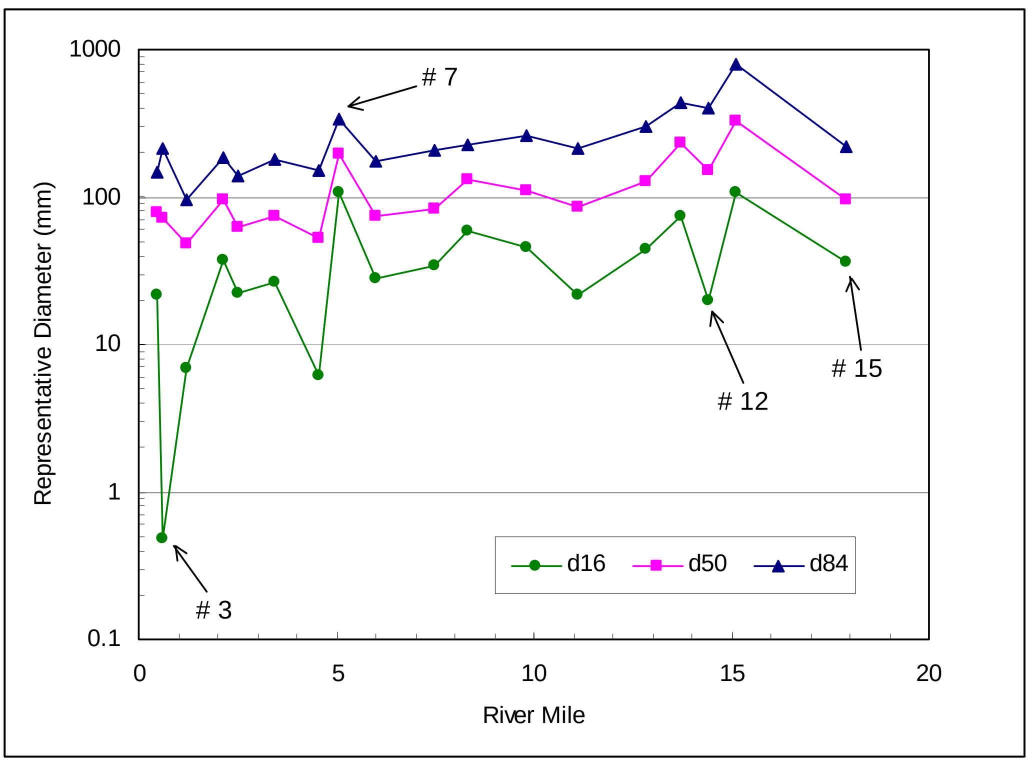


Figure 5.4. Measured representative diameters of surface bed material samples.

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 few exceptions to this correlation between slope and particle size were stated previously.

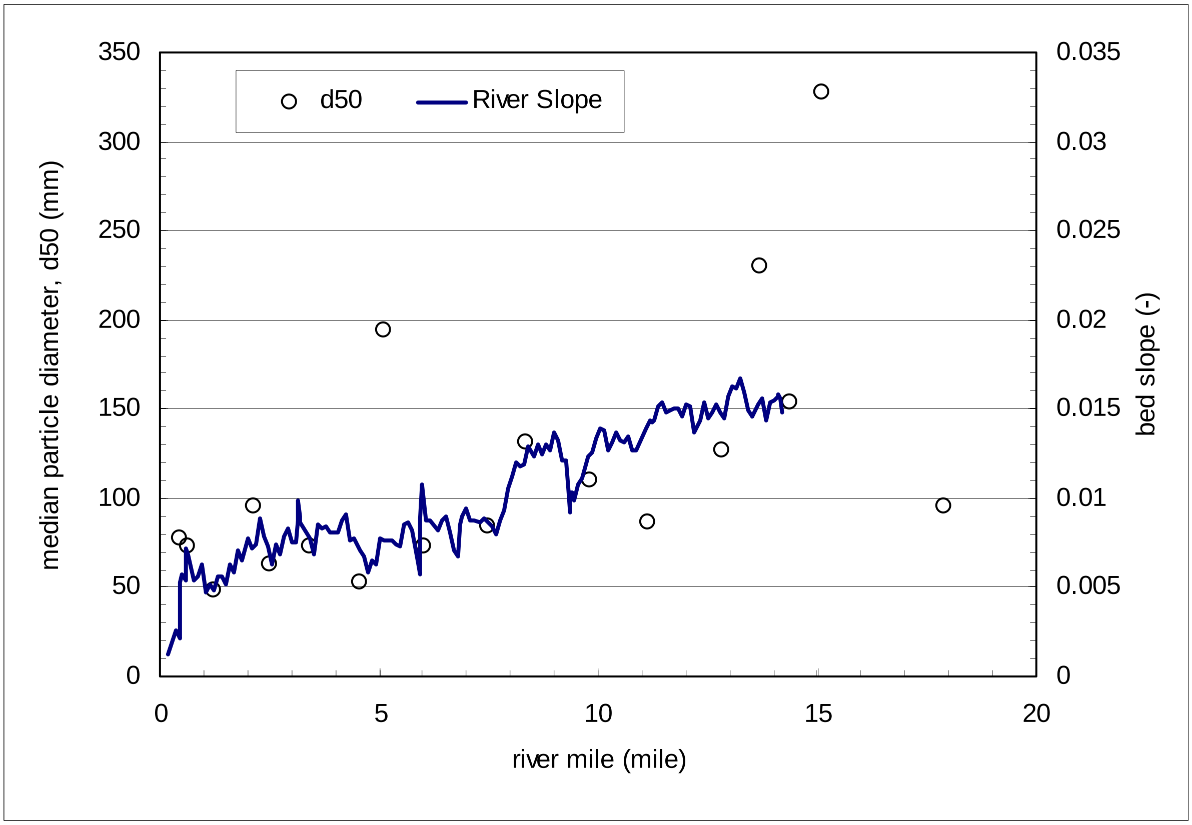


Figure 5.7. Average Bed Slope and d_{50} of Bed Material Samples.

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.

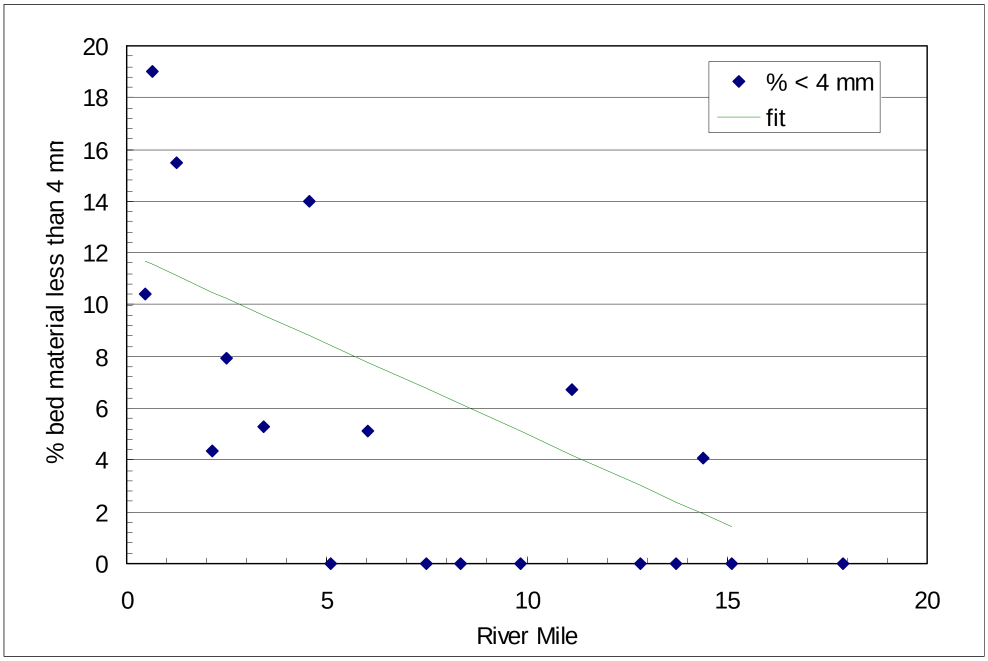


Figure 5.8. Fraction of Bed Material Less than 4 mm.

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.

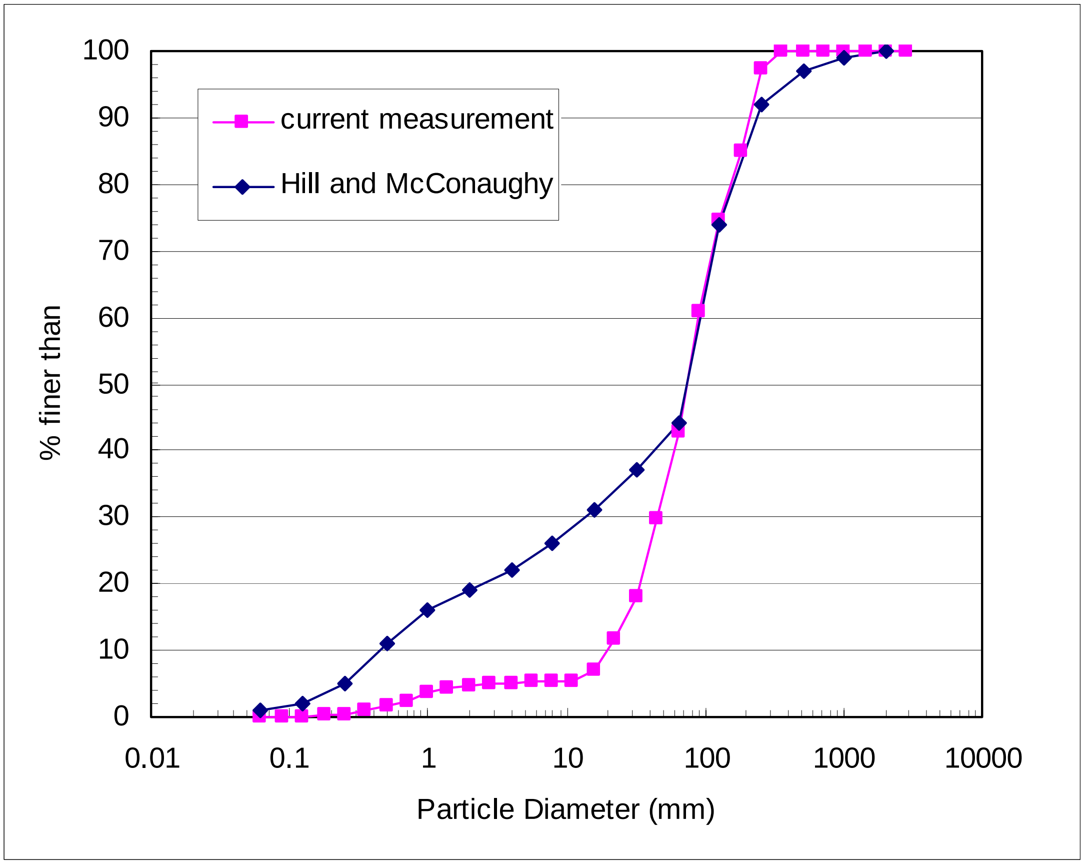


Figure 5.9. Comparison between USGS Composite Sample and Current Measurements of Bed Material near USGS Gage on the Ventura River near Foster Park.

Sub-Surface Sampling Results

The sub-surface gradations are given in Figure 5.10. There were 4 samples collected in the Ventura River and samples also collected in North Fork Matilija Creek and San Antonio Creek. The sub-surface bed material becomes finer as one goes downstream in the Ventura River, and San Antonio Creek is considerably finer than North Fork. The d_{20} for San Antonio Creek is about 2 mm, while for North Fork Matilija Creek it is 10 mm. The d_{20} of the Ventura River sub-surface bed material is approximately 6 mm above San Antonio Creek, and approximately 3 to 4 mm for the reach below San Antonio Creek.

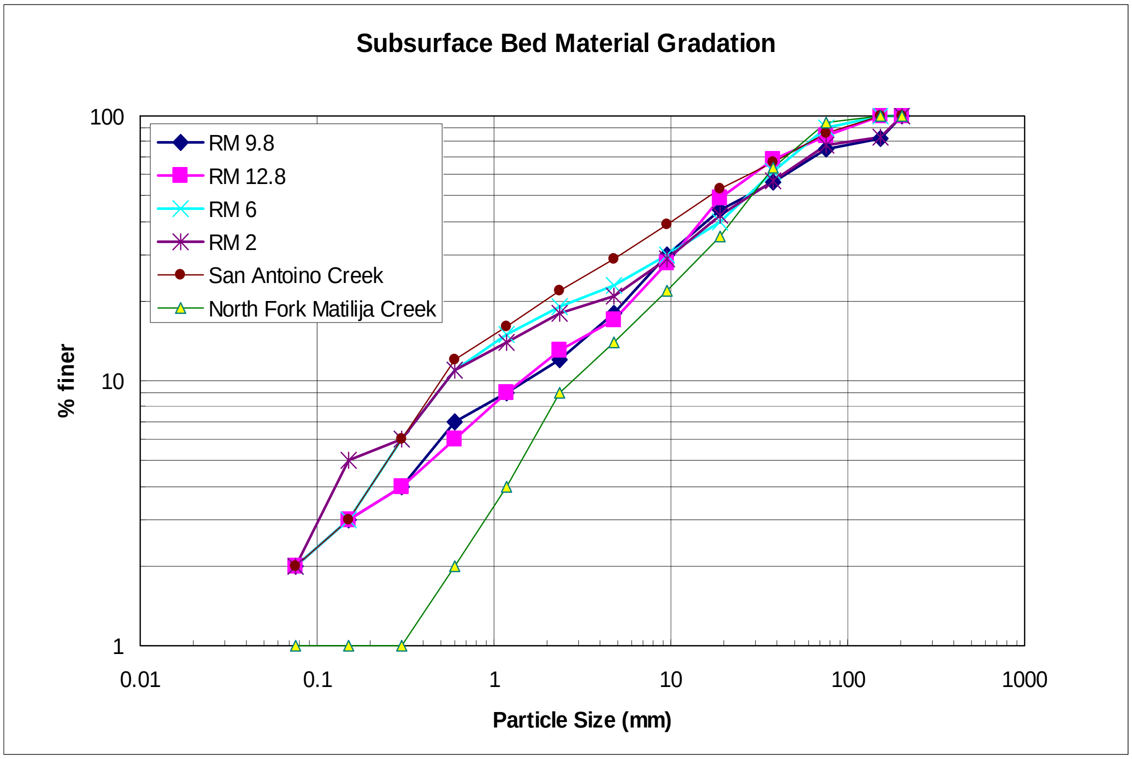


Figure 5.10. Sediment Gradation of Sub-Surface Material in Ventura River.

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). A photograph of the current reservoir is shown in Figure 5.11. 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. Figure 5.12 shows the profile of sedimentation from 1947 until 1999.

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). The estimated trap efficiencies 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 Brune 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 5.13. 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 5.11. 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
2006	1097.6	470	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 and 2005/2006 survey).

	Active Storage Volume (ac-ft)			
Elevation (NAVD 88)	1970	1983	1994	2006
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	1
1077.6	1199	468	153	20
1082.6	1479	662	283	103
1087.6	1789	906	447	213
1092.6	2121	1190	666	340
1097.6	2473	1480	930	477

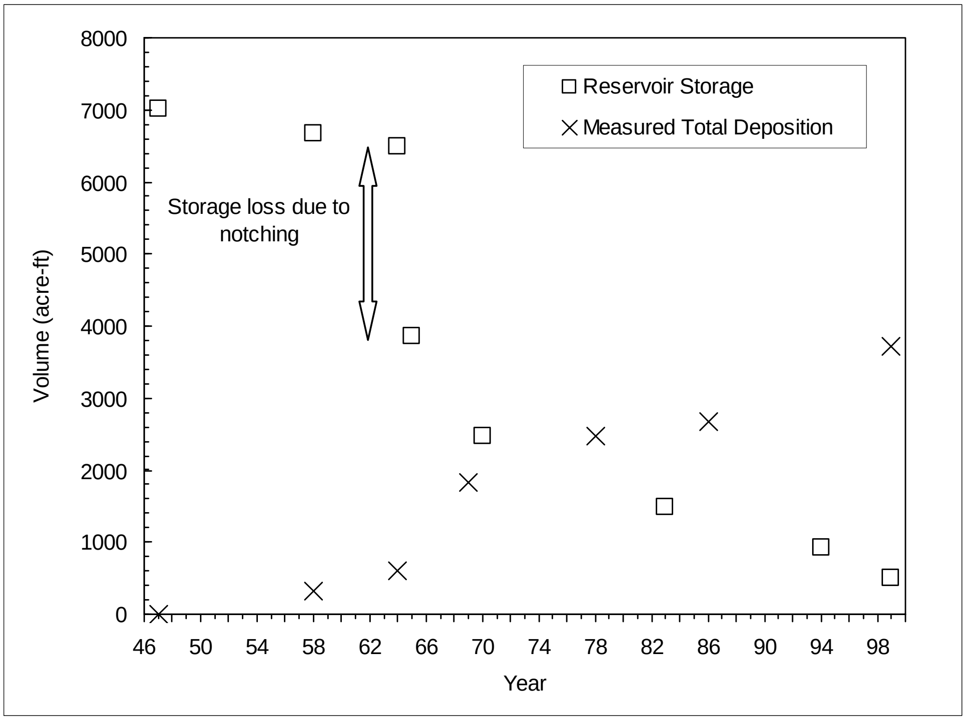


Figure 5.12. Plot of Matilija reservoir storage and deposition.

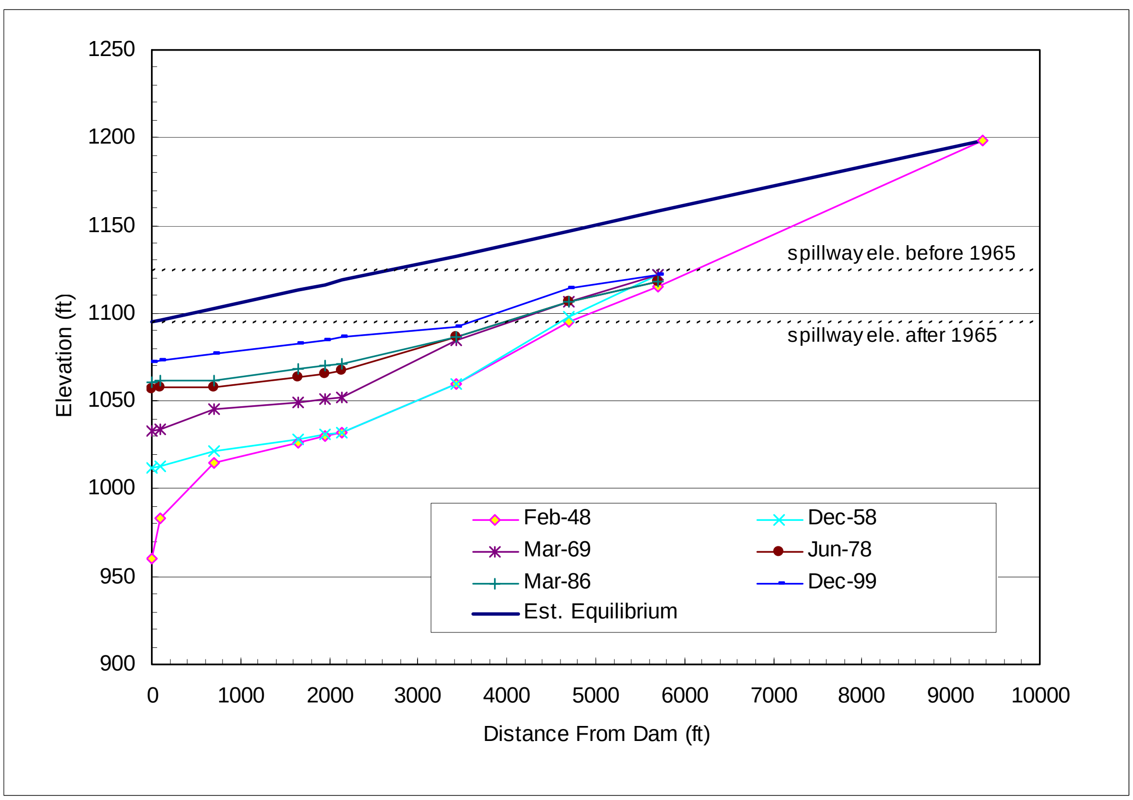


Figure 5.13. Profile plot of depositional history.

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

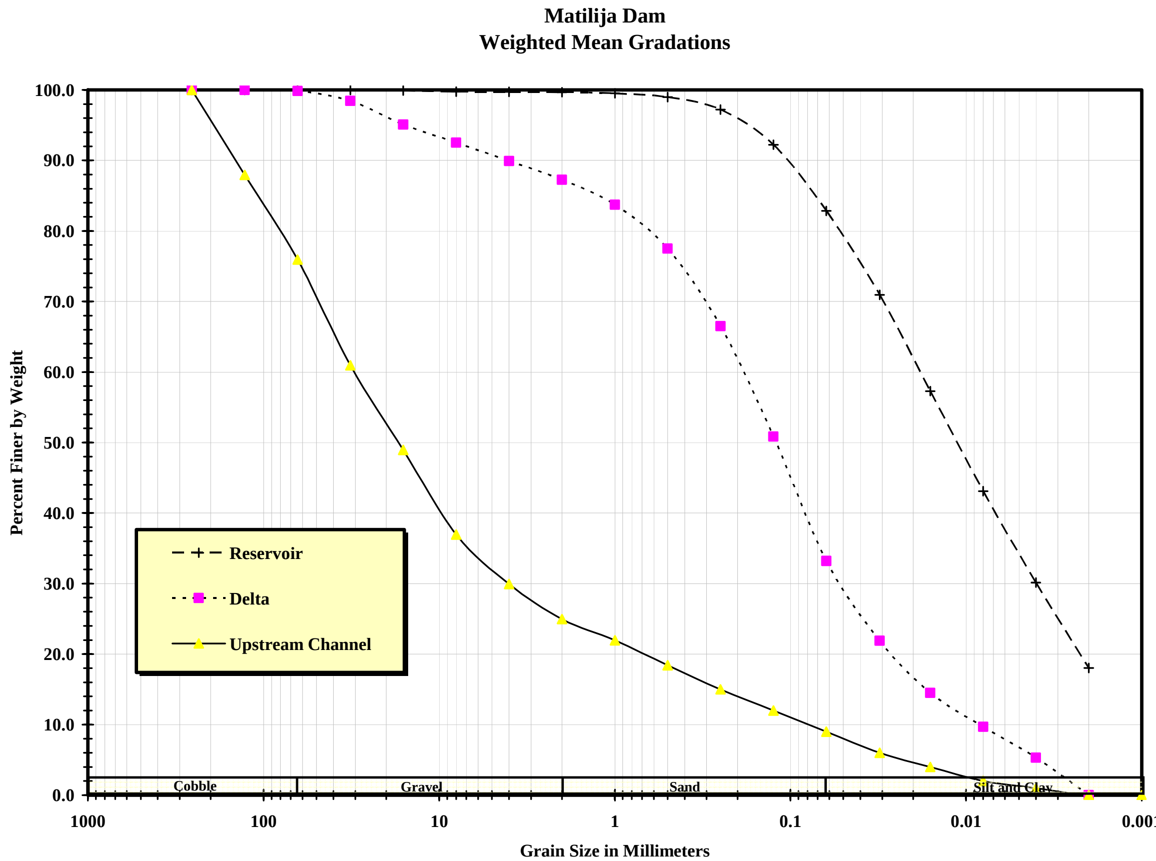


Figure 5.14. Average size gradations of reservoir deposits.

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	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. The Lara and Pemberton (1963) method of calculating the bulk density can be used as an estimate. 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.

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.

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.

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

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

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.

The quantity of sediment transported by the prevailing range of stream flows has been determined at four gaging stations in the Ventura River Basin, three tributaries; the Matilija Creek near Matilija Reservoir near Matilija Hot Springs (11114495), North Fork Matilija Creek near Matilija Hot Springs (11116000), and San Antonio Creek near Casitas Springs (11117500), as well as the Ventura River near Ventura (11118500). The Matilija Creek gaging station is located 1.4 miles upstream of a discontinued gage, Matilija Creek above the reservoir near Matilija Hot Springs (11114500). The difference in contributing area to the currently active gage compared to the discontinued gage is less than 6 percent. The locations of these gaging stations as well as other gaging stations with no sediment record are shown in Figure 2.1.

The sediment particles carried by rivers and streams are transported suspended with the water column or rolling and saltating along the bed. When the settling velocity of a sediment particle, which is primarily a function of the particle diameter, is less that the turbulent velocity fluctuations in the flow, the particle will be suspended in the flow. Conversely, when the settling velocity of a sediment particle is greater than the turbulent velocity fluctuations of the flow, the particle will be transported along the streambed and, at least, occasionally in contact with the bed. Sediment particles suspended in the flow are called suspended load, whereas those particles in occasional to continuous contact with the streambed are called bedload. Except in very unusual conditions, the vast majority, >90%, of all sediment carried over a typical annual range of discharges is transported in suspension. A portion of the suspended particles are sufficiently small, typically less than 0.062 mm in diameter, and do not settle through the flow quickly enough to accumulate on the streambed to any appreciable degree. This fraction of the suspended load is called washload. For a given sized particle, the distinction between washload and suspended load, or suspended load and bedload will depend on the turbulent intensity of the particular flow. For the purpose of this analysis sediment particles less than 0.062 mm in diameter, i.e. clay and silt, will be treated as washload, 0.062 – 1.00 mm, i.e. fine to medium sand, will be treated as suspended load, and particles greater than 1 mm will be treated as bedload.

In general, the most cost-effective and accurate approach to determine the quantity of suspended sediment transport at a given stream location and discharge is empirically by collecting a discharge-weighted sample of the flow. In particular, the great uncertainty involved with characterizing the supply of the washload to the channel at a given time and location precludes a computational approach. In contrast, the most cost-effective and accurate approach to determine the quantity of bedload sediment transport at a given discharge is by applying the most appropriate of several equations that relate the hydraulic characteristics of a given flow to the transport rate of bedload sized sediment that occur on the streambed. Consistent with the conventional approach, the concentration of suspended sediment has been sampled over a range of stream flows at four of the gaging stations in the Ventura River Basin, see Table 5.10. In contrast, bedload transport has not been sampled consistently at any of the gaging stations; therefore, a suitable computational approach will be employed. Except for the suspended sediment samples collected at the Matilija Creek near reservoir near Matilija Hot Springs, the percent of particles finer than 0.062 mm in a majority of the suspended sediment samples has been determined. Accordingly, the variation of silt and clay, particles <0.062mm, and sand, particles >0.062mm, concentration with discharge will be analyzed separately at three of the tributary gages as well as the Ventura River near Ventura gage.

The period of record when stream flows have been recorded and suspended sediment concentrations sampled are summarized in Table 5.10. These gaging stations are the only sites in the Ventura River Basin when the concentration of suspended sediment has been sampled an appreciable number of times. Typically, the concentration of suspended sediment varies significantly within a river cross-section. Accordingly, water sampling equipment and techniques have been developed by the Federal Interagency Sedimentation Project to collect a discharge-weighted sample of the flow that can be analyzed to determine to average concentrations of suspended sediment. The methods and techniques used to sample and determine the concentrations of suspended sediment considered in the report are described by Guy and Norman (1970), Porterfield (1972), and Edwards and Glysson (1988).

A detailed compilation of suspended sediment concentrations sampled at each gaging station prior to Oct. 2004 is available at http://nwis.waterdata.usgs.gov/ca/awis/qwdata. For this investigation, additional suspended sediment samples were collected during the WY 2005 at the North Fork Matilija Creek and San Antonio Creek gaging stations. All suspended sediment samples collected at the North Fork Matilija Creek at Matilija Hot Springs are listed in Table 5.10 and the San Antonio Creek at Casitas Springs gage are listed in Table 5.11. Suspended sediment rating curves for the suspended sediment concentrations of the form of Eq. 5.2 were developed at each of the four gage locations.

Bed load versus flow discharge was computed for North Fork Matilija and San Antonio Creek. The method used to bed load transport in North Fork Matilija and San Antonio Creek require an analysis of the shear stresses on the bed. Fluid forces acting parallel to the wetted perimeter of a channel are balanced by several sources of flow resistance, including form drag exerted by bars, bed form, bank irregularity etc., and grain resistance. The fluid forces per unit area of the channel is the shear stress, τ . Only a portion of the total shear stress sometimes called the skin friction shear stress, τ_{sk} is exerted on the bed-material. Meyer-Peter and Muller (1948), Einstein (1950), and England and Hansen (1967) developed and refined an approach to calculated the skin friction shear stress where the mean velocity, $\bar{u}$, hydraulic radius, R, slope, S, and size distribution of bed-material are known for a given discharge, Q. The England and Hanson approach was applied in this investigation to calculate the skin friction shear stress for each discharge measurement made at a gaging station over the period of record of stream flows greater than approximately the mean annual value. The skin friction shear stress,τ_{sk}, is given by

$$\tau_{sk} = \rho g R' S \qquad (1)$$

where ρ is the fluid density, g is the acceleration of gravity, and R' is the hydraulic radius sufficient to support the observed mean velocity at the given energy slope of grain resistance was the only source of flow resistance. An equation for the R' in terms of the mean velocity and other known values is obtained by integrating the logarithmic velocity profile over the water column and letting $z_o = 2d_{65}$ where $u(z_o) = 0$ and d_{65} is diameter of the 65th percentile fraction of the bed material. Hence,

$$\frac{\bar{u}}{\sqrt{gR'S}} = 2.5\, ln \frac{(6R')}{d_{65}} \qquad (2)$$

Equation (2) is solved interactively for R', which is then substituted into equation (1) to determine the skin friction shear stress. The dimensionless skin friction shear stress for the i^{th} percentile bed particle, $\tau_{sk}(i)$, is

$$\tau^*_{sk}(i) = \frac{\tau_{sk}}{(\rho_s - \rho)\, g\, d_i} \qquad (3)$$

Where ρ_s is the sediment density and d_i is the i^{th} percentile bed particle. Equation (3) shows that $\tau^*_{sk}(i)$ is the ratio of the fluid forces tending to initiate particle motion versus the gravitational force tending to maintain the particle at rest.

Parker (1990) formulated an empirical bedload transport function for poorly sorted mixtures of gravel and cobbles referenced to the particle size distribution of the surficial bed material. The Parker bedload function is

$$W^*_i = 0.00218 G(\phi_i) \qquad (4)$$

$$G(\phi) = \begin{cases} 5474\left(1-\dfrac{0.853}{\phi_i}\right)^{4.5} & \phi_i > 1.59 \\ \exp[14.2(\phi_i - 1) - 9.28(\phi_i - 1)^2] & 1 \le \phi_i \le 1.59 \\ \phi_i^{14.2} & \phi_i < 1 \end{cases} \qquad (5)$$

Where $W_i^* = [q_{bi}(\rho_{s/\rho} - 1)]/[f_i g^{1/2} (RS)^{3/2}]$, $\phi_i = \tau_{sk}^*(i)/\tau_{rsk}^*(i)$, and q_{bi} is the volumetric bed load transport rate of the *i*th particle fraction per unit width. This bed load function is similar to a function derived by Parker et al. (1982) referenced to the particle size distribution of subsurface bed material. Parker et al. (1982) found that the use of ϕ_i, rather than $\tau_{sk}^*(i)$, resulted in a similarity collapse, so that W_i^* is approximately a single-valued function of ϕ_i. The Parker bedload function for the domain $\phi_i > 1.59$ was derived by fitting ϕ_i and W^* to the Einstein (1950) bedload function and is in excellent agreement with most laboratory flume measurements of gravel transport including the measurements reported by Meyer-Peter and Muller (1948). For the domain $1.0 < \phi_i < 1.59$ the Parker bedload function was derived from bedload transport rates measured in Oak Creek (Milhous, 1973).

An essential aspect of this approach was the development of a reference dimensionless shear stress $\tau_{rsk}^*(i)$ such that:

$$\tau_{rsk}^*(i) = \tau_{rsk}^*(50)(d_i / d_{50})^{-b} \qquad (6)$$

where d_i is the diameter of particles of the *i*th size fraction of bed material and d_{50} is the median particle diameter of bed material. The values of the coefficients, $\tau_{rsk}^*(50)$ and *b*, vary somewhat from river to river depending primarily on the range of particle sizes in the streambed, particle shape, and bed-material packing. For this investigation, an average of coefficient values determined by an analysis of 6 rivers with poorly sorted bed-material were applied, such $\tau_{rsk}^*(50) = 0.0376$ and b = -0.994, Andrews and Nankervis (1995).

5.5.1. Discussion of Individual Rivers

Matilija Creek

The variation in the concentration of suspended sediment with stream flow at the Matilija Creek near reservoir near Matilija Hot Springs (11114495) is shown in Figure 5.15. The percent of particles finer than 0.062 mm has only been determined in 4 of the 25 suspended sediment samples collected at the Matilija

Creek. Therefore, it is not possible to analyze the concentration of sand and silt and clay separately. A curve has been fit to the log-transformed values using the method of least-squares regression. The regression demonstrates that stream flow explains 67 percent of the observed variation in suspended sediment.

Bedload transport at the Matilija Creek near reservoir near Matilija Hot Springs has not been analyzed to date. We expect to complete this analysis in late 2006.

North Fork Matilija Creek near Matilija Hot Springs

The variation in the concentration of suspended silt and clay with stream flow at the North Fork Matilija Creek near Matilija Hot Springs is shown in Figure 5.16. A curve has been fit to the log-transformed values for the 1956 to 1959 water year observations using the method of least-squares regression. The regression analysis demonstrates that stream flow explains only 35 percent of the observed variation in suspended silt and clay concentration. At a given stream flow, the observed concentrations of silt and clay during the 2005 water year are considerably less than had been observed during the 1956 to 1959 water years. Given the relatively few concentration samples collected during widely spaced periods, it is difficult to interpret the results of Figure 5.16. The weak correlation between stream flow and suspended silt and clay concentration probably reflects a relatively small quantity of silt and clay stored in the channel compared to the quantity of material supplied to the channel from time to time by adjacent hillslopes.

The variation in the concentration of suspended sand with stream flow at the North Fork Matilija Creek near Matilija Hot Springs is shown in Figure 5.17. At the observed stream flow, 6 of the 7 suspended sand concentrations sampled during the 2005 water year are considerably less than had been observed during the 1956 to 1959 water years, although the difference is not as great as the difference in silt and clay concentrations for the two periods. A curve has been fit to the log-transformed values for the 1956 to 1959 water year period using the period of least-squares regression. The regression analysis demonstrates that stream flow explains 69 percent of the observed variation in suspended sand concentration.

Computed whole-channel bedload transport rates over the historical range of stream flows sufficient to move bed particles for the 7 size fractions of bed-material in the North Fork Matilija Creek near Matilija Hot Springs are shown in Figure 5.18. The computed transport rates of very coarse bed material, those with intermediate axis > 256 mm, are consistent with conditions on January 9, 2005, at stream flows of nearly 3,000 ft^3/s, when cobble and boulder sized bed particles were observed in motion.

San Antonio Creek

The variation in the concentration of suspended silt and clay with stream flow at the San Antonio Creek near Casitas Spring is shown in Figure 5.19. A curve has been fit to the log-transformed values using the method of least-squares regression. The regression demonstrates that stream flow explains 91 percent of the observed variation in suspended silt and clay concentration. Observed silt and clay concentrations during the 2005 water year are consistent with concentrations sampled during the 1977 and 1978 water years.

The variation in the concentration of suspended sand with stream flow at the San Antonio Creek near Casitas Springs is shown in Figure 5.20. A curve has been fit to the log-transformed values using the method of least-squares regression. The regression analysis demonstrates that stream flow explains 85 percent of the observed variation in suspended sand concentration. Observed sand concentrations during the 2005 water year are consistent with concentrations sampled during the 1977 and 1978 water years.

Computed, whole-channel bedload transport rates over the historical range of stream flows sufficient to move bed particles for the 6 size fractions of bed material at the San Antonio Creek near Casitas Springs are shown in Figure 5.21. The transport rate of a particular fraction of bed-material depended upon the fluid forces acting on the size fraction as well as the relative abundance of that size fraction in the streambed. Thus, the transport rate of particles in the 8 - 16 mm fraction is less than the transport rate of particles in the 16-32 mm fraction, because the latter size fraction is much more abundant in the streambed.

Ventura River

The variation in the concentration of silt and clay with stream flow at the Ventura River near Ventura is shown in Figure 5.22. A curve has been fit to the log-transformed values using the method of least-squares regression. The regression analysis demonstrates that stream flow explains 63 percent of the observed variation in suspended silt and clay concentration. The variation in the concentration of suspended sand with stream flow is shown in Figure 5.23. A curve has been fit to the log-transformed values using the methods of least-squares regression. The regression analysis demonstrates that stream flow explains 73 percent of the variation in suspended sand concentration.

The concentration of suspended sediment during periods of relatively high flow has been sampled, more or less, continuously since 1968. An inspection of Figure 5.22 and Figure 5.23 does not reveal any conspicuous periods when suspended sediment concentrations appeared to deviate consistently from the regression curve. Indeed, the time series of the residuals about either regression curve shown that the concentration of suspended sediment at a given stream flow has been remarkably consistent in spite of significant changes in the drainage basin, i.e. major fires, water and land development. The consistent relations between the concentrations of silt and clay, and sand with stream flow at the Ventura River near Ventura contrasts with the changes in these relations at the North Fork Matilija Creek near Matilija Hot Springs, as described above. The Ventura River channel is bounded by an extensive floodplain for several miles upstream of the

Ventura River near Ventura gaging station. The channel and floodplain through this reach contain many times the annual load of suspended sediment transport by the Ventura River gage. Consequently, year-to-year, even multi-year, variations in the supply of sediment to this reach do not alter appreciably the concentration of relatively fine sediment transported past the Ventura River near Ventura gage at a given stream flow. Although the floodplain and alluvial terraces are less extensive along San Antonio Creek compared to the Ventura River, they are nevertheless substantial and smooth out most of the year-to-year variations in sediment supply. In contrast, the channel of the North Fork Matilija Creek is bound almost continuously by colluvial hillslope and bedrock outcrops. Alluvial deposits composed of relatively fine sediment that can be transported in suspension are uncommon along the North Fork Matilija Creek. Accordingly, year-to-year variations in the supply of relatively fine sediment to the channel have an appreciate effect on the concentration of suspended sediment at a given stream flow.

Table 5.10. Summary of gaging station records in the Ventura River Basin, California where there has been suspended sediment samples collected.

Gaging Station Name	Gaging Station Number	Drainage Area mi^2	Period of Record		# Samples Analyzed for Concentration and (Particle Size)
			Stream flows	Suspend Sediment	
Matilija Creek nr Matilija Reservoir nr Matilija Hot Springs	11114495	47.8	2002-current	2002-current	25 (4)
North Fork Matilija Creek at Matilija Hot Springs	11116000	15.6	1929-35 1934-73 1974-current[1]	1955-59 2005-current	36 (25)
San Antonio Creek nr Casitas Springs	11117500	51.2	1949-current[1]	1977-1978 2005-current	26 (23)
Ventura River nr Ventura	11118500	188	1929-current	1968-current	231 (130)

[1] Gage operated by Ventura County since October, 1983.

Table 5.11. Summary of suspended sediment concentration samples collected at the North Fork Matilija Creek at Matilija Hot Springs gage.

Date	Time	Discharge ft^3/s	Suspended Sediment Concentration mg/L	Percent of Sample Finer Than 0.062mm
12/25/55	15:15	47	793	97
12/27/55	09:45	6.6	24	na
01/25/56	11:30	12	119	na
01/25/56	16:00	20	372	na
01/26/56	11:00	252	2440	na
01/26/56	15:10	340	3430	87
01/27/56	10:30	59	876	na
01/27/56	15:30	46	588	94
01/13/57	10:05	47	2800	88
01/14/57	11:15	7.8	292	na
02/23/57	10:00	53	3540	91
12/17/57	14:10	48	4140	96
01/26/58	09:30	15	1330	97
02/03/58	14:00	72	4580	84
02/04/58	14:15	1450	14400	72
02/06/58	13:50	50	1830	85
03/17/58	14:10	120	594	81
03/27/58	13:50	97	786	74
03/28/58	09:10	74	325	85
03/28/58	09:40	74	325	na
04/02/58	15:40	206	1710	75
04/03/58	15:10	1120	20600	72
04/18/58	12:00	64	269	na
04/18/58	23:59	64	269	na
01/06/59	10:55	16	867	99
02/11/59	10:35	61	911	93
02/17/59	14:35	33	205	na
02/21/59	11:15	45	463	95
04/01/59	12:15	4.2	2	na
01/09/05	13:00	2599	6204	64
01/09/05	13:00	2599	6073	64
01/10/05	15:30	2020	4126	56
01/10/05	16:00	1801	6157	33
01/11/05	11:30	540	702	68
01/11/05	11:30	540	733	63
01/12/05	14:30	254	284	89

Table 5.12. Summary of suspended sediment concentration samples collected at the San Antonio Creek near Casitas Springs gage.

Date	Time	Discharge ft^3/s	Suspended Sediment Concentration mg/L	Percent of Sample Finer Than 0.062mm
01/03/77	08:15	23	400	na
01/03/77	16:20	15	503	na
01/06/77	08:30	153	1110	99
01/06/77	11:00	164	1540	97
01/06/77	12:15	96	773	100
01/07/77	08:20	34	157	99
03/16/77	10:30	88	231	91
05/09/77	08:35	37	412	na
05/09/77	16:20	2.8	10	99
06/27/77	17:45	0.01	66	97
12/28/77	16:20	46	537	99
01/06/78	08:00	106	611	100
01/17/78	13:30	171	985	96
02/10/78	08:25	4980	22700	71
03/02/78	09:20	520	6160	62
12/28/04	10:30	936	7536	90
12/31/04	12:30	1201	14126	82
01/09/05	15:51	7875	28152	53
01/10/05	10:27	10100	41462	73
01/10/05	10:57	9994	49813	59
01/10/05	11:25	8087	46956	60
01/10/05	12:00	7204	48691	54
01/11/05	09:40	1409	41754	49
01/11/05	10:00	1331	27798	57
01/12/05	15:40	424	5633	53
01/13/05	11:30	266	1611	66

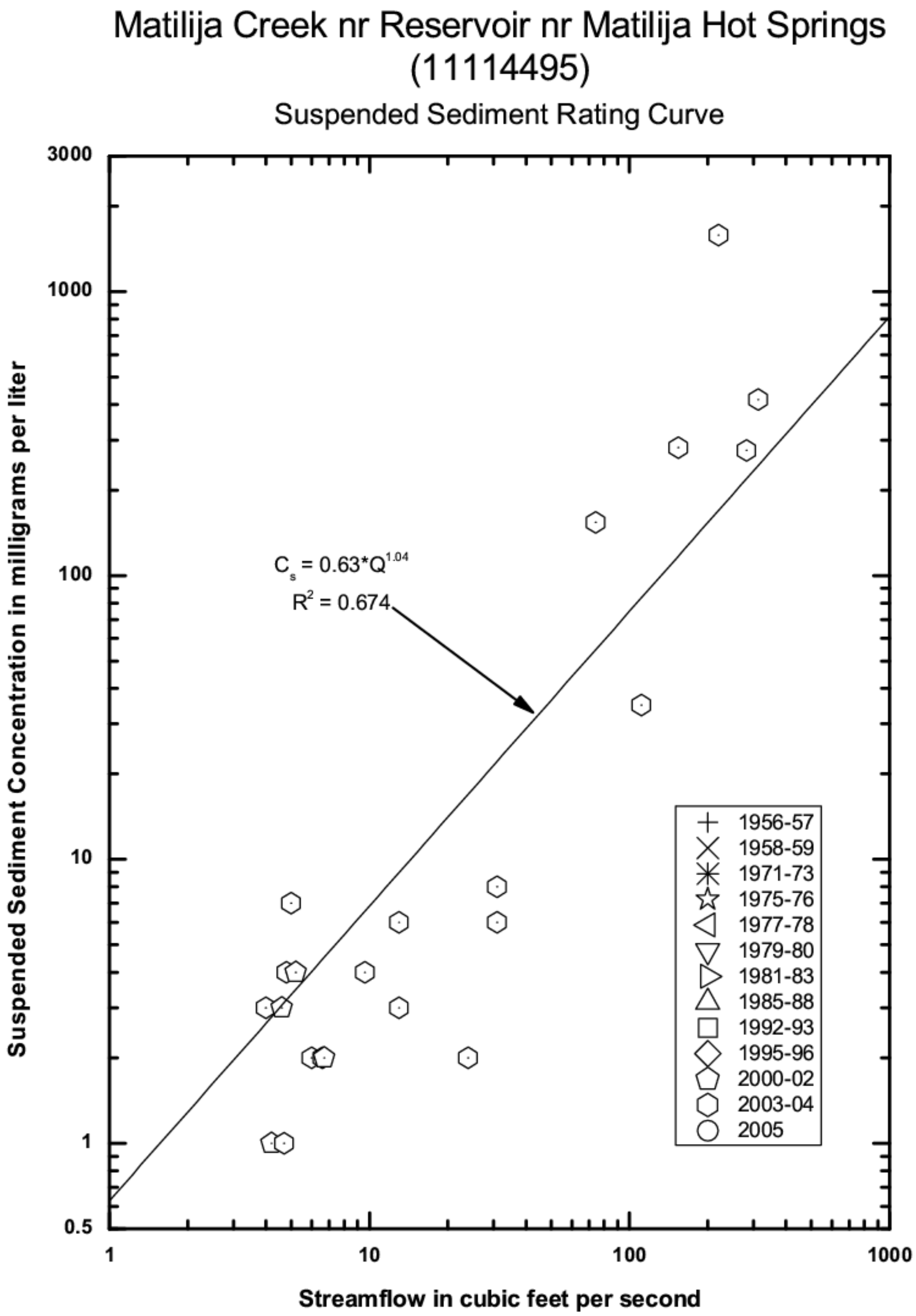


Figure 5.15. Total Suspended Sediment Versus Streamflow On Matilija Creek Upstream of Matilija Dam (USGS Gage #11114495).

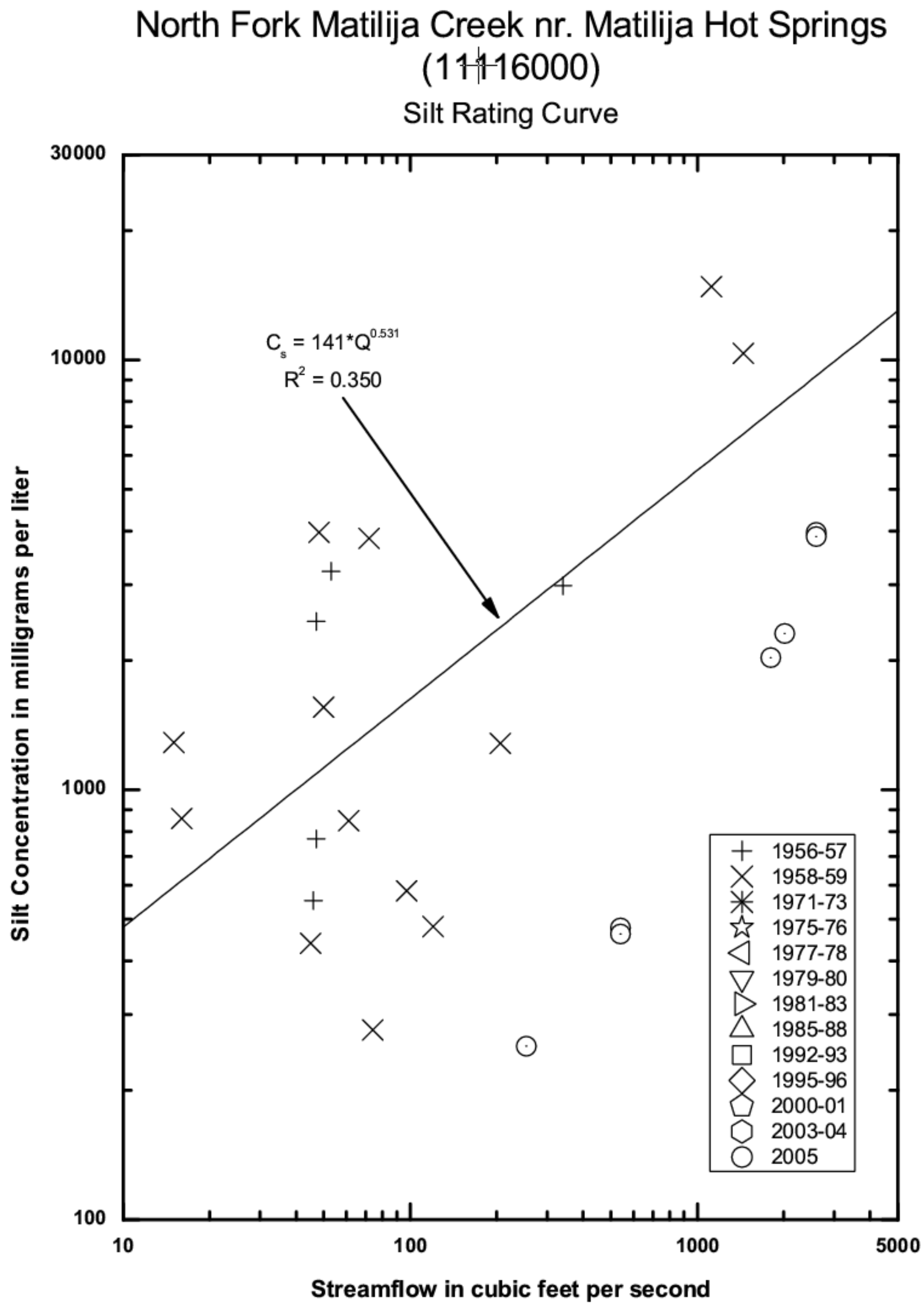


Figure 5.16. Silt Concentration on North Fork Matilija Creek.

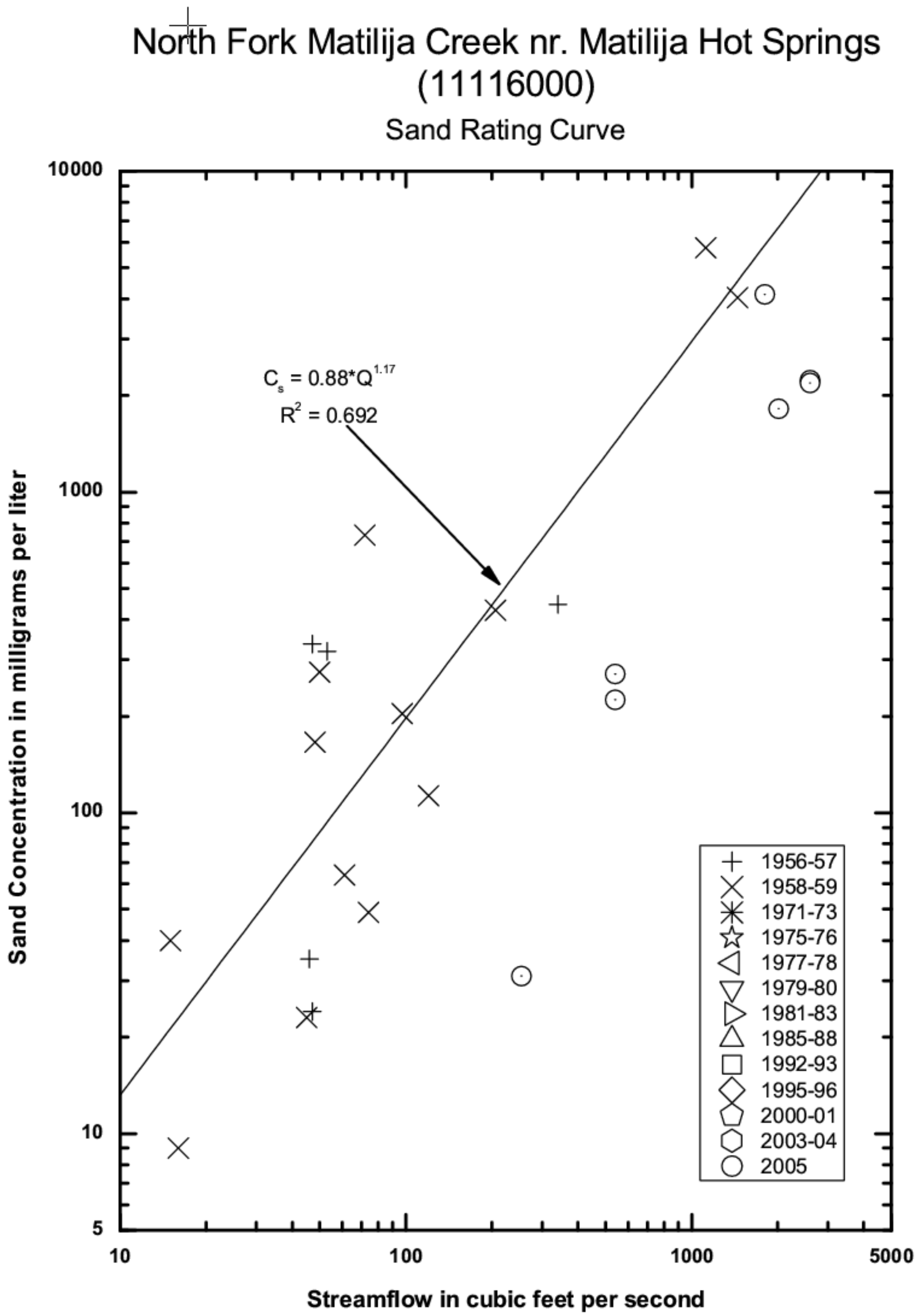


Figure 5.17. Sand Concentration on North Fork Matilija Creek.

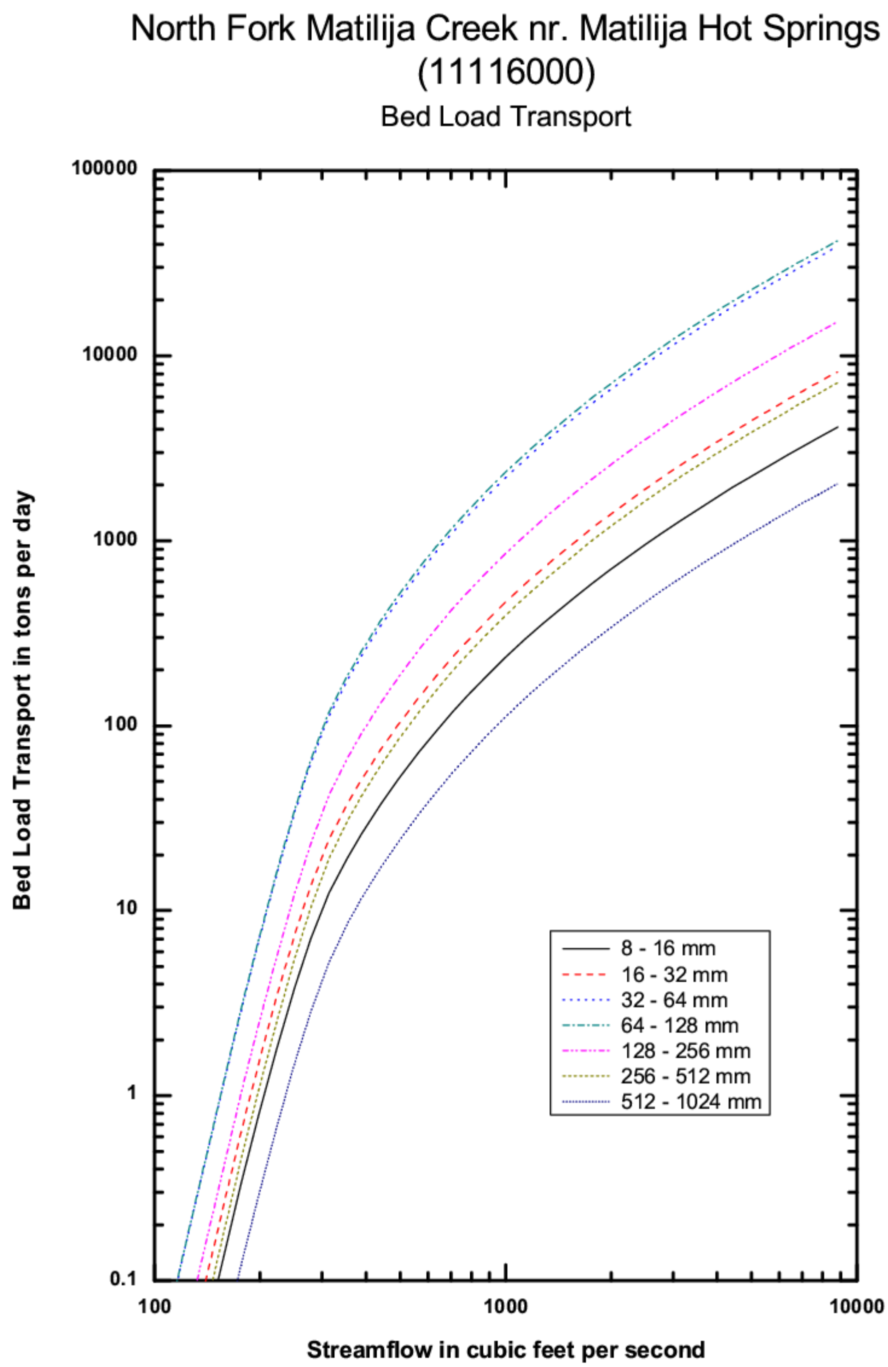


Figure 5.18. Computed Bed Load Transport on North Fork Matilija Creek.

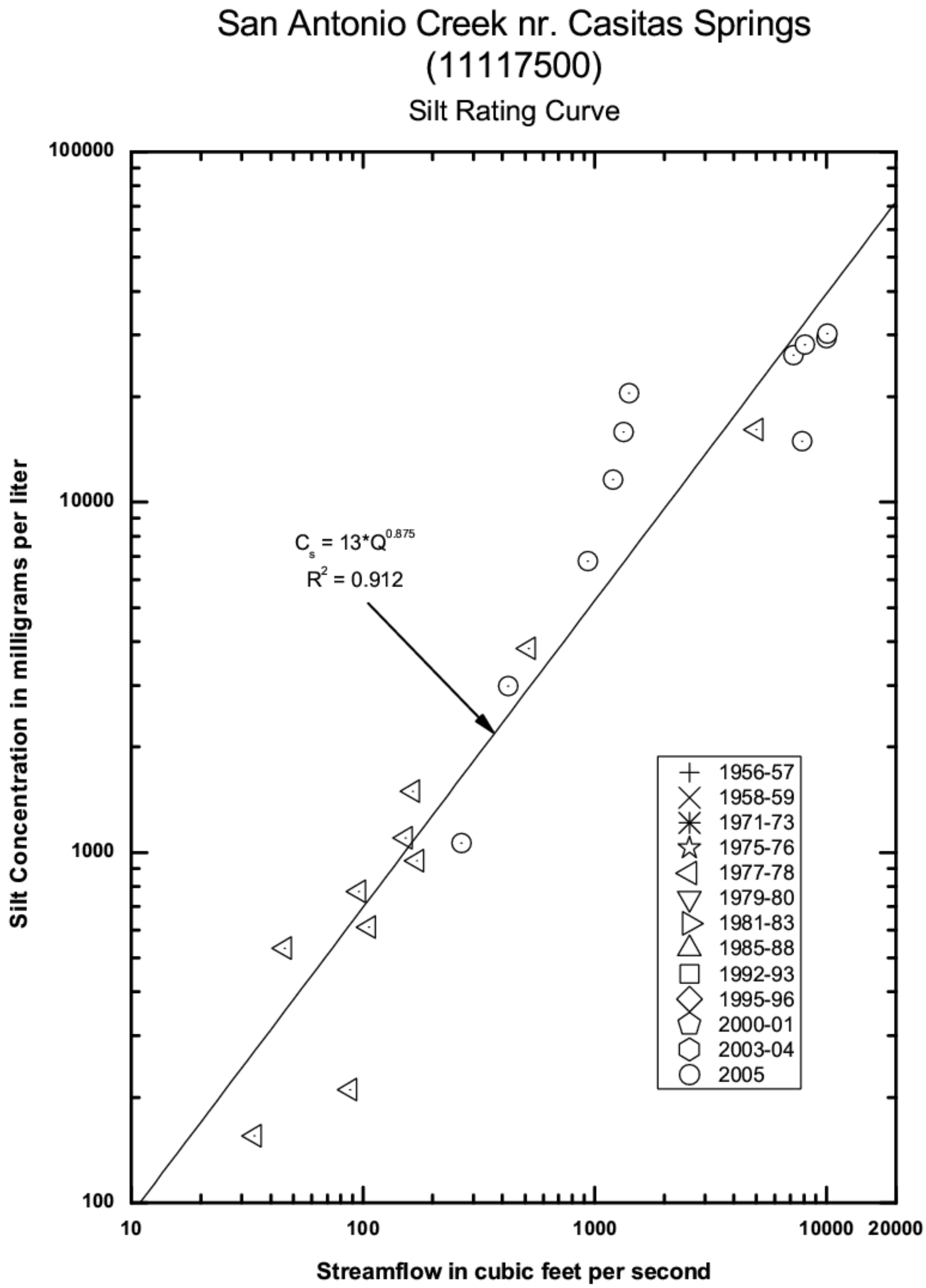


Figure 5.19. Silt Concentration on San Antonio Creek.

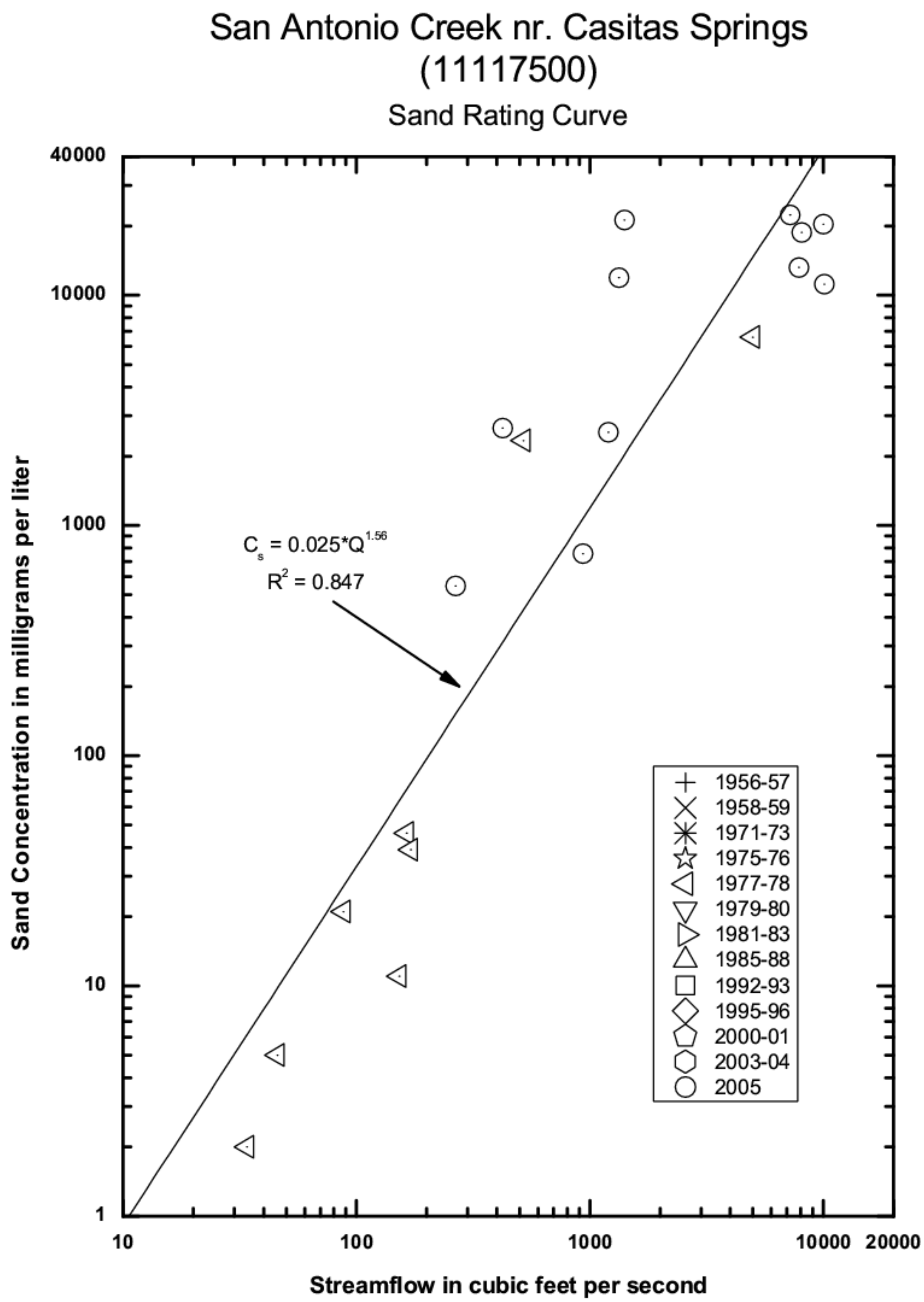


Figure 5.20. Sand Concentration on San Antonio Creek

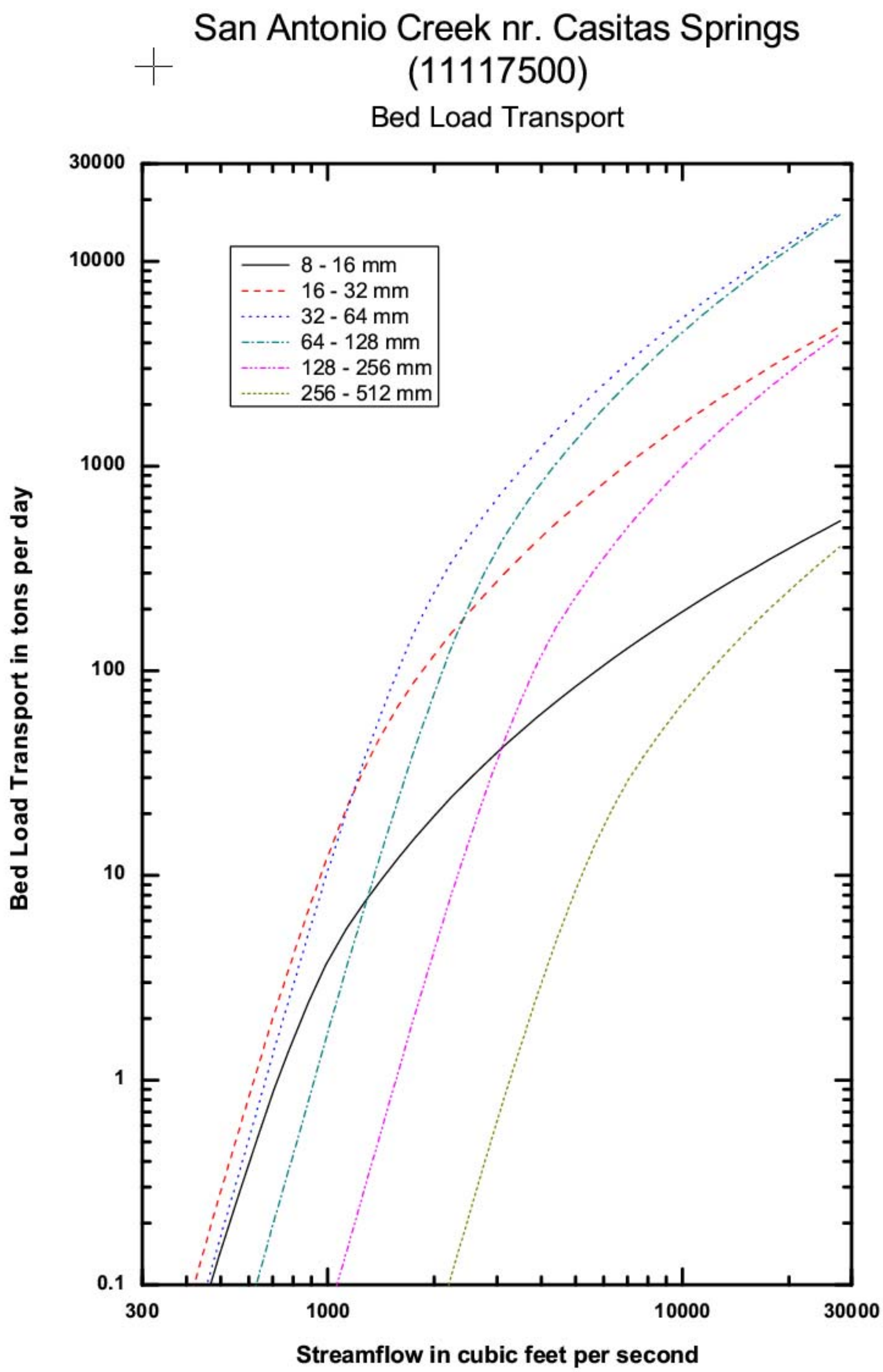


Figure 5.21. Computed Bed Load Transport on San Antonio Creek.

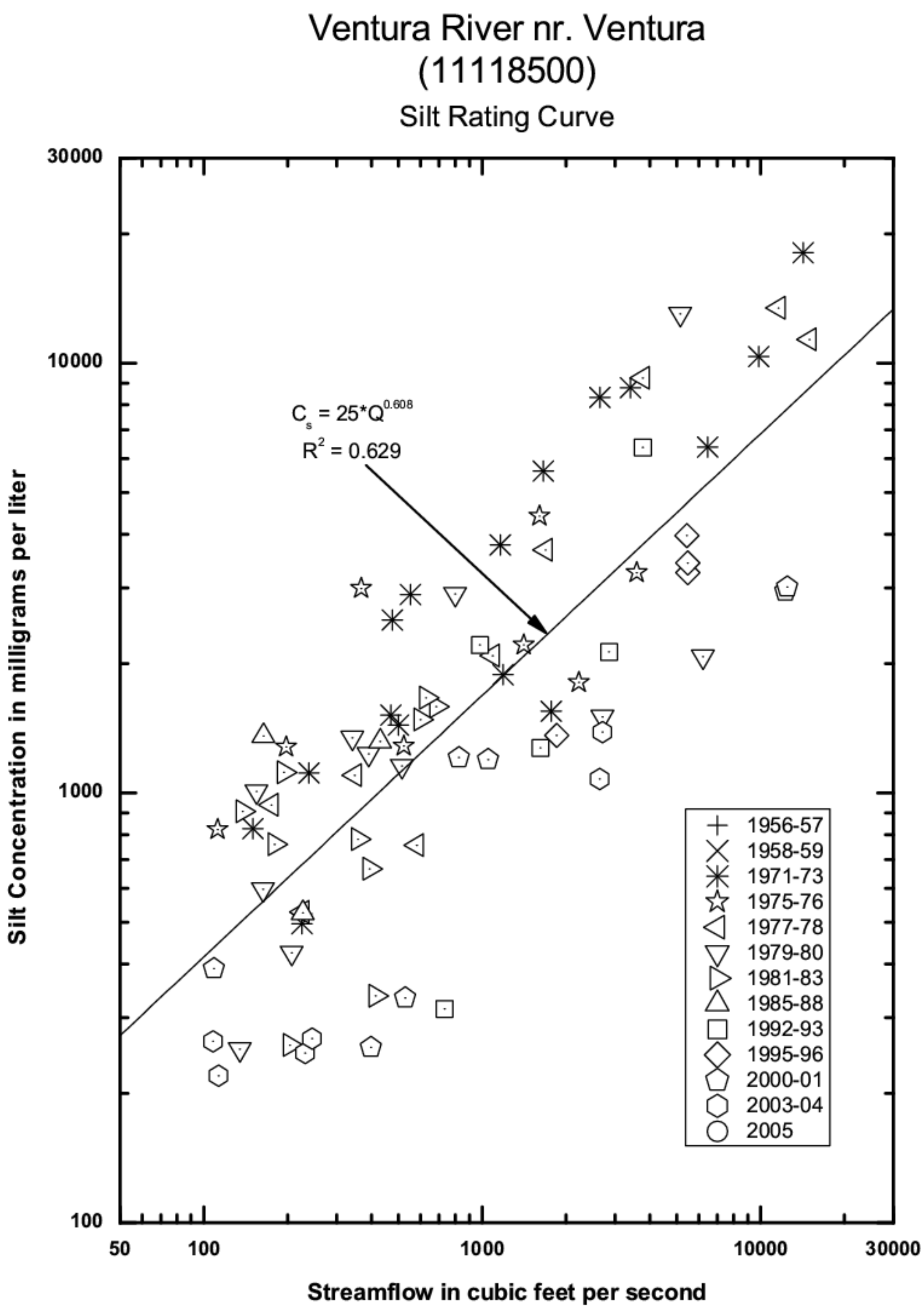


Figure 5.22. Silt Concentration on Ventura River at Foster Park.

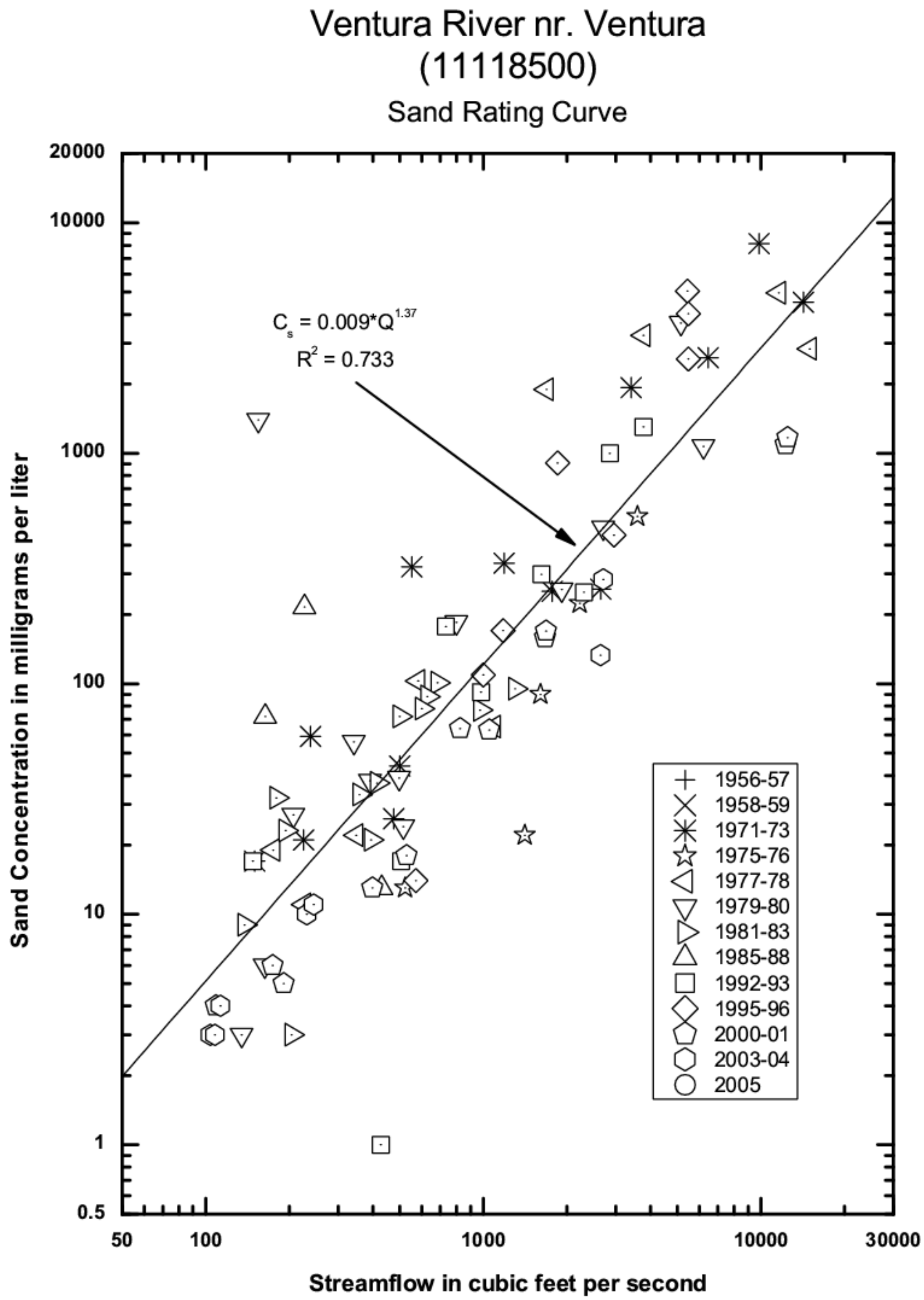


Figure 5.23. Sand Concentration on Ventura River at Foster Park.

5.5.2. Long Term Sediment Yields

The previous section described the sediment transport using measured sediment loads. To estimate yields 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 5.13. 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 5.13 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 5.14. 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 5.15). The ratio of sediment coarser than 0.125 mm to total sediment is 0.26, which 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 5.15.

Table 5.13. 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 5.14. 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 5.15. Average annual sediment delivery to the ocean.

type	yd^3/yr of sediment delivered				
	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

5.5.3. 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 area 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 there may be 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 5.16 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 5.17Table 5.16 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.

Using the data from Table 5.16 and Table 5.17, 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 5.24 presents the fire frequency in the Matilija Creek and Ventura River Watersheds.

Table 5.16. 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 5.17. 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

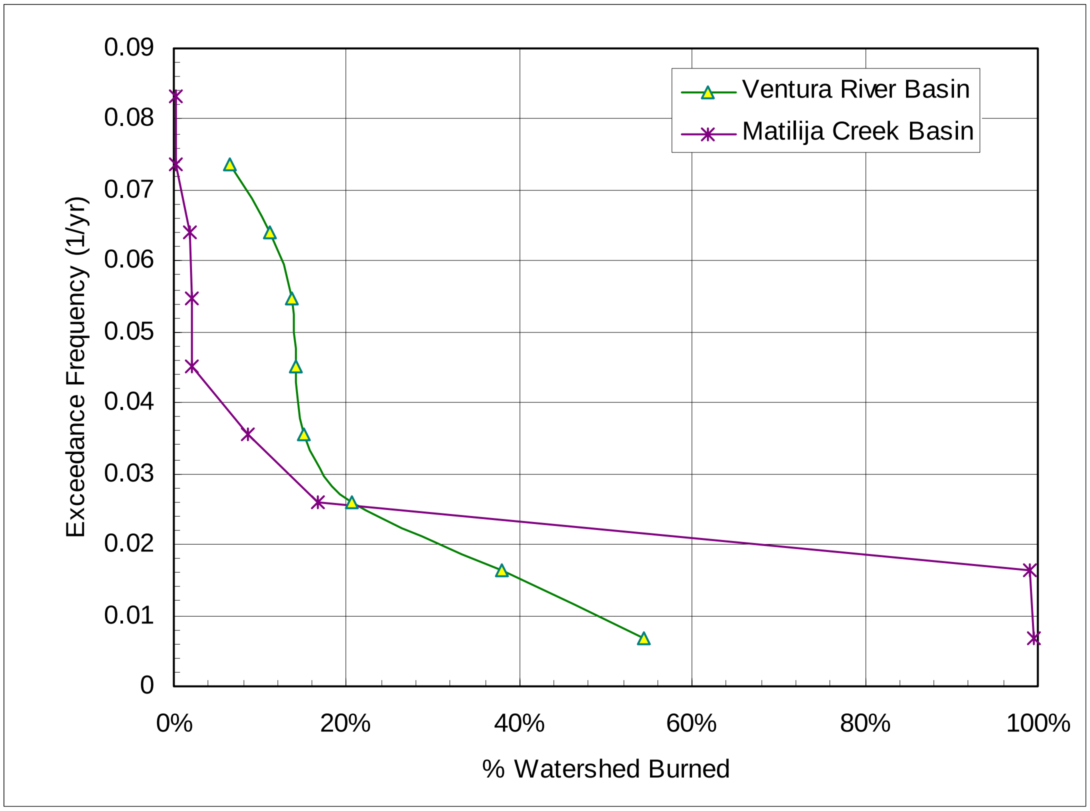


Figure 5.24. Fire frequency in the Matilija Creek and Ventura River Watersheds.

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 5.25) 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 θ_{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 5.25 and Figure 5.26.

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 (Pemberton and Lara, 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. 5.6).

The depths to an armored layer for the 2-, 10-, and 100-yr flows are found in Figure 5.27. 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 (Section 5.3). 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 (1999):

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

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

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

Eq 5.8

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 5.28. 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 5.29. 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 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 could 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 5.29.

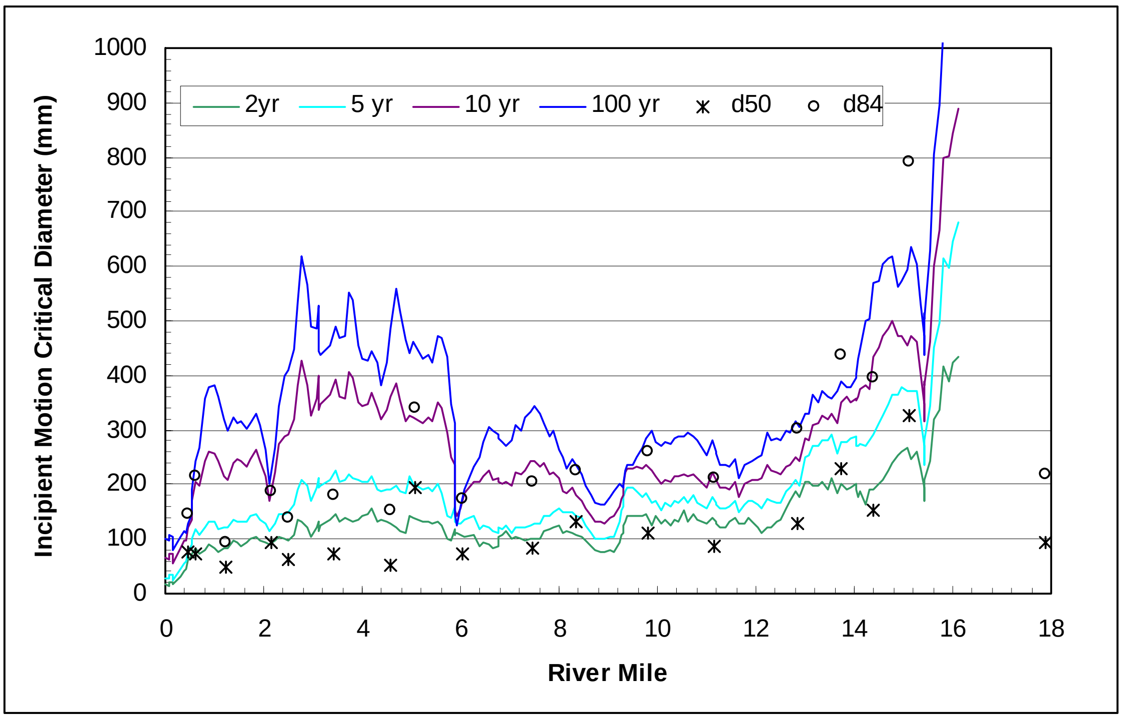


Figure 5.25. Incipient motion critical diameter for the Ventura River and comparison with the d_{50} and d_{84} of the bed material.

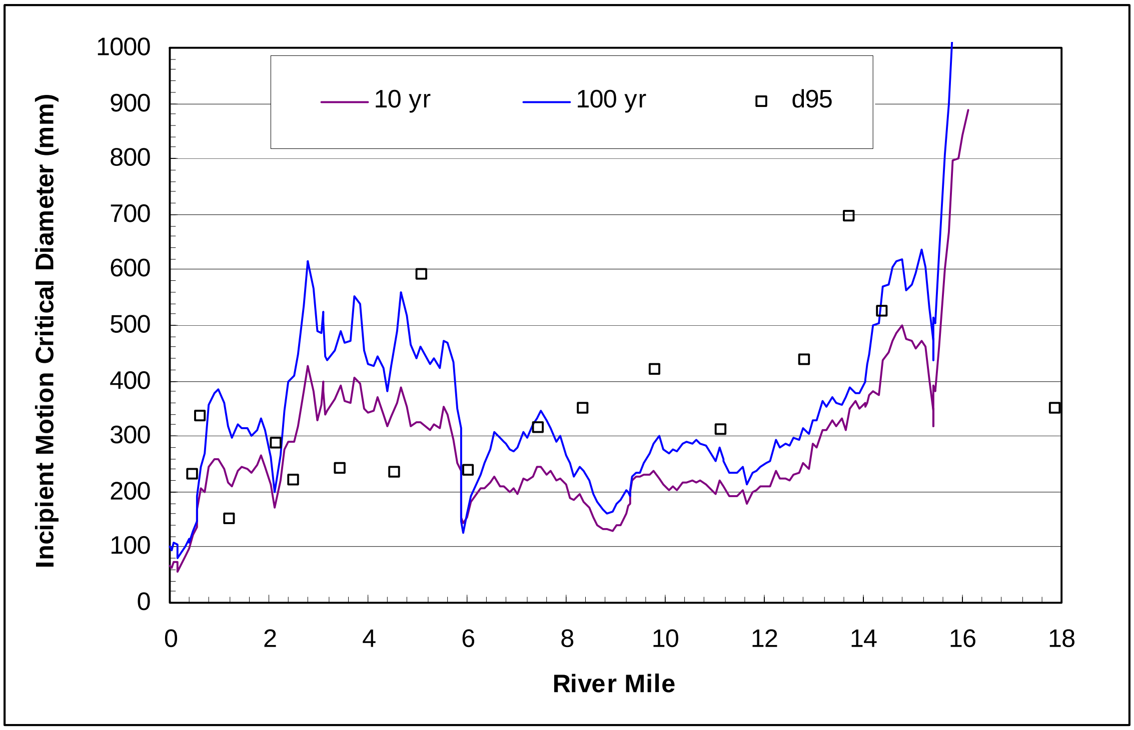


Figure 5.26. Incipient Motion Critical Diameter for 10- and 100-yr Floods, Plotted with d_{95}.

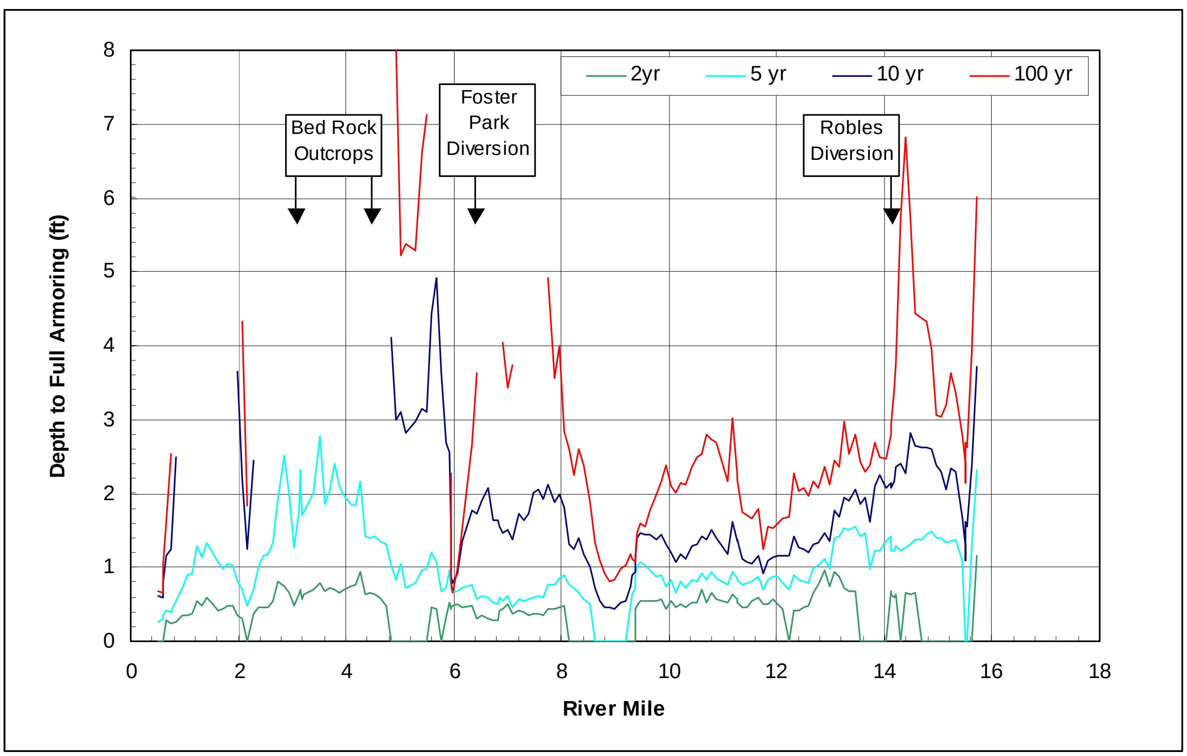


Figure 5.27. Estimated Depth to Full Armoring.

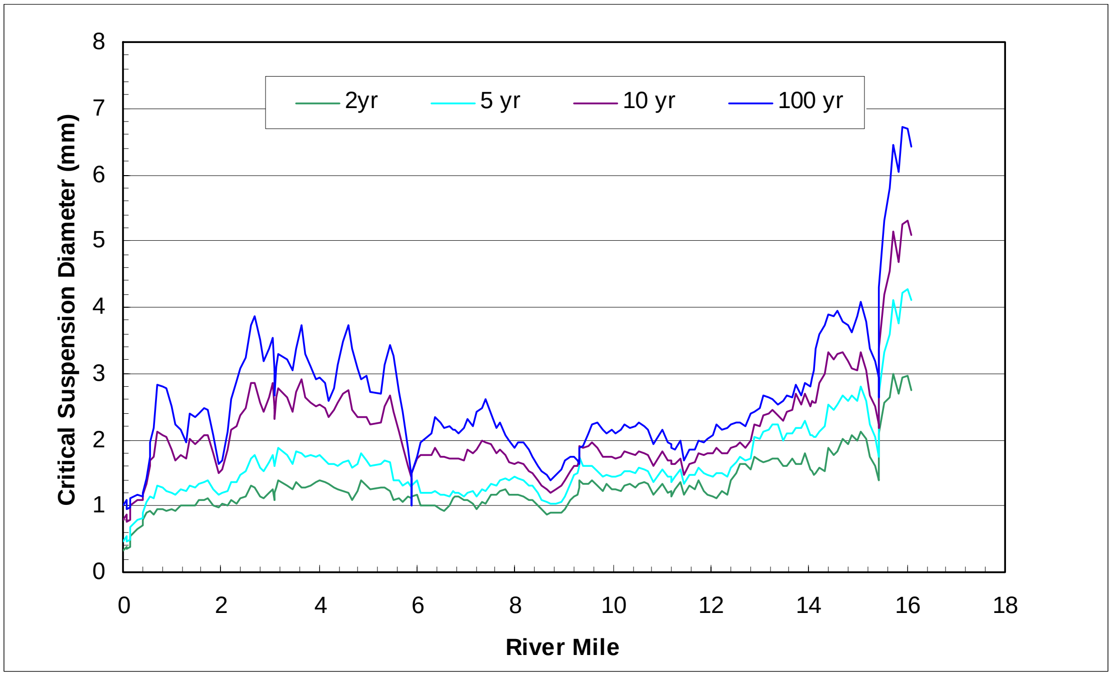


Figure 5.28. Critical suspended diameter along Ventura River for selected floods.

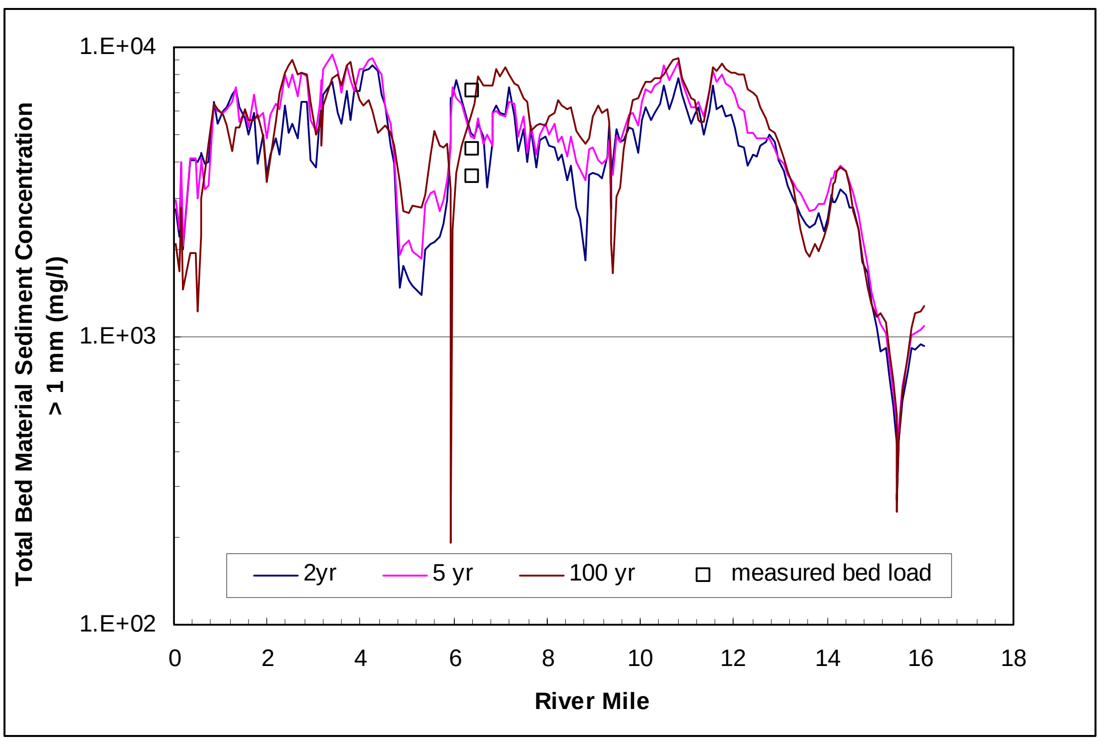


Figure 5.29. Bed material sediment capacity concentration of sediment sizes greater than 1 mm, for the Ventura River using Meyer-Peter-Müller sediment transport equation.

5.7. River Morphology

5.7.1. SUMMARY OF CURRENT VENTURA RIVER GEOMORPHOLOGY

Table 1.3 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. These criteria 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 5.18), 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 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 5.18. Geomorphic Descriptions of Reaches of Matilija Creek and Ventura River. The reach numbers correspond to those found in Table 1.3 and Figure 1.3.

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

Reach #	Land Marks	River Miles	General Geomorphic Characteristics
			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. 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 the channel bed 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.

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 5.30). The figure shows that the river is relatively dynamic during a flood and the riverbed elevations can change several feet during a single event.

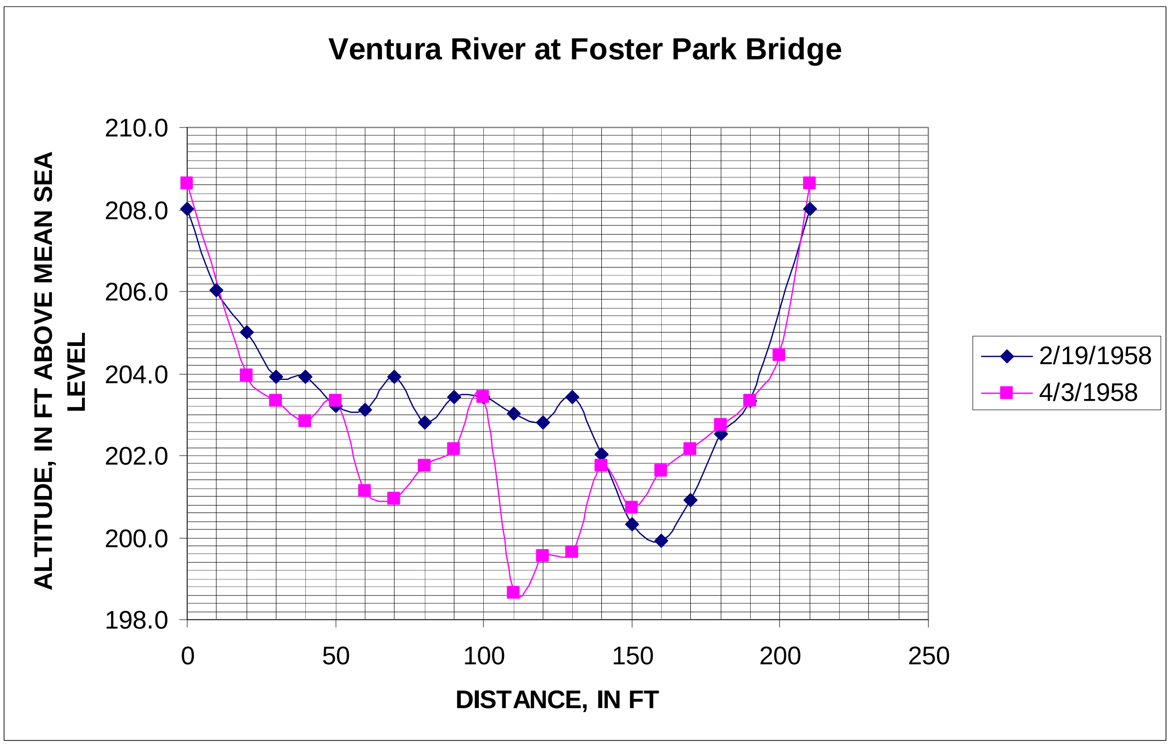


Figure 5.30. Change of Cross Section at Foster Park Bridge due to 1958 flood.

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 5.31). 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.

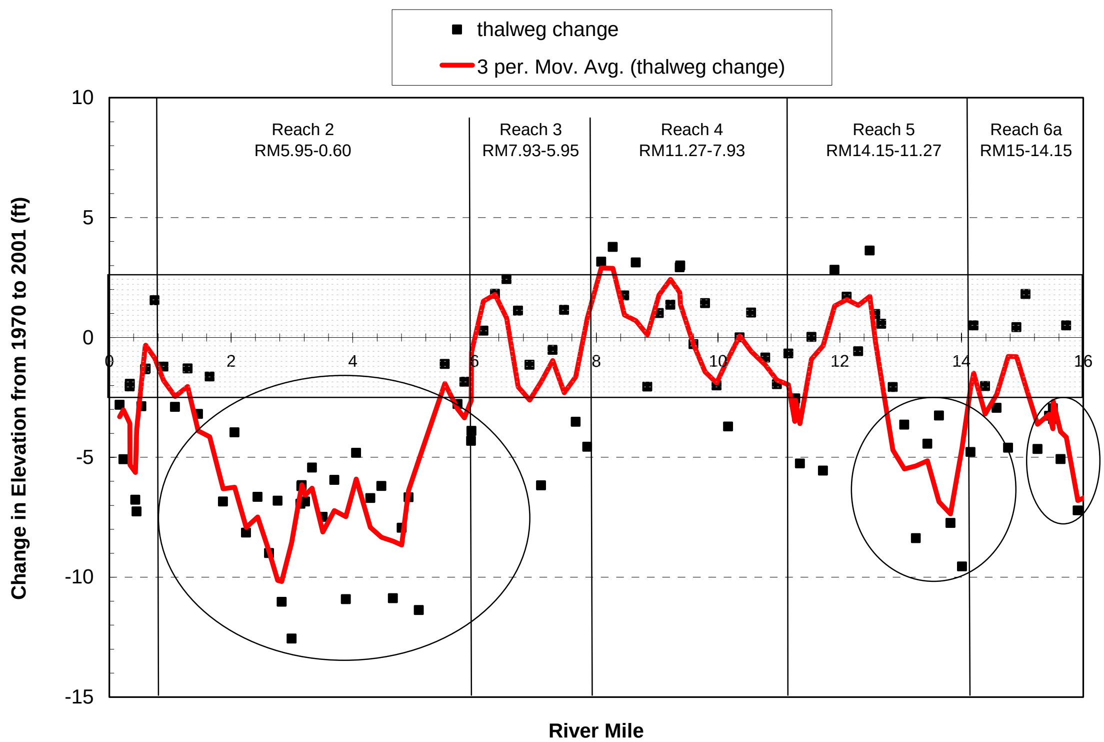


Figure 5.31. Comparison of change in thalweg elevation between 2001 and 1970. Negative changes indicate areas of degradation in the channel bed. Positive changes indicate areas that have aggraded. Areas within 2.5 feet of change are considered to be within the error range of the 1970 data.

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 5.32).

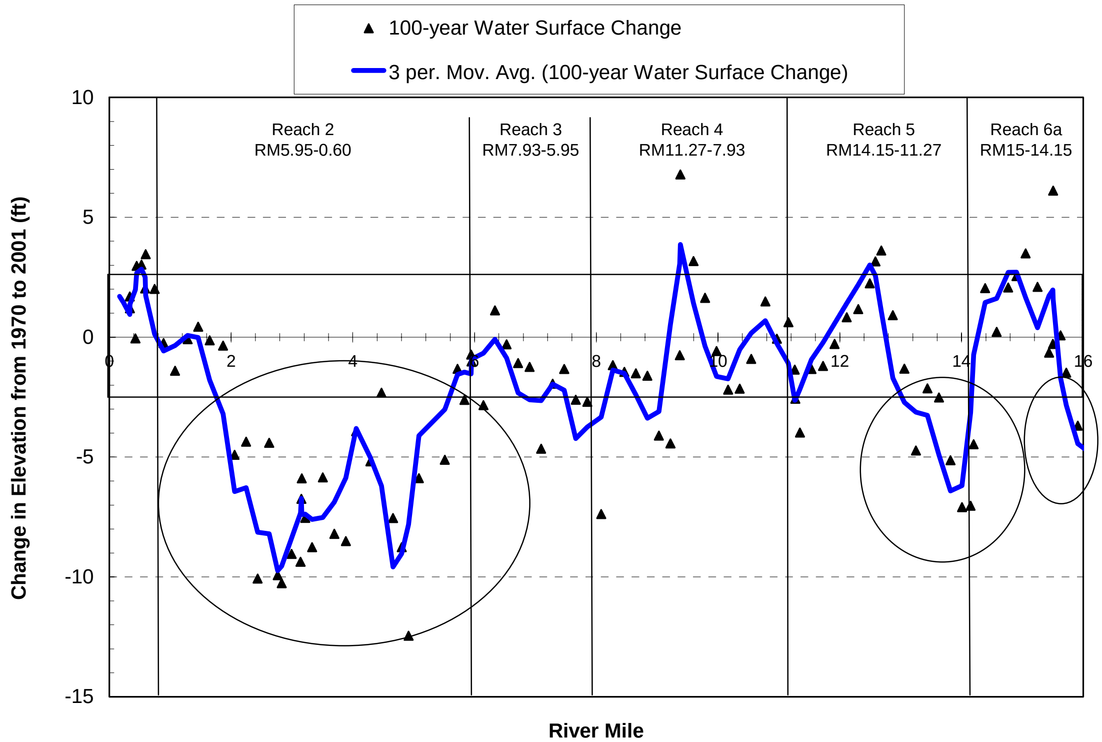


Figure 5.32. Comparison of change in 100-year flood stage between 2001 and 1970. Negative changes indicate areas where the flood stage has lowered. Positive changes indicate areas where the flood stage has increased. Areas within 2.5 feet of change are considered to be within the error range of the 1970 data.

Plots of the cross sections in 1970 and 2001 are given in Figure 5.35. 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.

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 5.36). 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 22.1). 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, ortho-rectified, 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, California V, 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 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 5.33. 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 is shown in Figure 5.34. 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.

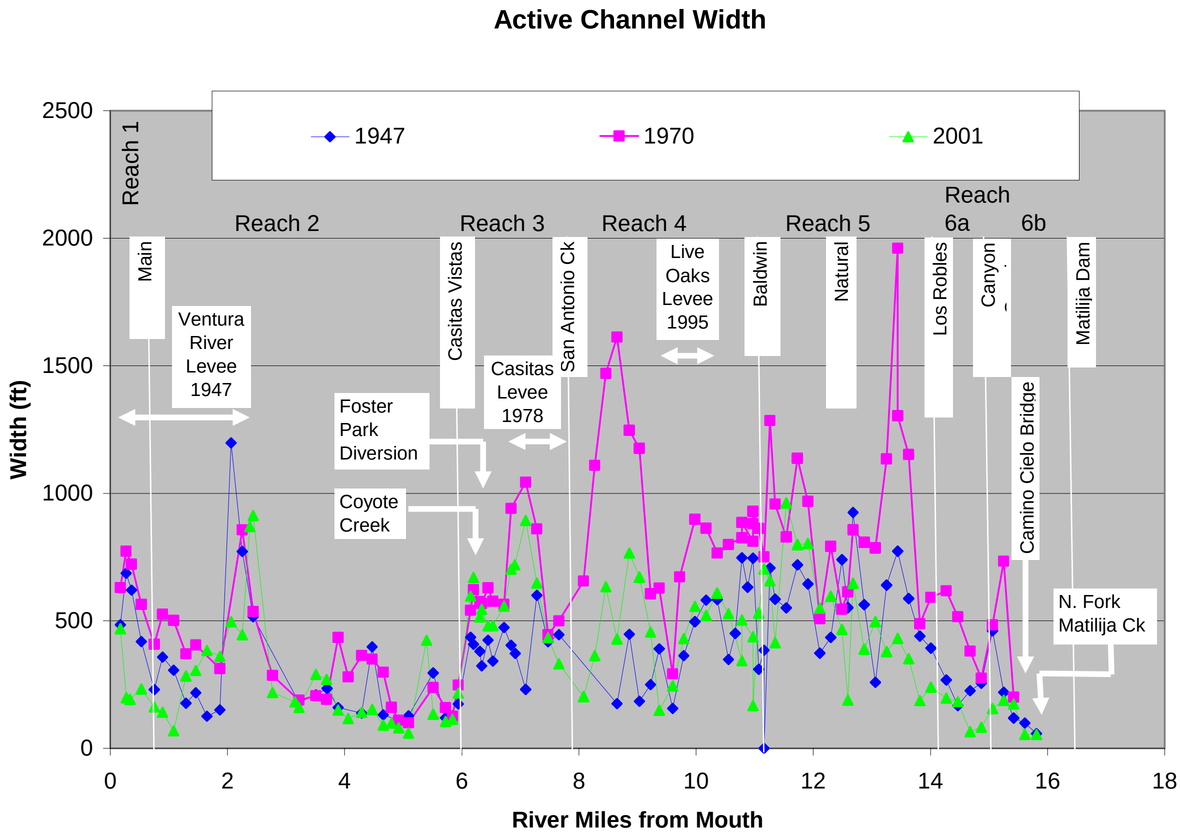
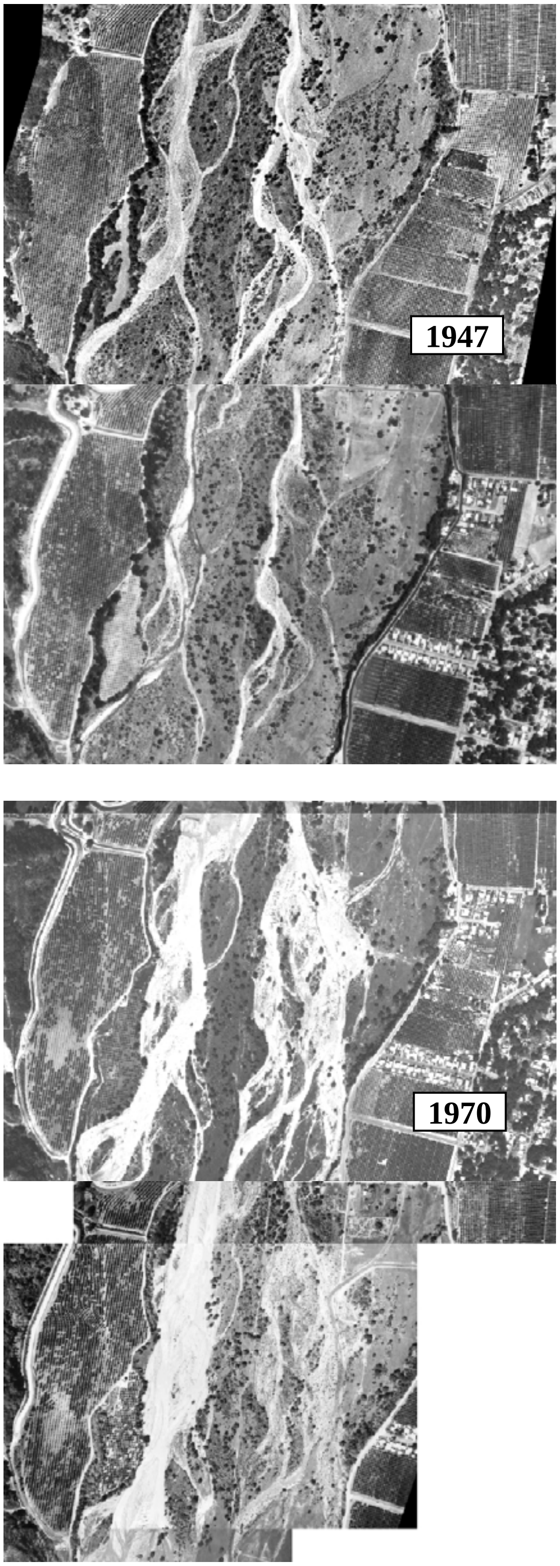


Figure 5.33. Historical Active Channel Widths of the Ventura River in 1947, 1970, and 2001.



1965

1978

Figure 5.34. 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 notion 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 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 5.19). Of these eight floods following the closure of Matilija Dam, only the flood of 1952 occurred prior to the floods of 1969.

Table 5.19. Ten Largest Floods at USGS Gage 11118500 since 1927.

Rank	Date	Flow at Foster Park (cfs)
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

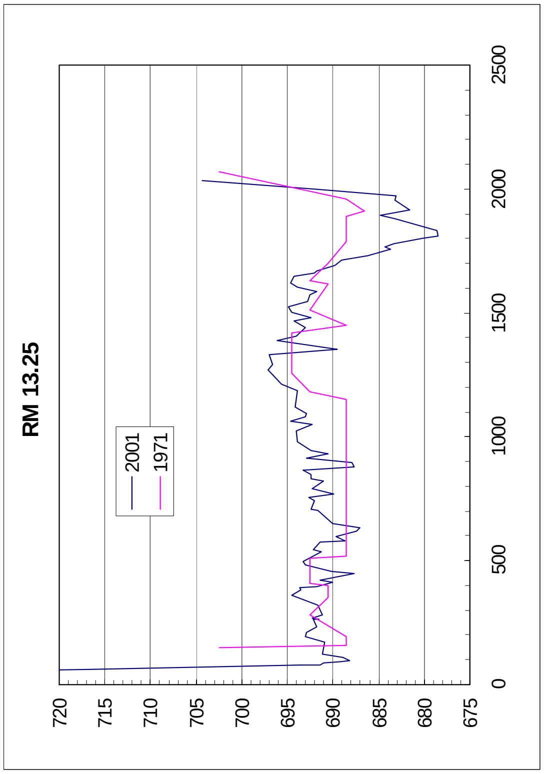
While the geologic setting primarily controls the current morphology of the river, the current climate conditions (during the last 35 years) and the associated hydrology strongly influence the movement of sediment within the river system, and thus the channel form. Based on the climate regime and the 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.

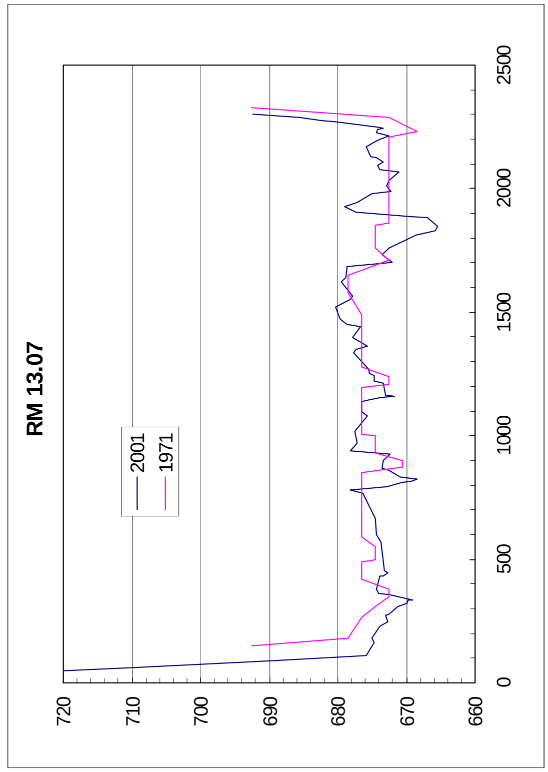
As mentioned previously, there has been degradation documented in the Ventura River since 1971 in three reaches. Causes for the degradation could be:

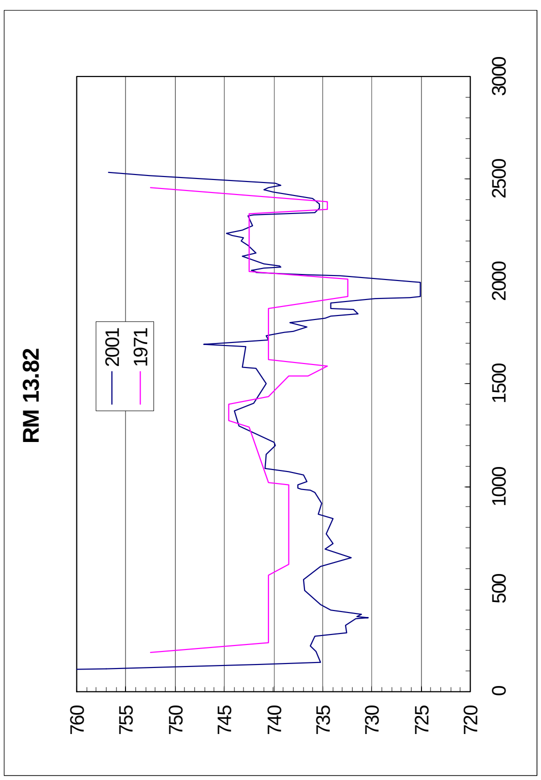
1. A shift from a relatively dry period to a wet period.
2. Trapping of sediment behind Matilija Dam and associated downstream degradation.
3. Trapping and removal of sediment at Robles Diversion.
4. Trapping of sediment and water behind Casitas Dam.

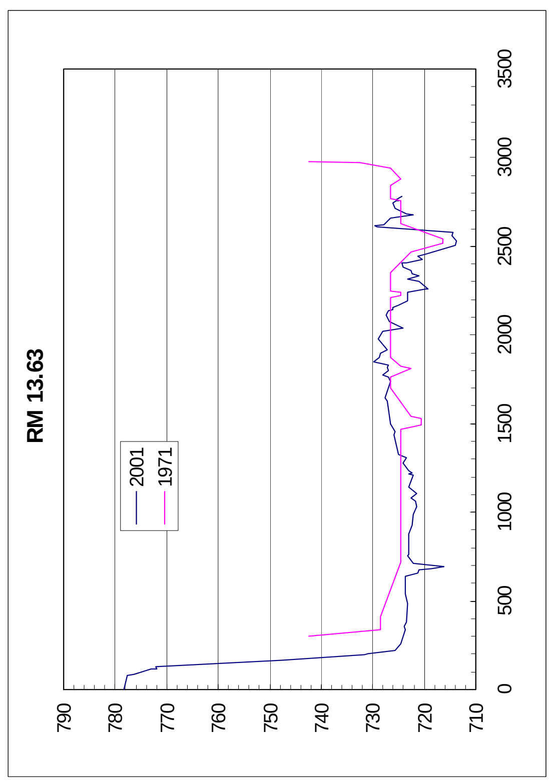
The hydrological record and the large sediment supply in the Ventura River floodplain supports reason 1. Because the coarse sediment sizes are transport limited, increasing the volume of water will cause degradation. The degradation of the Ventura River may be the result of the rivers increased ability 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.


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


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


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


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

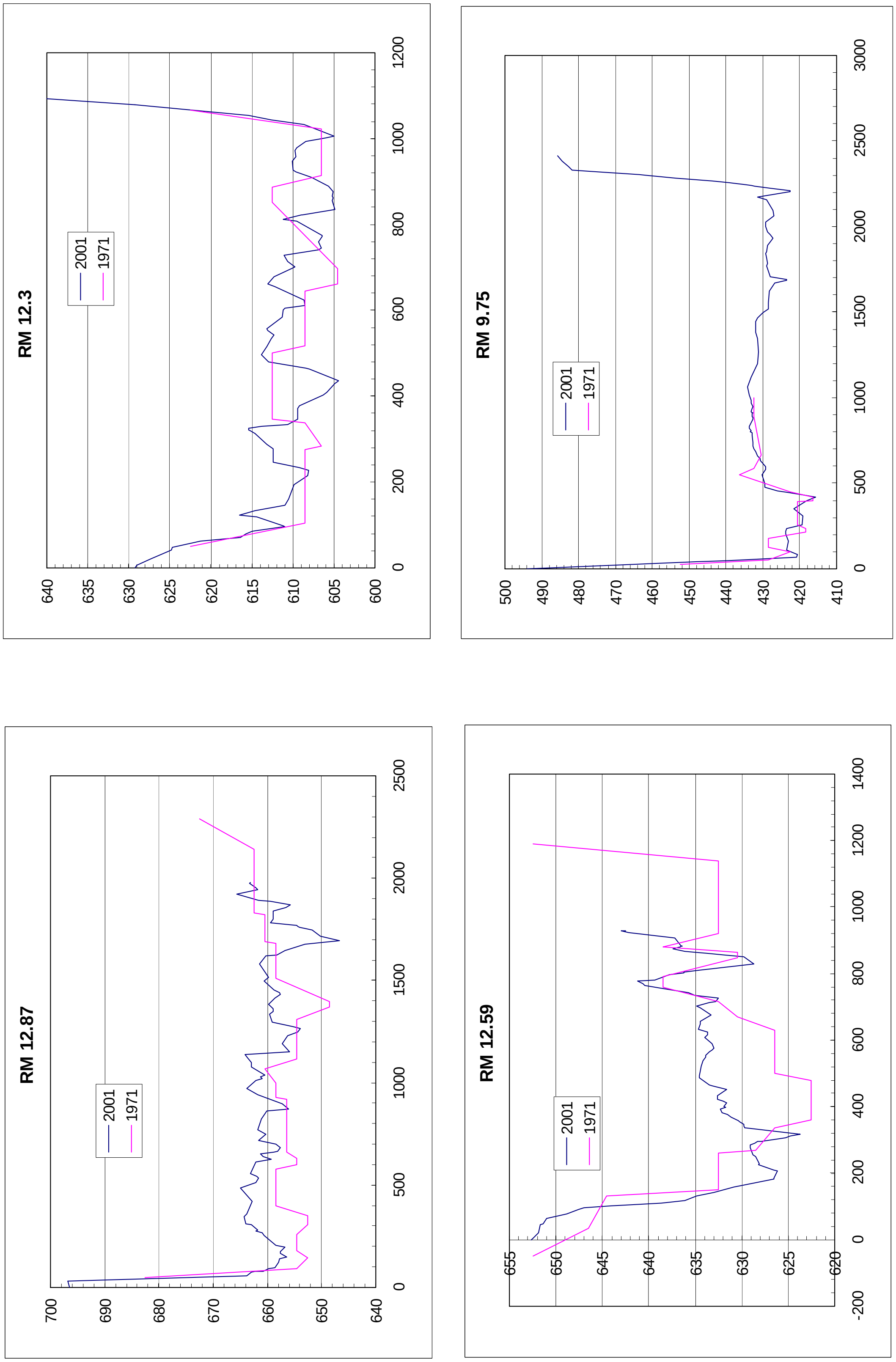


Figure 5.35. Cross section comparison between 1971 and 2001 surveys.

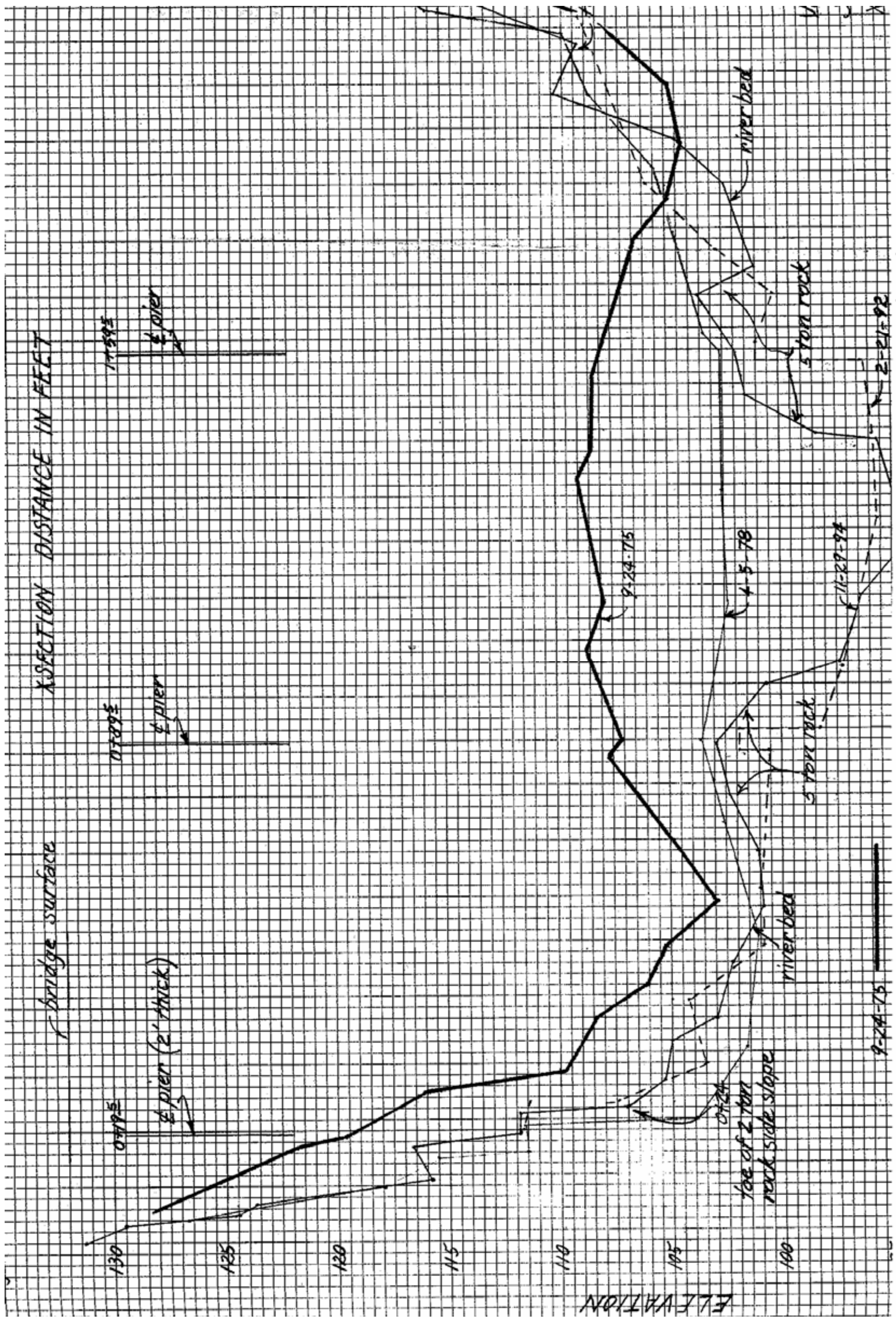


Figure 5.36. Bed elevation changes all Shell Road Bridge (from Ventura County Records of William Carey).

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

The 1947 Matilija Creek channel is shown in Figure 5.37, 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.

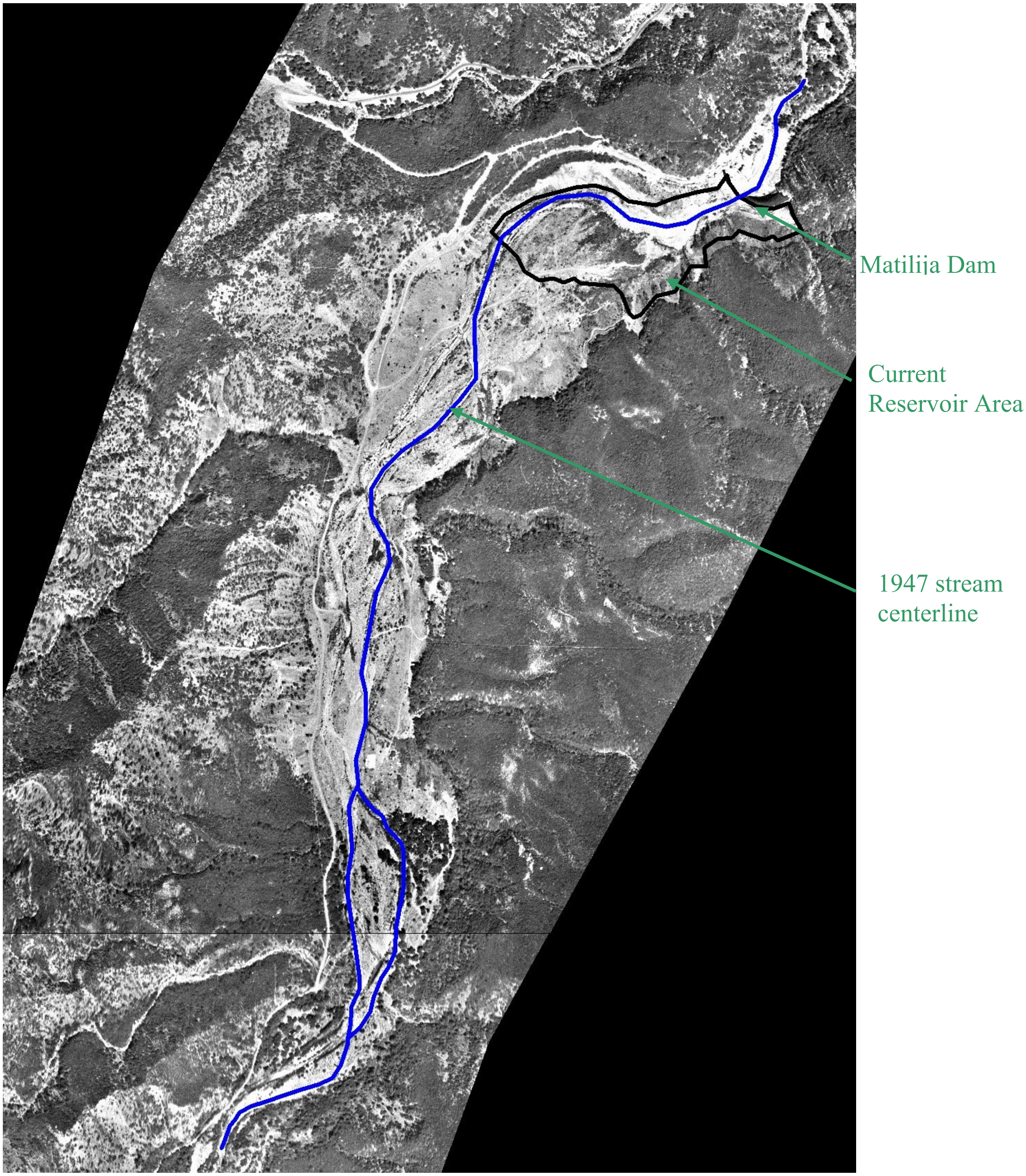


Figure 5.37. Aerial Photograph Taken in 1947 of Matilija Creek Upstream of Matilija Dam.

5.8. Historical Coastline Changes at Mouth of Ventura River

The 1947, 1970, 1978, 1992, 2001, and 2005 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 5.38. The 1970 and 2005 coastlines 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. The photo taken in 2005 immediately after a large flood.

The tidal variation will also affect the location of the coastline. The elevation of the Pacific Ocean can vary between approximately 7 feet based upon the tidal data near Ventura. It is uncertain whether the photos were taken at high tide, low tide or somewhere between. 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.

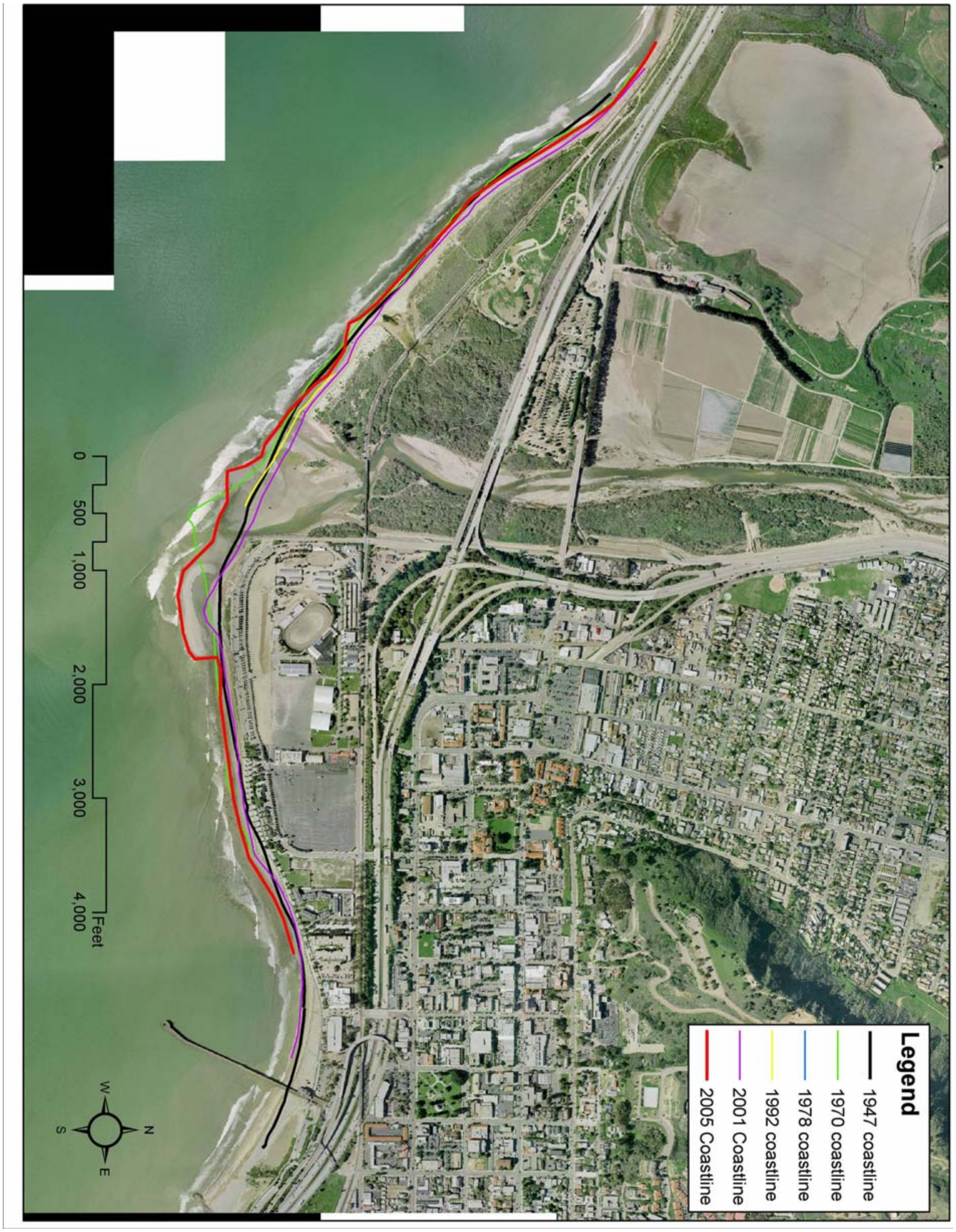


Figure 5.38. Aerial Photograph of Coastline at Mouth of Ventura River.

6. Sediment Transport Modeling

The GSTAR-1D (Generalized Sediment Transport model for Alluvial Rivers – One Dimension) model was used to model the sediment transport in the Ventura River (Huang and Greimann, 2007). 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.

GSTAR-1D is used to estimate the With- and Without-Project Future Conditions. The results from the model are used in Sections 10 "Future Conditions Hydraulics" and Section 11 "Future Conditions Channel Morphology, Sediment Transport, and Reservoir Sedimentation."

6.1. Hydrologic

Several different hydrological inputs were used to estimate future conditions. Single event hydrographs were used to estimate short-term impacts for various sized storms. Historical reconstructed hydrographs were used to evaluate potential long-term trends under conditions similar to historical conditions. Synthetic hydrographs were used to evaluate other potential hydrologic scenarios.

6.1.1. Design Storms

A flood-frequency analysis was performed for the entire length of the Ventura River in 2002 (Section 0). Frequency discharges for the 2-, 5-, 10-, 20-, 50-, 100-, and 500-year events were developed. Two stream gage records were used in the initial analysis: Matilija Creek at Matilija Hot Springs (USGS gage 11115500), and Ventura River near Ventura (USGS gage 11118500). The standard Log-Pearson III approach to flood frequency analysis was rejected for this location as detailed in Bullard (2002). A regression equation approach was implemented with the results detailed in Table 2.2.

In Table 2.2, gage 11115500 corresponds to the Upstream of Confluence with N. Fork Matilija Creek location and gage 11118500 corresponds to the Casitas Road Bridge location. The peaks are therefore explicitly identified for probabilistic floods at these two gage sites from previous analysis. To stay consistent with the previous analysis and to specify the probabilistic floods at gages 11116000 and 11117500 an implicit approach was taken to identify the relationship among the floods at gages 1111600, and 11117500. Figure 6.1 shows a linear regression analysis for the peak discharges among the model storm set between gages 11116000 and 11117500. The variance explained by the linear models is approximately 70%. The peaks discharges at gages 11116000 and 11117500 were specified at each return period from two equations: mass balance and the relationship specified in Figure 6.1. The estimated peak discharges for all gages are given in Table 6.1.

Table 6.1. Probabilistic peak discharge estimates for specific gage locations.

Return Period	Gage #11115500	Gage #11116000	Gage #11117500	Gage #11118500
2	3,060	358	1,102	4,520
5	7,090	1,095	2,874	11,060
10	12,500	6,844	16,695	36,040
20	15,200	9,094	22,105	46,400
50	18,800	11,944	28,955	59,700
100	21,600	14,059	34,040	69,700
500	27,900	19,083	46,116	93,100

For gages 11115500, 11116000, and 11117500 the design storm of 2/7/1992 was scaled such that the scaled peak matched the flood frequency analysis predicted peak at return periods 2-, 5-, 10-, 20-, 50-, 100-, and 500-years. Scaling was performed using the software DSO-EXTRAP_v2, produced by the Flood Hydrology Group, Technical Service Center, Bureau of Reclamation (Swain et al., 2004) The software scales the hydrograph in log space to meet the peak discharge requirement. Gage 11118500 was then calculated based on mass balance that the sum of the contributing gages is equal to the flow at 11118500.

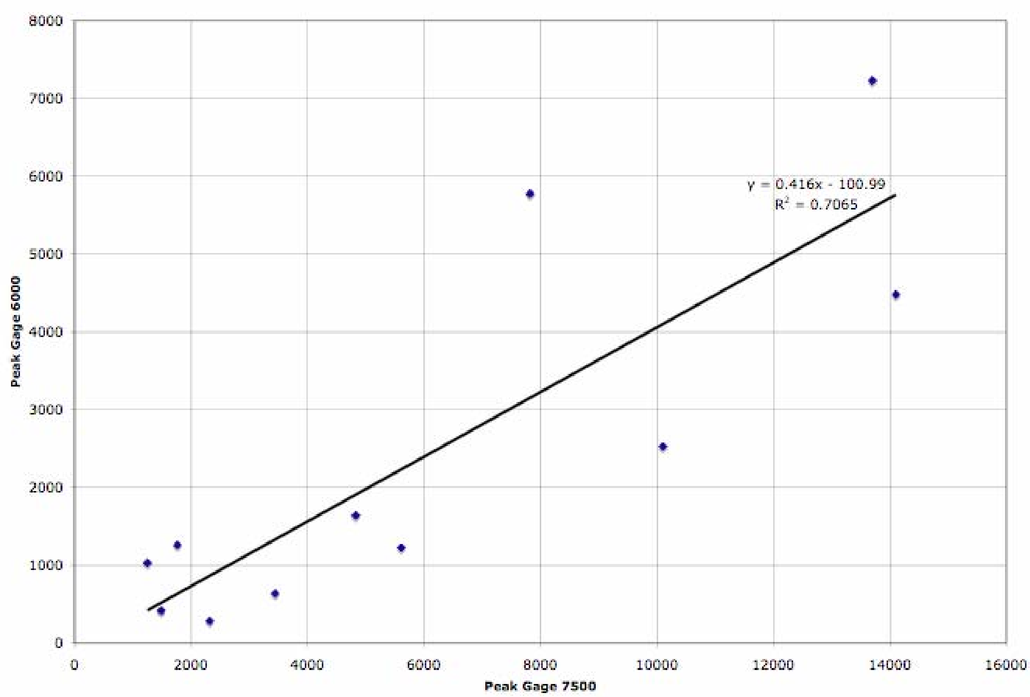


Figure 6.1. Peak Discharge Relationship between USGS Gage 11116000 and 11117500.

6.1.2. RECONSTRUCTED HYDROGRAPHS

The reconstruction of peak flow hydrographs for the period of record is necessary because the peak discharge, an instantaneous value, differs from the average daily flow for the same time extent. To model a hydrograph for durations less than one-day such that the peak is accurately represented, the daily discharges must be temporally disaggregated into 15-minute intervals. The process used within this analysis uses a single historical event for each reconstruction location to disaggregate partial peak information over the period of record. The historical events used for the disaggregating are given in Table 6.2.

Table 6.2. Design storm dates used for disaggregating daily averaged annual peak values.

Gage ID	**Gage Name**	**Storm Date**
11118500	Ventura River near Foster Park	1/13/93
11117500	San Antonio Creek at Casitas Springs	1/13/93 – 2/5/93
11116000	North Fork Matilija Creek @ Matilija Hot Springs	1/14/93 – 2/5/93
11115500	Matilija Creek @ Matilija Hot Springs	1/19/95 – 1/21/95

Each date within the daily average discharge record corresponding to a partial peak within that record was disaggregated into fifteen-minute time steps. The disaggregating routine was performed by inserting a single day from the design storm (15-minute data) into the partial peak record. The design storm, defined as the largest 24-hour volume from the storms identified in Table 6.1, was then scaled in arithmetic space such that the scaled peak is equal to the peak specified within the partial peak record for that day. The volume is then adjusted for the scaled hydrograph such that the volume matches the original specified in the daily average record. The volume adjustment routine is performed by leaving the 1.5-hour interval directly around the peak unaltered and adjusting the remaining values such that the desired volume is achieved. The design storm does not change from event to event, while the scaled design storm does.

Three different 50-year hydrographs were constructed using this data. The first hydrograph was begun with Water Year (WY) 1950 and extended until WY 2000. The second hydrograph began in WY 1969, extended until WY 2001, and then recommenced with WY 1950 and ends with WY 1967. The last hydrograph extended from 1991 to 2001, then from 1950 to 1989. These three hydrographs are used in the predictive simulations. WY 1950 corresponds to a dry period. WY 1969 is the wettest on record and WY 1991 corresponds to an average WY (see Figure 2.8).

6.1.3. 50-YEAR SIMULATED FLOW SERIES FOR FUTURE SCENARIOS

A stochastic approach was taken to simulate forty sets of fifty-year hydrographs for the gages 11115500, 11116000, 11117500, and 11118500. The decision was made to ignore any mean trends and concentrate the stochastic models only on error variances, cross-correlation and autoregressive traits. This means that the future scenarios will reflect the past in a manner that assumes stationarity in time. The first test performed was to assess the degree to which the gages are cross-correlated. For all overlapping years among all gage combinations the correlation coefficient

$$r = \frac{\sum_{i=1}^{N}\left[\left(x_i - \bar{x}\right)\left(y_i - \bar{y}\right)\right]}{\sqrt{\sum_{i=1}^{N}\left(x_i - \bar{x}\right)^2}\sqrt{\sum_{i=1}^{N}\left(y_i - \bar{y}\right)^2}} \qquad \text{Eq 6.1}$$

was calculated (Table 6.3) for the annual peak discharge. As can be seen, the cross-correlation coefficients for annual daily maximums are all greater than about 0.85. The correlation coefficient describes the degree to which information at one site coincides with information at another site. For this case, the cross-correlation coefficients indicate that the portion of the Ventura basin being measured by the four gages shown in Table 6.3 coincide with each other in terms of relative magnitude.

Table 6.3. Cross-correlation, *r*, structure for peak annual discharges.

	Gage 1115500	Gage 1116000	Gage 1117500	Gage 1118500
Gage 1115500	-	0.87	0.88	0.93
Gage 1116000	0.87	-	0.97	0.86
Gage 1117500	0.88	0.97	-	0.87
Gage 1118500	0.93	0.86	0.87	-

Each gage was further examined for the autoregressive properties of their peak annual discharges (Figure 6.2 through Figure 6.5). The only autoregression coefficient that is statistically significant is a three-year lag on gage 1115500. This feature does not appear on any other gage and only explains approximately 6.25 percent of the variance of the information. Therefore, no autoregressive traits were used within the model. Only randomness and the cross-correlation between gage 1115500 and 1118500 were included. Random numbers, R_i, were generated from a uniform distribution representing the probabilistic peak annual discharge at gage 1118500. The peak discharge was related to this random number through the following equations:

- $R_i < 0.1$

- $P_i = 14494 Ln\left(\frac{1}{R_i}\right) + 2997.4$ Eq 6.2

- $R_i > 0.1$
  - $T_i = \frac{-1}{Ln(1-R_i)}$ Eq 6.3
  - $P_i = -109.67T_i^2 + 3080.4T_i - 2600$ Eq 6.4

A random number for the probabilistic peak annual discharge at gage 1115500 was then correlated to the random number generated for gage 1118500 by $R_{i5500} = R_i + 0.0748 * N(-0.5,0.5)$. The peak discharge for gage 1115500 in year I was then determined by the following equations:

- $R_{i5500} < 0.1$
  - $P_{i5500} = 3937.3Ln\left(\frac{1}{R_{i5500}}\right) + 3432.5$ Eq 6.5
- $R_{i5500} > 0.1$
  - $T_{i5500} = \frac{-1}{Ln(1-R_{i5500})}$ Eq 6.6
  - $P_i = -46.125T_{i5500}^{\ 2} + 1921.2T_{i5500} - 1601.5$ Eq 6.7

Further, yearly maximum floods were limited to the 500-year return period flood, 93,100 ft^3/s for gage 1118500 and 27,901 ft^3/s for gage 1115500. Thus, the flood frequencies for gages 1118500 and 1115500 are represented in Figure 6.6 and Figure 6.7.

The annual maximum discharges were disaggregated temporally by assigning the historical year that most closely had a peak corresponding to the simulated peak at gage 11118500. This year was then assigned to the corresponding simulation peak and scaled arithmetically such that the annual peak value matched the simulated peak value. The gage 11118500 values were then disaggregated to 1115500, 11116000, and 11117500. All tributaries not explicitly gaged were lumped into gages 11116000 and 11117500. For each year simulated, a peak flow value at gage 11115500 was explicitly defined during the peak flow simulation step discussed previously. The temporally disaggregated gage 11115500 was then scaled year-by-year such that the peak flows at 11115500 were preserved. The remaining discharge was then allocated to gages 11116000 and 11117500 as a function of their relative drainage areas. The drainage areas used were 15.6 mi^2 for gage 11116000 and 51.2 mi^2 for gage 11117500.

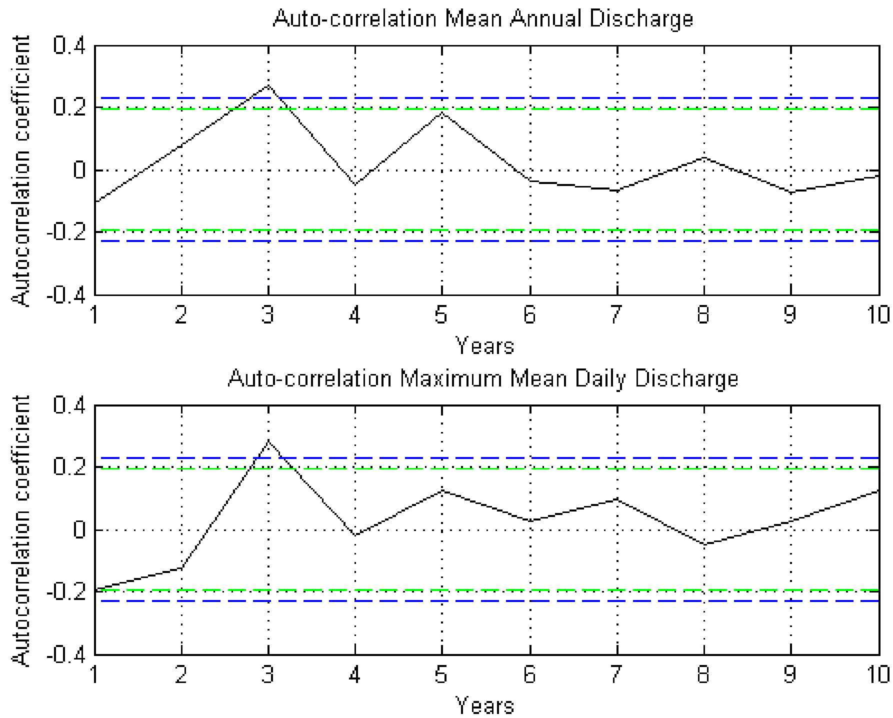


Figure 6.2. Gage 11115500 autocorellation structure.

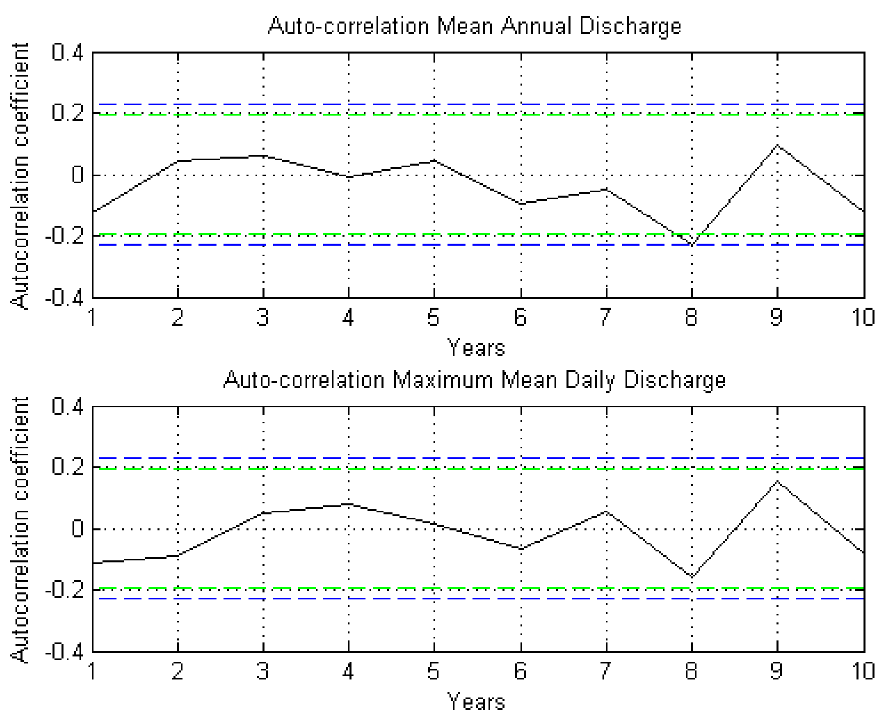


Figure 6.3. Gage 11116000 autocorrelation structure.

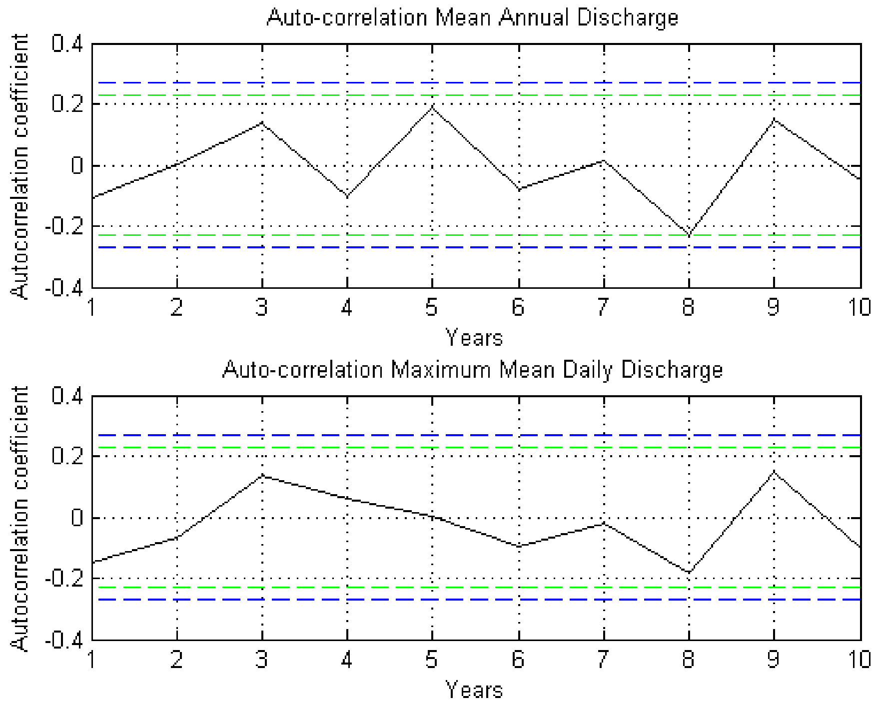


Figure 6.4. Gage 11117500 autocorrelation structure.

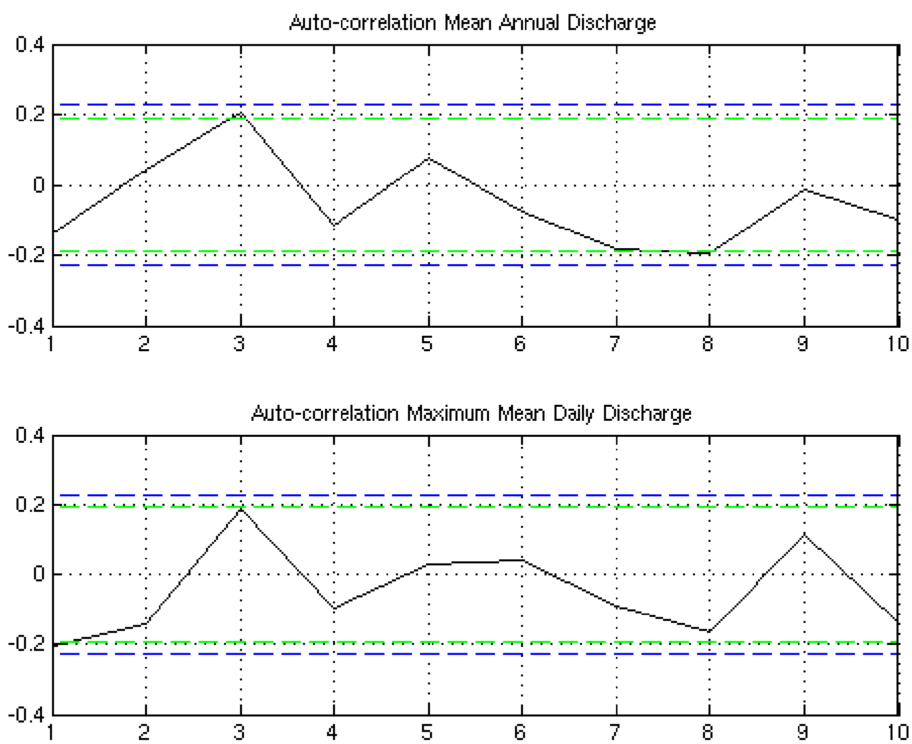


Figure 6.5. Gage 11118500 autocorrelation structure.

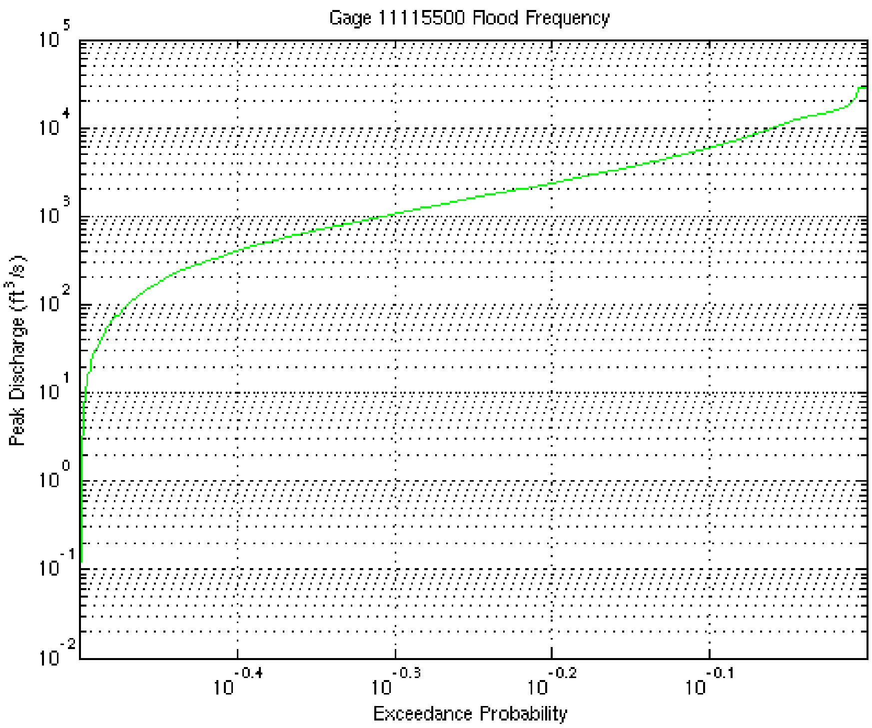


Figure 6.6. Flood Frequency curve for gage 11115500 used in stochastic simulations.

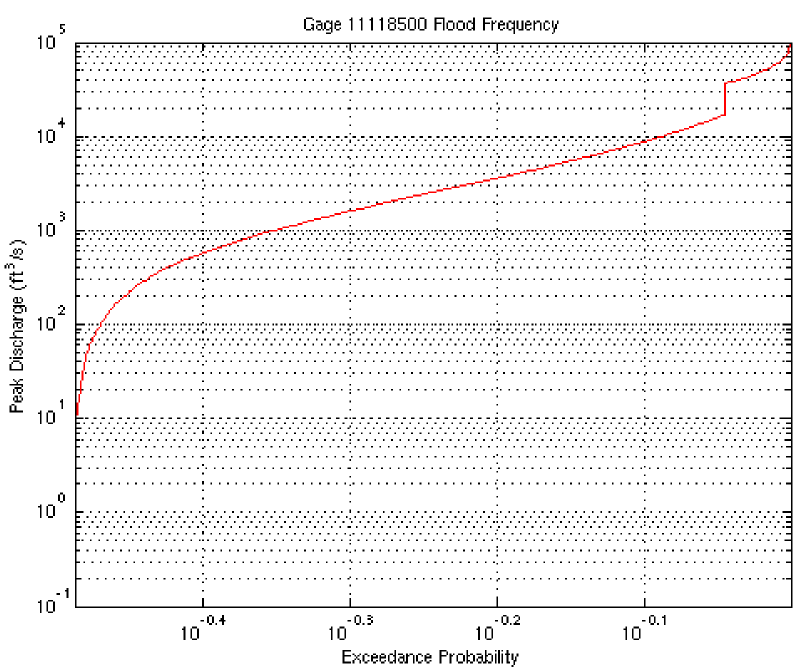


Figure 6.7. Flood Frequency curve for gage 11118500 used in stochastic simulations.

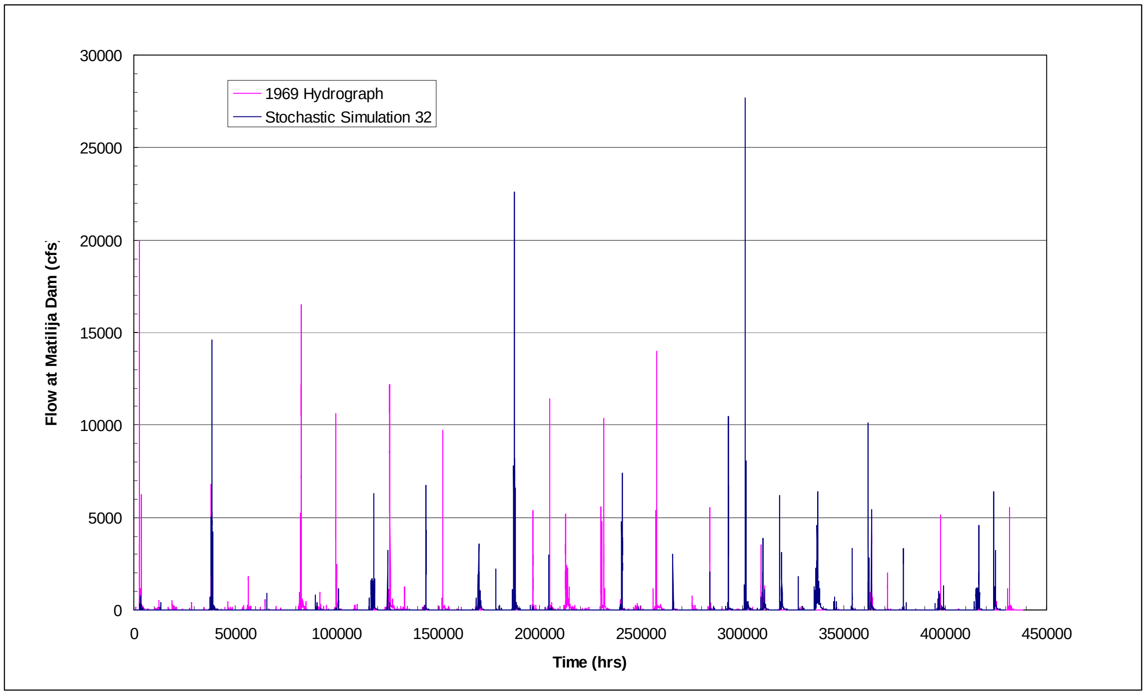


Figure 6.8. Sample Hydrograph Traces as Compared to 1969 Hydrology.

6.2. 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 the same as used in the HEC-RAS model described in Section 4.2. The GSTAR-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.

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. Camino Cielo will be replaced with a bridge that can pass much larger flows beneath it. It is assumed that the replacement bridge will not reduce the sediment transport capacity relative to the natural stream channel at that location. Santa Ana Bridge currently passes the 100-yr flood beneath its bridge deck and it is assumed that it will not affect sediment transport computations.

6.3. 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.

6.3.1. 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 (6.3.4). 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 6.5). 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 (#11118500). The trap efficiency of the silt was adjusted based on the volume of deposition in Matilija Reservoir and using the Brune Curve (Table 6.4). 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 6.4 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 6.4. 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 6.5. 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

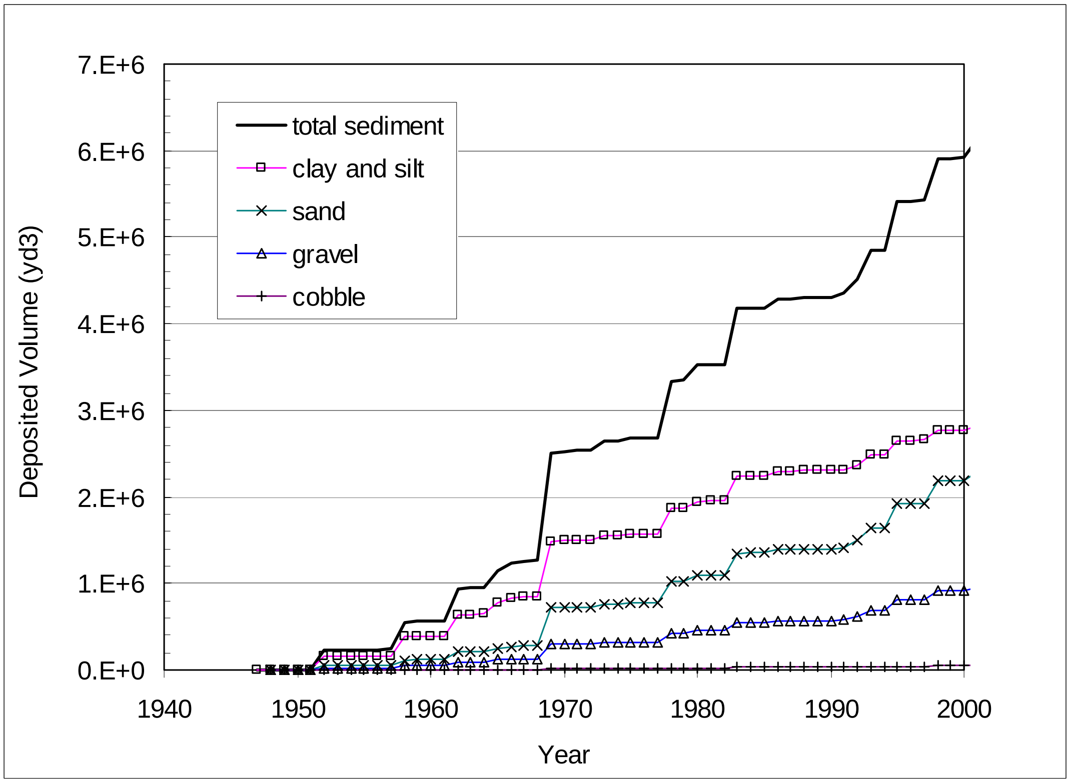


Figure 6.9. Simulated deposition in Matilija Reservoir using sediment rating curve for Matilija Creek.

6.3.2. TRIBUTARY INFLOW

It was also necessary to calculate the sediment load entering at North Fork Matilija Creek and at San Antonio Creek. The suspended sediment rating curves and computed bed load given in Section 5.5 titled "Sediment Loads and Sediment Yield from Watershed" are used in the GSTAR-1D model.

6.3.3. 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.

The active layer thickness needs to be set in the model and it represents the vertical distance over which bed material is assumed to be fully mixed. Its value was set to be approximately equal to the largest size of material in motion at high flows. Its value was also checked by analyzing model sensitivity to its value. The value of active layer thickness used in simulations was 2.9 feet.

6.3.4. 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, 2002) 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 combining the Wilcock and Crowe's model and Englund-Hansen into 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:

$$q_s = 0.05V^2 \left(\frac{d_{50}}{g(s-1)} \right)^{0.5} \left(\frac{\tau_b}{\gamma(s-1)d_{50}} \right)^{1.5} \qquad \text{Eq 6.8}$$

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:

$$\frac{q_{si}g(s-1)}{(\tau_b/\rho)^{1.5}} = f_i \max\left[14, \frac{0.05V^2}{g(s-1)d_i}\right] G(\phi_i) \qquad \text{Eq 6.9}$$

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 (2003) and is computed as:

$$G(\phi_i) = \begin{cases} \left(1 - 0.894/\phi_i^{\ 0.5}\right)^{4.5} & \phi_i \geq 1.35 \\ 0.000143\phi_i^{\ 7.5} & \phi_i < 1.35 \end{cases} \qquad \text{Eq 6.10}$$

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

$$\phi_i = \theta_i / (\theta_c \xi_i) \qquad \text{Eq 6.11}$$

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

$$\theta_i = \tau_b / (\gamma(s-1)d_i) \qquad \text{Eq 6.12}$$

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:

$$\xi_i = (d_i / d_{50})^{-\alpha} \qquad \text{Eq 6.13}$$

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

$$\alpha = 0.67[1 + \exp(1.5 - d_i / d_m)]^{-1} \qquad \text{Eq 6.14}$$

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 6.10 and Figure 6.11. 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 Toffaleti (1966) and Meyer-Peter and Müller (1948) formulas. The combination of Toffaleti 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:

$$\frac{q_{si}g(s-1)}{(\tau_b/\rho)^{1.5}} = 8\left[\left(\frac{K_s}{K_r}\right)^{1.5} - \frac{\theta_c}{\theta_i}\right]^{1.5} \qquad \text{Eq 6.15}$$

where, $\theta_c = 0.047$ Eq 6.16

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:

$$K_s = \frac{V}{C_m R^{2/3} S^{1/2}} \qquad \text{Eq 6.17}$$

and

$$K_r = \frac{26}{d_{90}^{1/6}} \qquad \text{Eq 6.18}$$

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 6.12. The function g^* represents the Right Hand Side of Eq. 6.10 and Eq. 6.15. 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.

The hydraulic properties used to compute the transport capacity is given in Table 6.6. The bed material is given in Section 21 Exhibit G. River Bed Material.

Table 6.6. Hydraulic properties used to compute sediment transport capacity at Foster Park.

Flow Rate	Area	Top Width	Velocity	Depth	Hydraulic Radius	Friction Slope
ft^3/s	ft^2	ft	ft/s	ft	ft	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

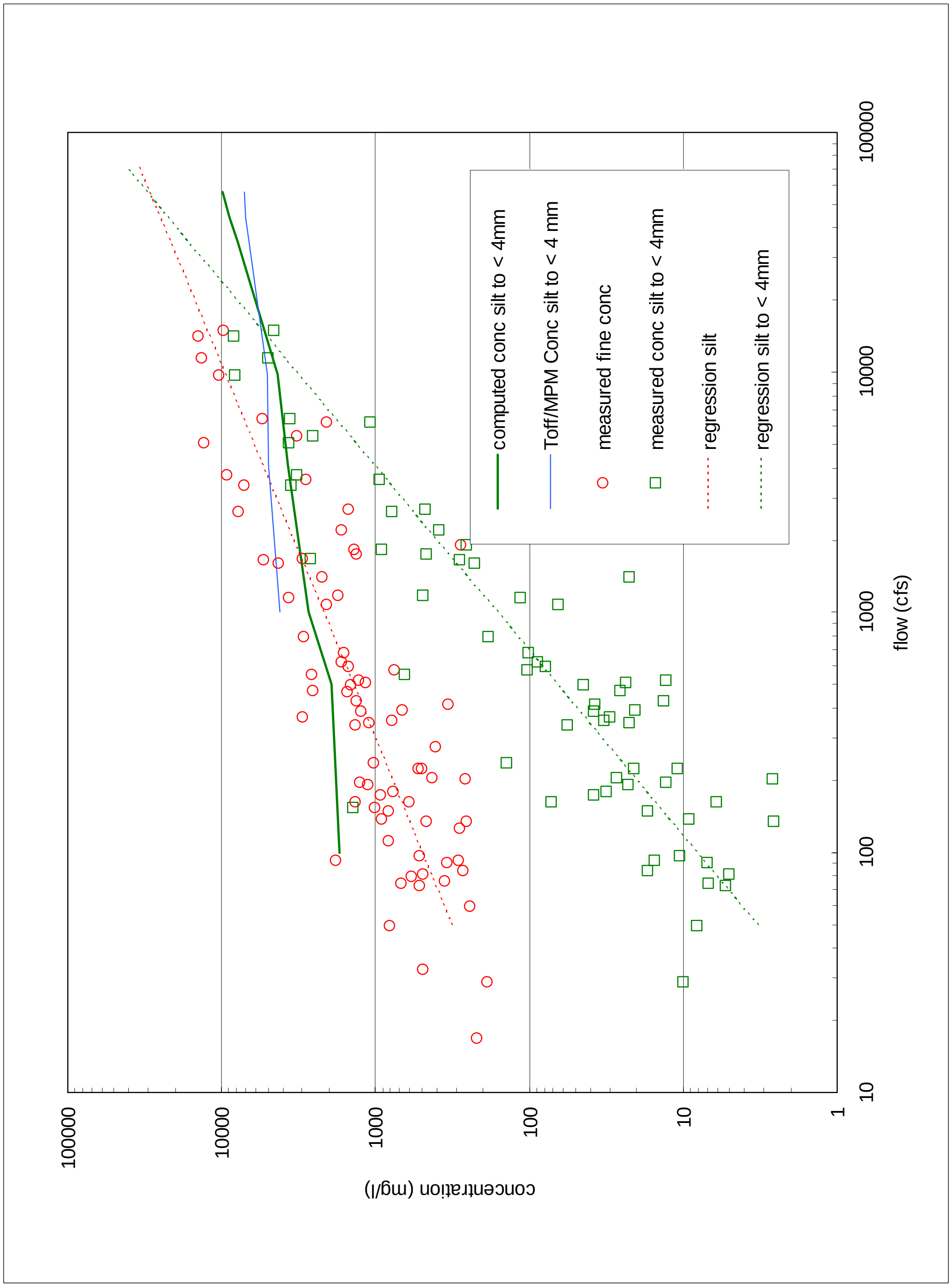


Figure 6.10. Computed Using Combined Transport Equation (6.9) and Measured Suspended Sediment Concentrations at Foster Park on the Ventura River.

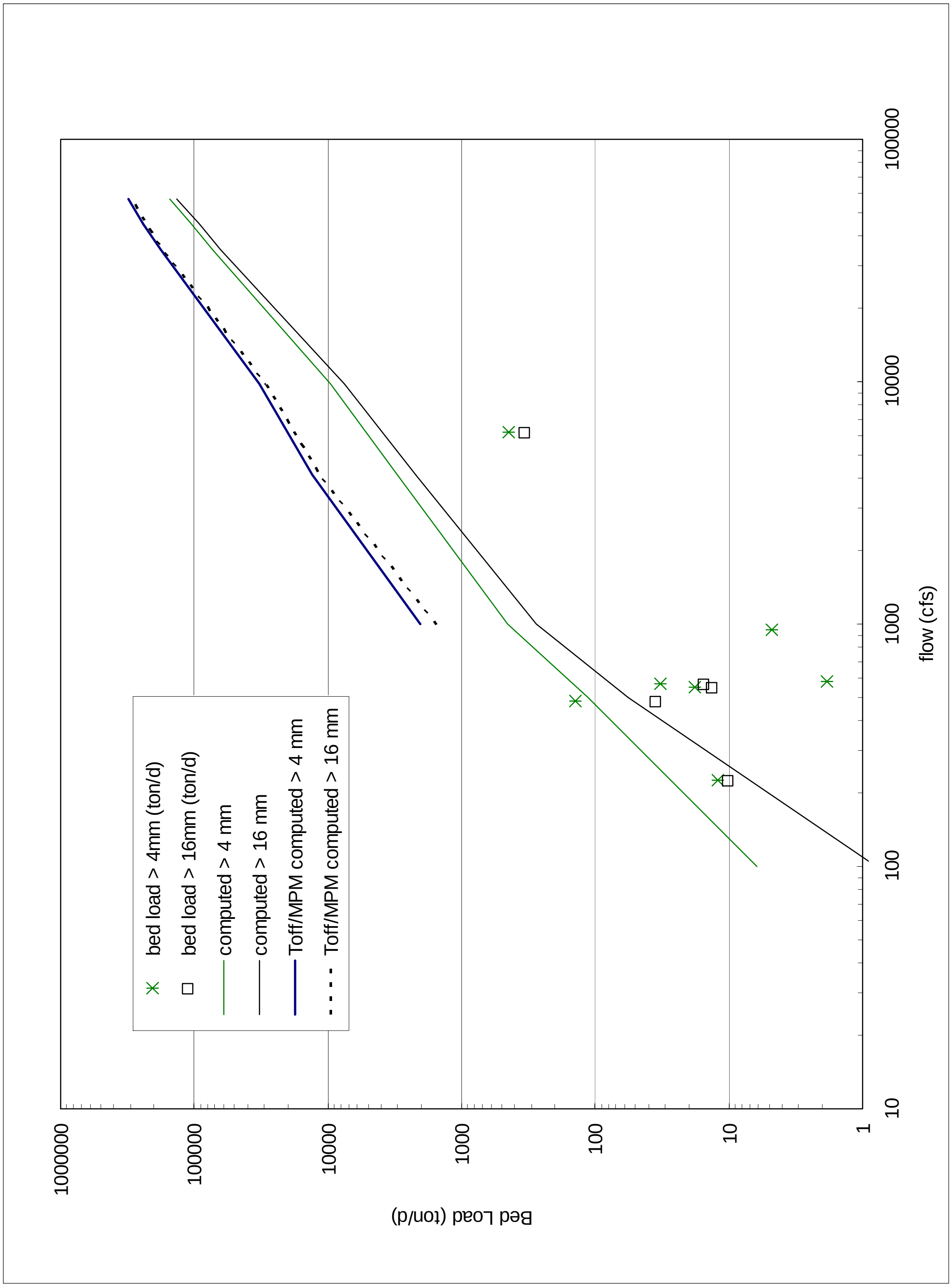


Figure 6.11. Computed Using Combined Transport Equation (6.9) and Measured Bed Load in Ventura River at Foster Park.

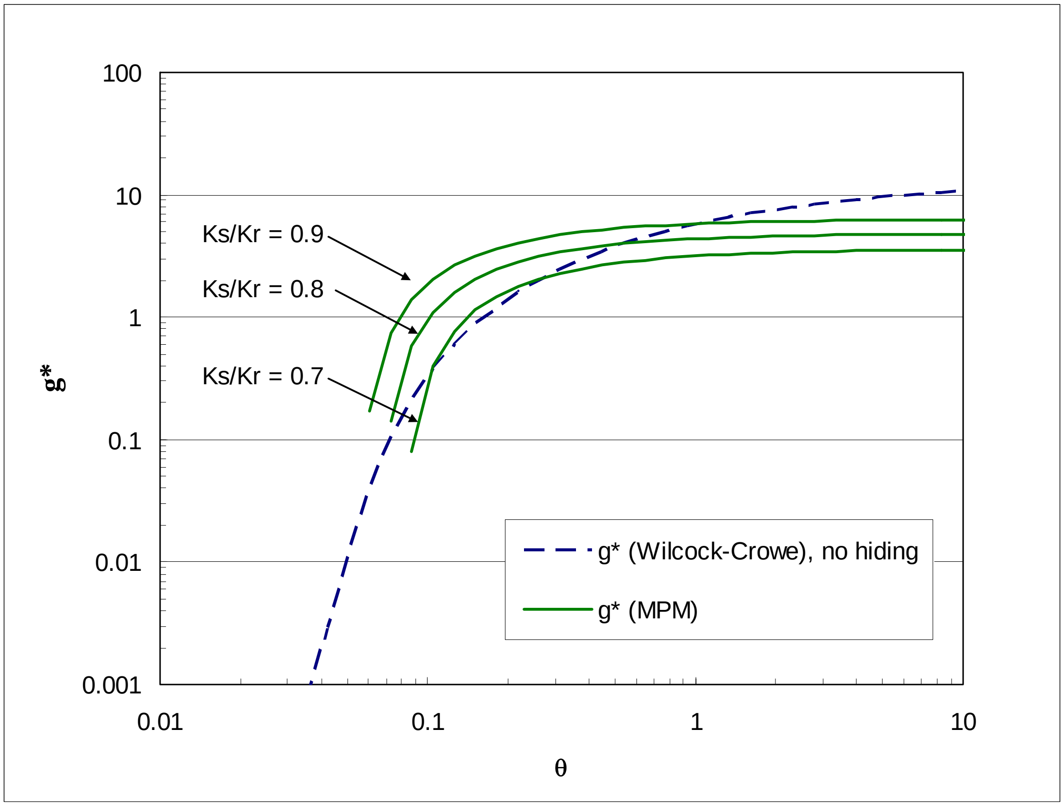


Figure 6.12. Comparison between the Wilcock and Crowe method and the Meyer-Peter-Muller Method (MPM).

6.3.5. Cohesive Sediment Transport Parameters

In this study, all sediment is assumed to behave as non-cohesive sediment. Therefore, the transport equations given in the previous section are assumed to apply to the silt and clay size fractions. For the with-project conditions, almost all of the clay and a majority of the silt will be removed from behind the dam by slurry pipeline and deposited on the downstream floodplain. Under without-project conditions, the silt and clay stored behind the dam will remain behind the dam. Therefore, the erosive characteristics of the material are not important.

6.3.6. 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 Schumm (1984). The various stages are shown in Figure 6.13 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 be the 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 the 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 trapezoidal shape. The user inputs the initial width and the model calculates the evolution of this channel based on transport capacity.

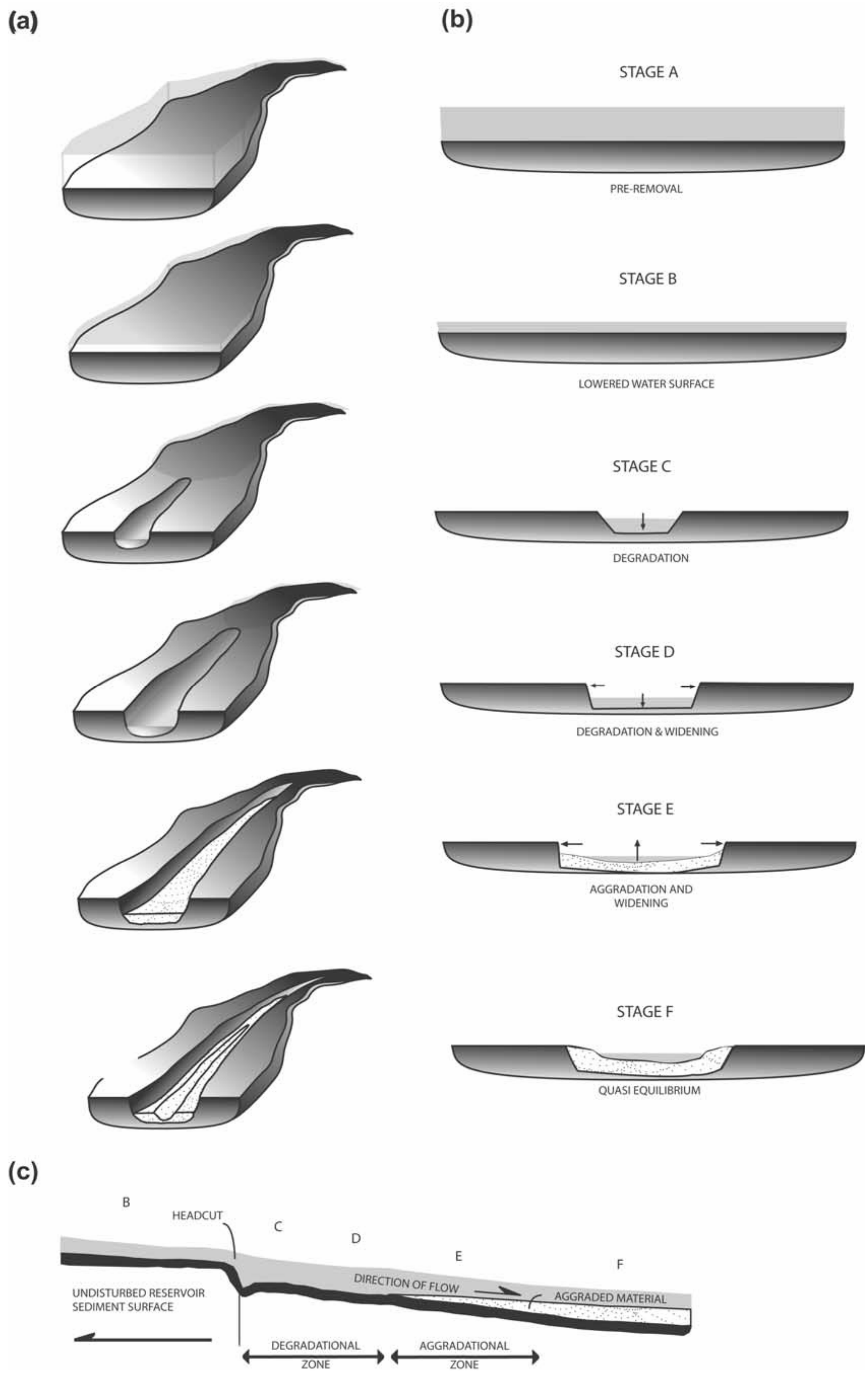


Figure 6.13. Schematic description of reservoir erosion process through delta deposits, from Doyle et al. (2003). (a) oblique view, (b) cross section view, (c) profile view.

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 other cross sections. The non-equilibrium sediment transport method modifies the sediment transport capacity using the following equation:

$$\frac{dqC}{dx} = \min\left(\alpha w_f, q/L_b\right)\cdot\left(C^* - C\right)$$

where q is the flow rate for unit width, C is the sediment concentration, x is the stream wise distance, α is a constant, w_f is the sediment fall velocity, L_b is the bed load adaptation length, 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 erosion 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 explain the procedure for predicting reservoir erosion, the computational procedure used within GSTAR-1D at each cross section within the reservoir is given below:

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. In addition, the erosion limits caused by bank protection are included in the model. If the water elevation is below the bank protection elevation, the banks are not allowed to erode. If the water elevation is above the bank protection, the portion of the bank that is wetted and above the bank protection is allowed to erode.

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 GSTAR-1D.

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 assuming the dam is removed instantaneously. While this alternative is not the preferred alternative, it is used to demonstrate the incision and widening capabilities of GSTAR-1D. The results are shown in Figure 6.14 to Figure 6.16. The model predicts the first flood erodes to the pre-dam thalweg elevations through most of the delta region (Figure 6.14). 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 6.15 and Figure 6.16. 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 GSTAR-1D.

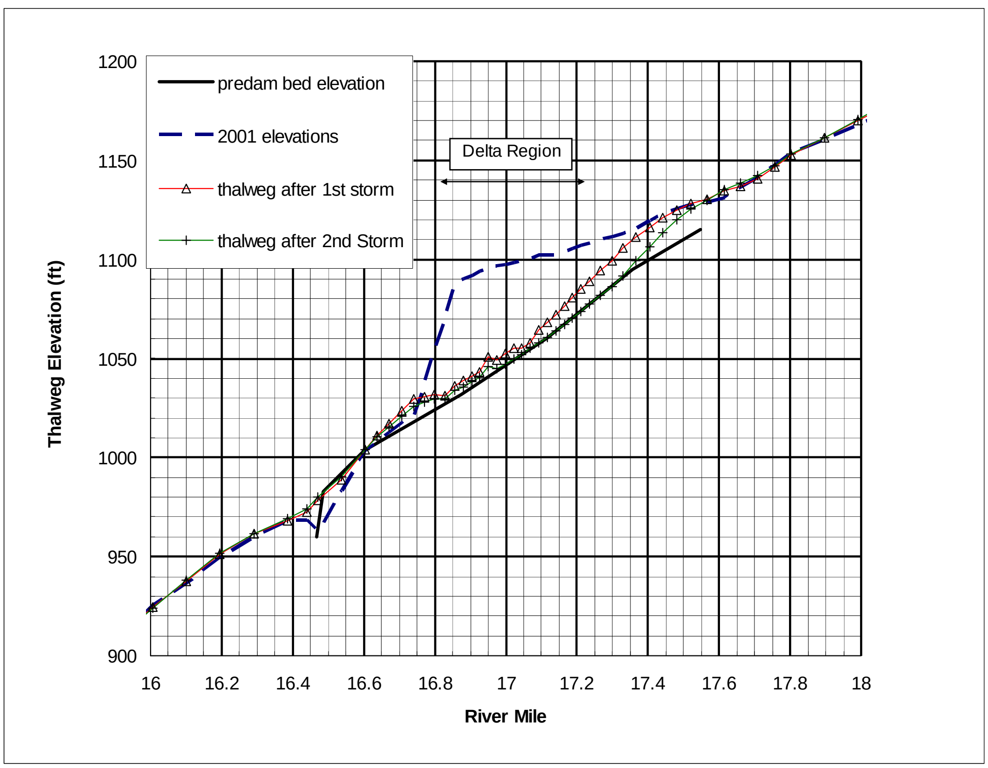


Figure 6.14. Thalweg elevations through reservoir region for Alternative 2a after the simulation of the 1998 flood twice in succession.

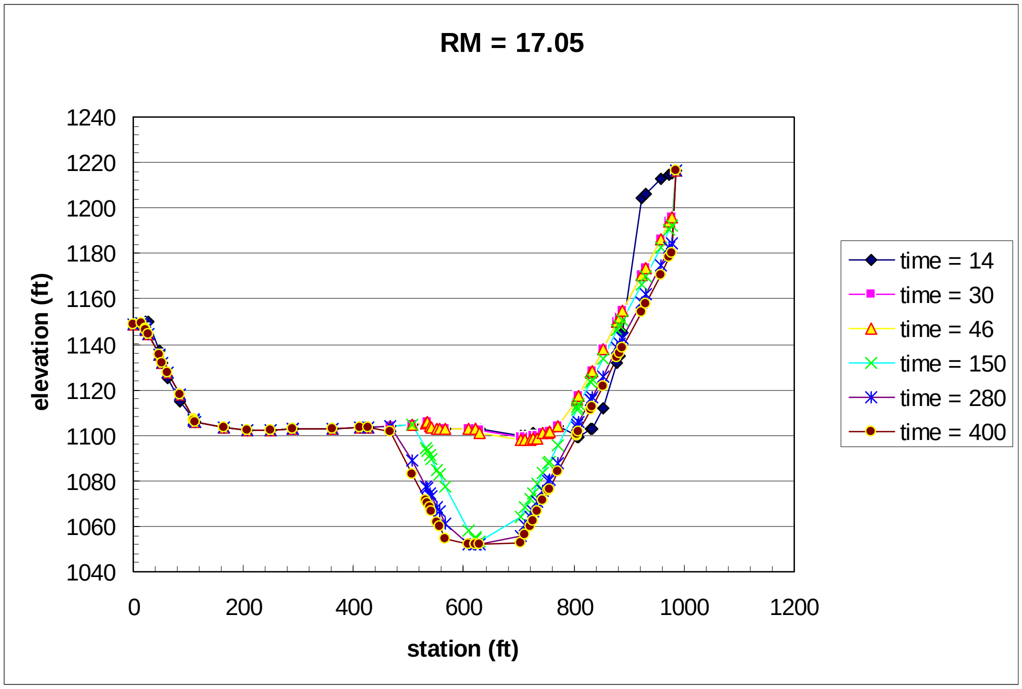


Figure 6.15. Example of Cross Section at Reservoir Delta for Alternative 2a for Two 100-yr Floods in Succession.

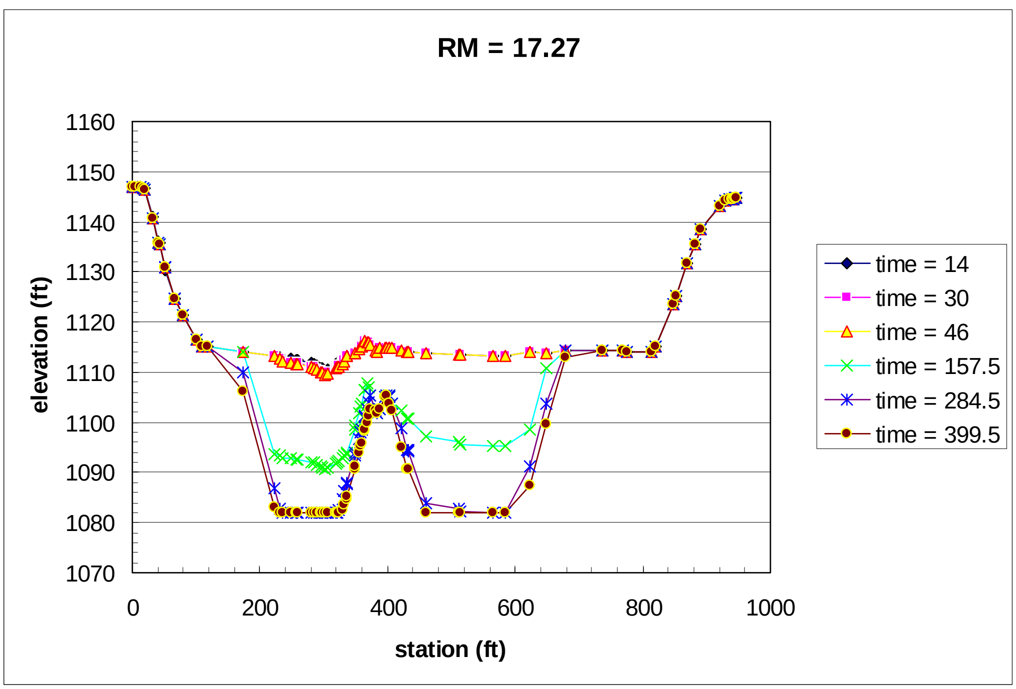


Figure 6.16. Example of Cross Section in Upstream Delta for Alternative 2a for Two 100-yr Floods in Succession.

6.4. Changes in Sediment Modeling from Feasibility

The Feasibility level report on the hydraulics, hydrology, and sediment transport was completed in 2004 (Reclamation, 2004). In this report, the sediment modeling was updated in this report and new simulations were performed. There were changes in the input data that caused differences between the Feasibility report and the current report. The most important changes are as follows:

1. Hydrology: Different hydrological scenarios were simulated. In the Feasibility phase, the hydrology used in the simulations was derived from the record from 1991 to 2000. This period was relatively wet but lacked floods equal to or greater than the 50-yr flood. For this report, several hydrologic scenarios were used in the simulation. The 15-minute flows were reconstructed from the historical daily average flow data. The period from 1950 to 2001 was used in the simulations performed in this study. This includes the 1969 and 1978 floods, the two largest recorded floods. In addition, 40 different stochastic 50-yr hydrographs were simulated. In general, larger floods will increase the magnitude of bed elevations changes.

2. Topography: Aerial LIDAR was collected in the entire Ventura River Basin in February 2005. Cross sections used in this study were derived from this 2005 LIDAR information. The feasibility study was based upon 2001 topography collected by Photogrammetry. A comparison between the 2001 and 2005 topographies showed little difference in general. This would be expected because only one large flood (January 2005) occurred between the two surveys. One exception to the agreement between the two surveys include the area around Matilija Hot Springs, approximately 1000 to 2000 feet downstream of Matilija Dam. However, ground surveys proved that the 2005 LIDAR survey overestimated the channel bed elevations in this area by approximately 3 feet. The actual bed elevations were similar to the bed elevations measured in the 2001 photogrammetry survey. The cross sections used in this study were corrected using the ground survey information.

3. Tributary sediment loads: Additional suspended sediment data was collected in 2005 and 2006 at North Fork Matilija and San Antonio Creeks and the sediment rating curves were updated.

A new GSTAR-1D code was also used. In the previous study, Version 1.0 was used in the study. The version used in this study was 1.1.4. The changes to the code are summarized in the “Version History” file located on the Web at: www.usbr.gov/pmts/sediment. The contents of the file are copied below:

Version 1.0.1 – August 1, 2005

1. Fixed bug in interpolation of D03 record.

2. Changed bank stability routine so that sediment eroded due to angle of repose conditions is added to the sediment continuity equation as a lateral sediment input.

Version 1.0.2 – October 14, 2005

1. Changed record FIM and FIW to include minimum and maximum horizontal locations of erosion and deposition.
2. Ensured water surface does not because adverse when energy balances cannot be satisfied in steady flow solver.
3. Added the ability to change the weight given to the bed load during transfer of material to the sub-layer. See record SAT.

Version 1.1 – April, 2006

1. Changed subroutine "repose" so that fixed points are not adjusted by angle of repose conditions. For example, if point was set fixed from erosion in FIM or FIW record then angle of repose condition does not apply.
2. Added XRX record to account for riprap in section.
3. Fixed bug when using US1. The sediment load for the first size fraction was incorrectly assigned.
4. Increased number of digits output in mass balance file.
5. Added bed load adaptation length.
6. Required that bed gradation for all layers (including active layer) be entered.
7. Prevented the bed from exceeding the minimum bottom elevation.
8. Added ability to enter multiple GIS referenced locations within the cross section in record XLS.
9. Fixed bug in weir subroutine. Submergence ratio was not initialized properly.
10. Fixed initialization of bottom elevation.
11. Added expansion and contraction coefficients to the steady flow equations. These replace the local energy loss coefficient.
12. Changed many subroutines so all variables were passed through the subroutine call.
13. Created error file to which errors are written.
14. Added adaptive time step for sediment computations.

Version 1.1.1 – August, 2006

1. Improved convergence of unsteady flow routing.
2. Changed unsteady sediment transport routing to be consistent with Greimann et al. 2006 (submitted to J of Hydr. Engr in Aug 2006).
3. Added output file "*_OUT_TimeSeries.DAT" which contains times series data at select cross sections.
4. Added output file "*_ERR.DAT" which contains run time error messages. Please check this file if program terminates before simulation is finished.
5. Changed output file naming convention so that multiple simulations can be performed in same directory.

Version 1.1.3 – September, 2006

1. Fixed bug that moved channel endpoints when performing angle of repose routine.
2. Fixed error in friction slope for first cross section when using unsteady flow.
3. Fixed error in method used to compute equilibrium sediment inflow for ISOLVES = 2.
4. Fixed error in computation of material volume exiting reach, reported in mass balance output.
5. Prevented angle of repose subroutine from eroding points below minimum bottom elevation.

6. Made unsteady flow routing stable for "waterfall" conditions if ISOLVE = 2 or 3. Upstream weighting of friction slope for supercritical flow.
7. The reading of variables during a 'Hotstart' was debugged.

6.5. Testing of GSTAR-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 GSTAR-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 sediment rating curves for the incoming and tributary sediment loads were assumed the same as was used for the current conditions. The hydraulic roughness is the same as used in the current conditions HEC-RAS hydraulic modeling. Bed material data is not 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 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 1.5) 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.

Summary of Historical Comparison

The comparison between measured and computed thalweg elevation change from 1971 until 2001 is shown in Figure 6.17 and Figure 6.18. The model predicted that the reach from the dam until Robles Diversion (RM 16 to 14) would not degrade, but the measured data shows that it did. One reason for the discrepancy is that the model uses the bed material data from 2001, but the bed material in 1970 may have been significantly finer in this reach. Below a dam, one would expect the riverbed to become gradually coarser over time.

The area of erosion from Robles Diversion until RM 12.6 is reasonably well predicted. The location of maximum erosion was predicted slightly downstream of where the actual maximum occurred. However, the extents of the eroded reach were accurately predicted.

The length of river eroding from RM 12 to 10.5 was predicted to be shorter than measured and the magnitude of erosion was slightly greater.

The next reach downstream from RM 10 to 8 was predicted to deposit around 3 to 4 feet for almost its entire length. The measured deposition varied between 0 to 3 feet in this reach. The measurements showed degradation from RM 6 to RM 1 while the model predicted degradation only from RM 6 to 3.5.

In general, the model predicted the locations of significant deposition and erosion reasonably well. However, at any particular cross section there could be significant discrepancies between the simulation and reality.

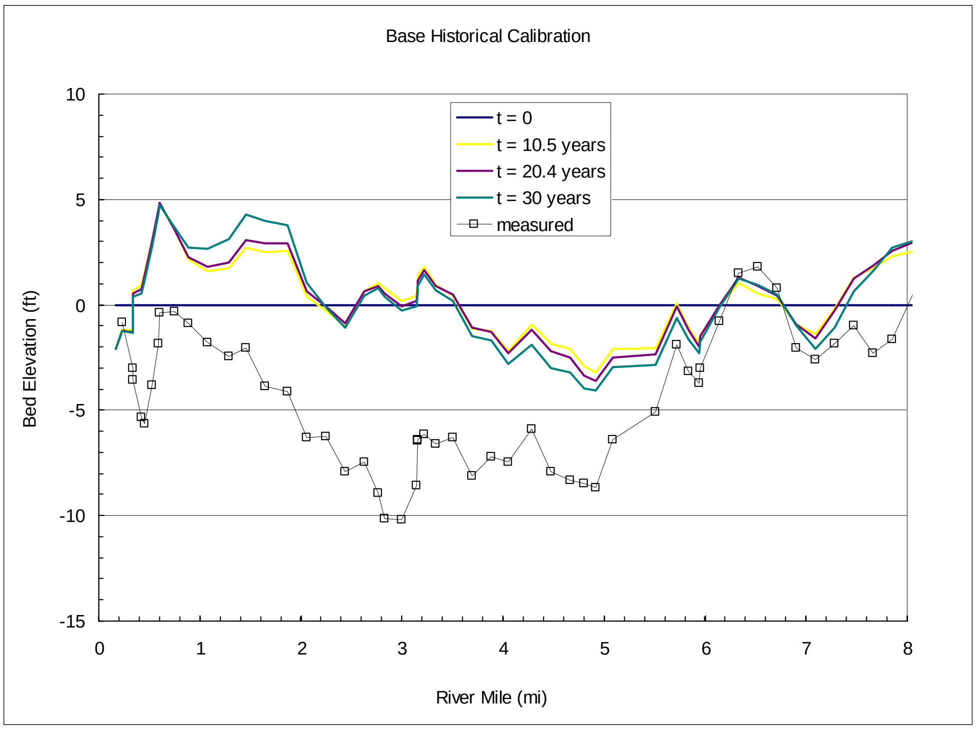


Figure 6.17. Computed Thalweg Elevation Change from 1971 to 2001, RM 0 – 8.

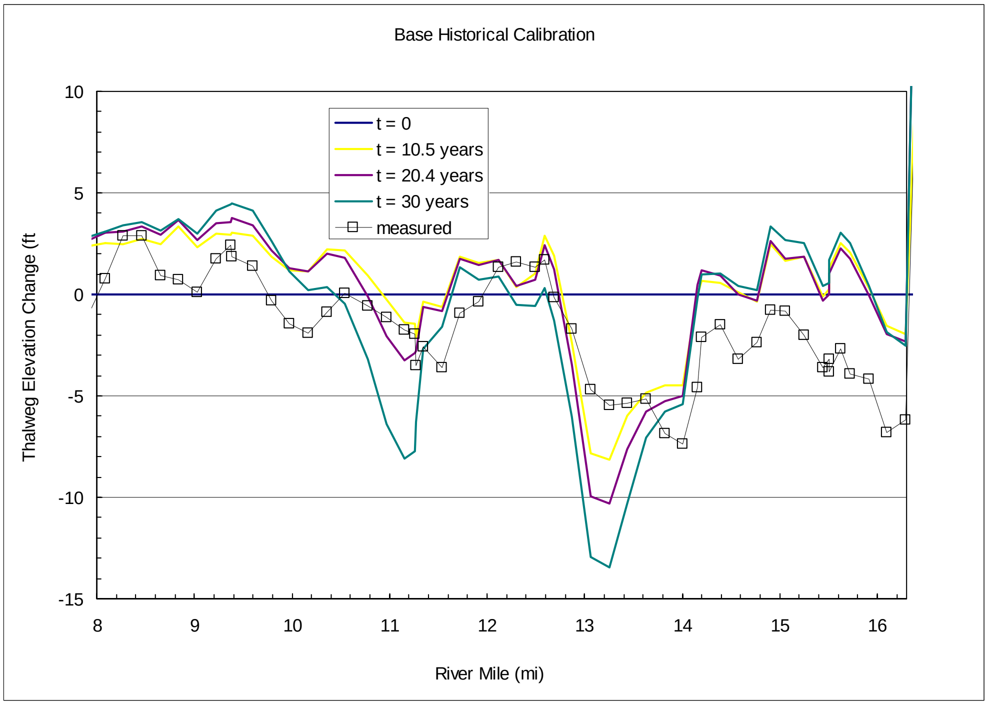


Figure 6.18. Computed Thalweg Elevation Change from 1971 to 2001, RM 8 – 16.

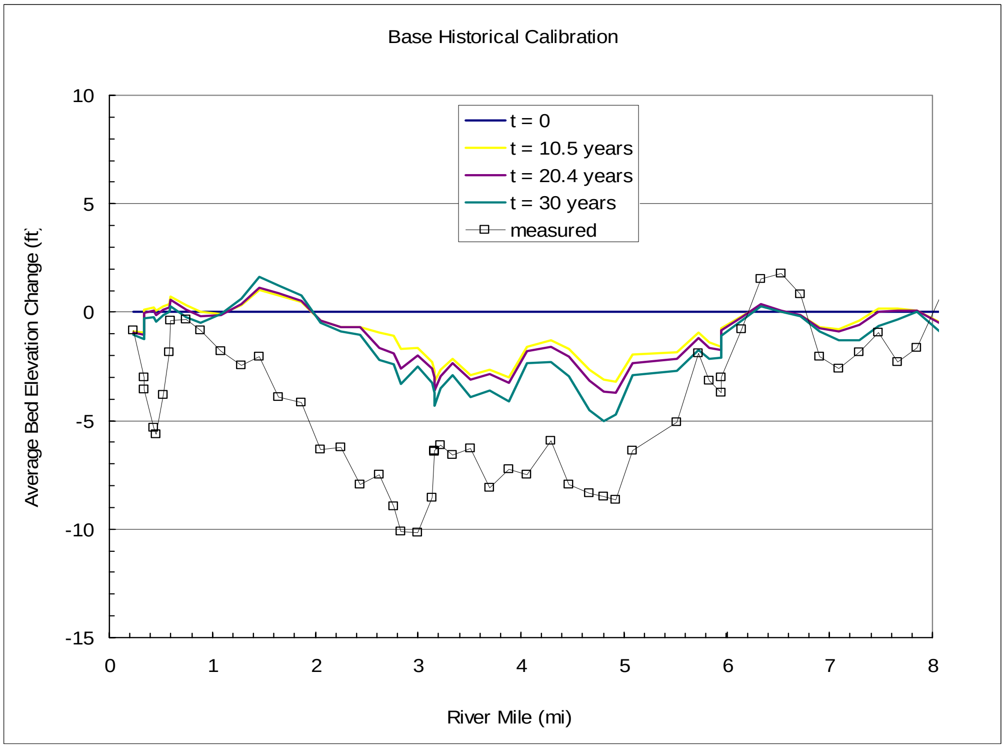


Figure 6.19. Computed Average Bed Elevation Change from 1971 to 2001, RM 0 – 8.

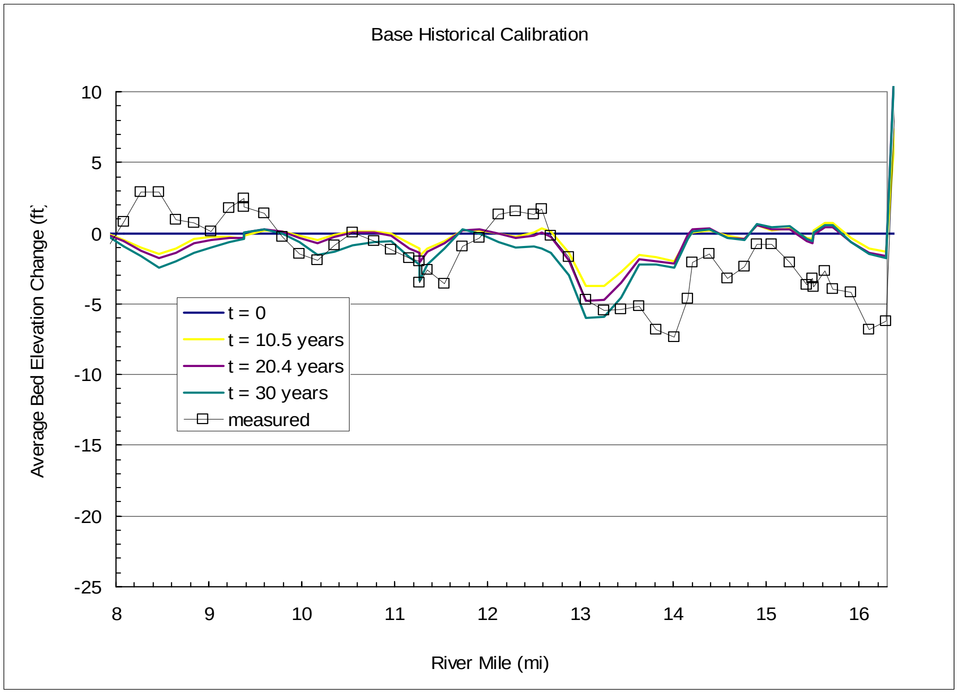


Figure 6.20. Computed Average Bed Elevation Change from 1971 to 2001, RM 8 – 16.

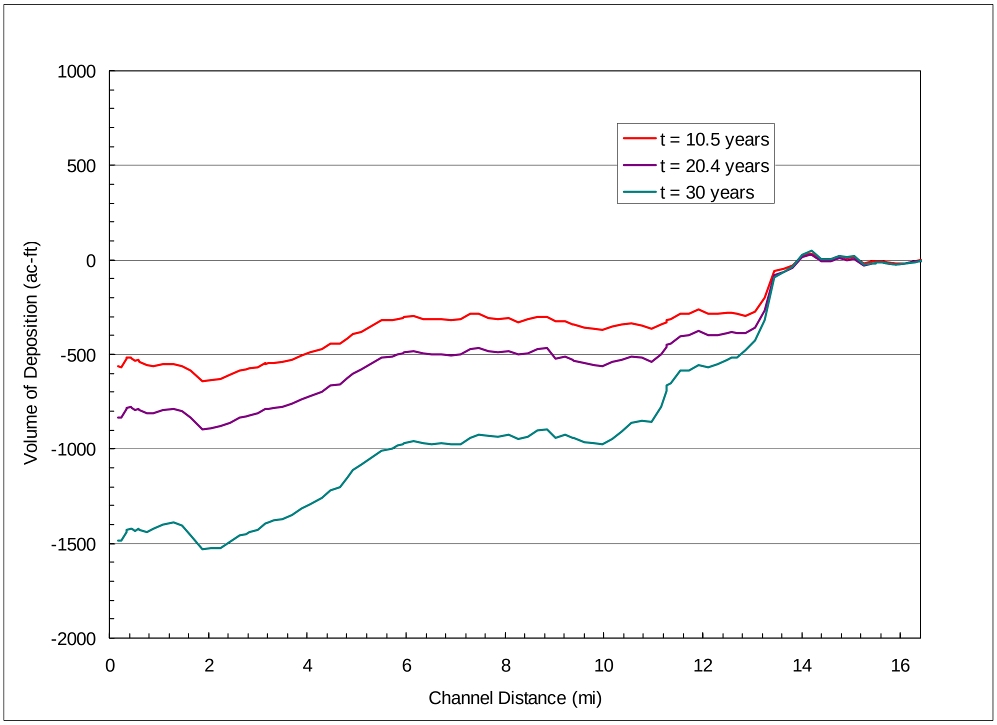


Figure 6.21. Computed Volume of Deposition between 1971 to 2001.

6.5.1. Model Sensitivity

Several model parameters were adjusted to evaluate the model sensitivity. The description of the model parameters that were adjusted is given in Table 6.7.

Increasing the roughness generally had a small effect on the erosion and deposition values predicted along the river. The upper reaches were slightly more sensitive to changes in roughness than the lower reaches. Increasing the roughness increased the erosion from RM 14 to 13.5, but increased the deposition from RM 12.5 to 11.5.

Increasing the active layer thickness, or the thickness over which the surface bed material is assumed to mix, generally increases the magnitude of erosion of deposition within the reach. However, even doubling the active layer thickness to almost 6 feet had a relatively small effect. The base active layer thickness of 2.9 feet is already considered relatively large and its value should probably not be increased significantly.

Using a different transport formula had a larger effect on the model results. The Parker (1990) bed load transport formula was used to compare against the Wilcock and Crowe (2003) formula. Generally, the Parker transport formula predicted greater erosion than the Wilcock and Crowe formula (Figure 6.22 and Figure 6.23).

Table 6.7. Description of Model Sensitivity Simulations.

#	**Description**	**Values**
0	Base	
1	Increased Roughness	Manning’s n increased to 0.05
2	Increased Active Layer	Doubled Value of Active Layer to 5.8 feet
3	Parker Transport Formula	Used Parker’s bed Load formula instead of Wilcock and Crowe

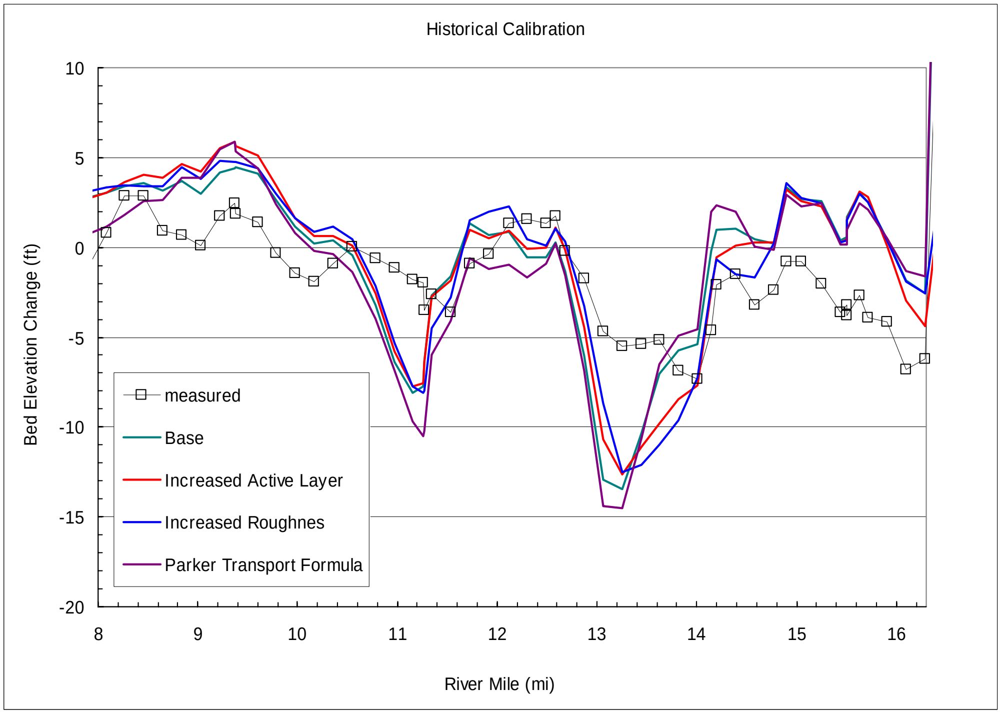


Figure 6.22. Sensitivity of Model Results to Various Model Parameters, RM 16 to 8.

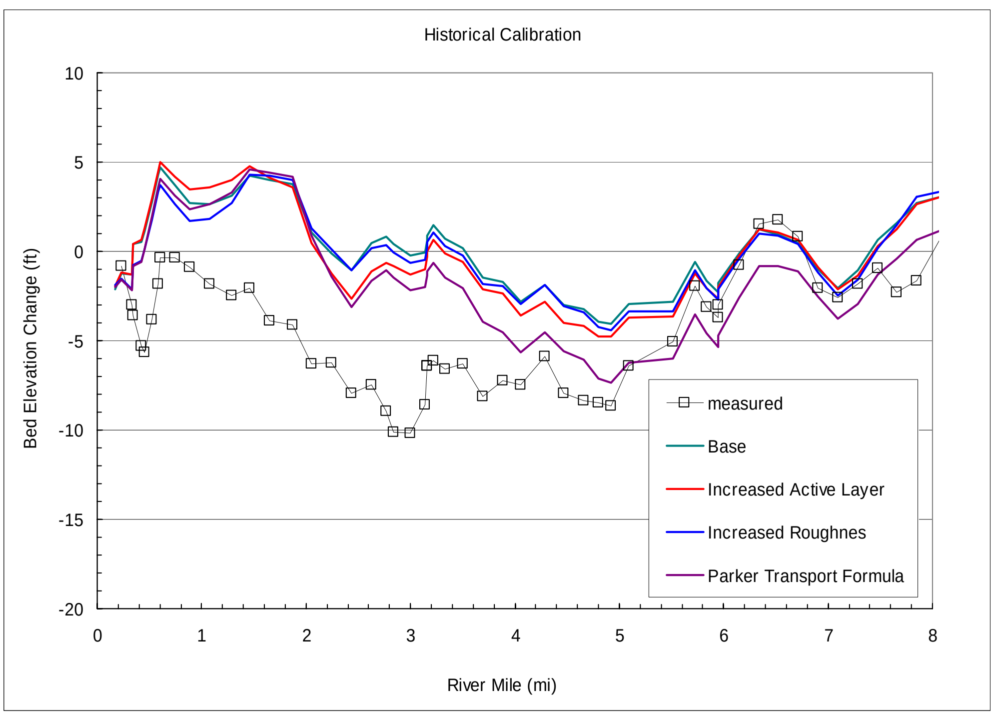


Figure 6.23. Sensitivity of Model Results to Various Model Parameters, RM 8 to 0.

7. Project Features

The next chapters will describe estimated future conditions under without- and with-project conditions.

The without-project conditions assume the following:

1. Matilija Dam remains in place indefinitely.
2. The Camino Cielo Bridge remains as it currently is.
3. Future Robles Diversion operations are similar to current operations.
4. The Santa Ana Bridge is not modified.

The with-project conditions assumed the following project features:

1. Removal of reservoir fines by hydraulic slurry line. Approximately 2.1 million yd^3 of sediment in the reservoir area will be removed and deposited on the terraces in the downstream river valley.
2. Complete removal of dam in one stage after the reservoir fines are removed.
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. Camino Cielo Bridge is replaced with a bridge with at least 20-yr flow capacity.
8. A high-flow sediment bypass is constructed at Robles Diversion
9. The flow capacity at Santa Ana Bridge is increased by creating another bridge opening.
10. Levees are constructed or increased in elevation at Meiner Oaks, Live Oak Acres and Casitas Springs so that the 100-yr flood does not overtop the levees

Figure 7.1. Oblique View of Channel Immediately after Dam Removal.

8. Future Conditions Hydrology

8.1. Future Without-Project Conditions Hydrology

Matilija Dam will continue to fill with sediment and the effective storage of the dam will continue to decrease. As mentioned previously, Matilija Dam does not currently affect the peak flows and therefore additional sedimentation in Matilija Dam will not affect the peak flows.

Currently, Matilija Reservoir allows additional water to be diverted at Robles Diversion as demonstrated in Section 2.3. It is estimated that Matilija Reservoir with a reservoir capacity of 470 ac-ft could potentially increase the diversion volume at Robles by up to 560 ac-ft/yr. After the present 470 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 2006, of Matilija Dam is shown in Figure 8.1. The benefit was assumed to decrease linearly with storage capacity of Matilija Reservoir. The storage capacity was taken from Table 11.1. Based on this analysis, the total benefit of Matilija Dam under the Without-Project Conditions is approximately 5,200 ac-ft from 2006 until the reservoir capacity is completely gone, which occurs effectively in 2025.

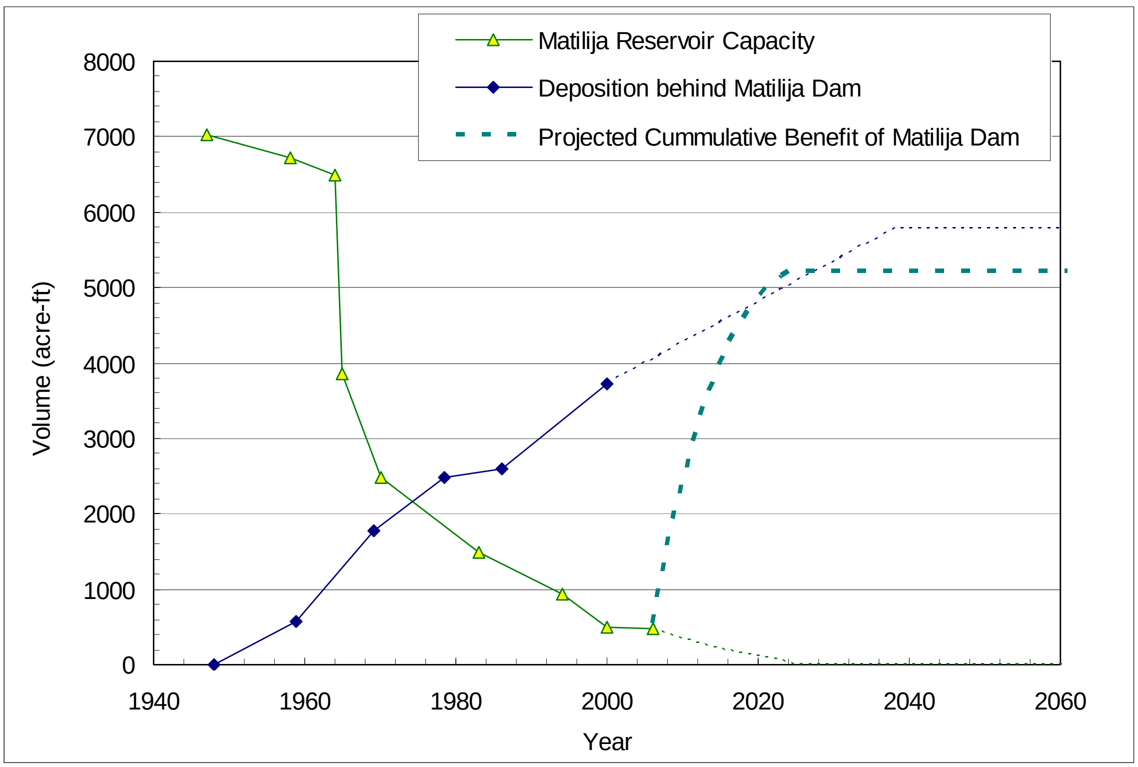


Figure 8.1. Storage Capacity of Matilija Reservoir and Projected Benefit of Matilija Dam to the Amount of Water Diverted at Robles.

Another important factor is that there is some evaporation loss due to the open pool of water of Matilija Reservoir. The surface area of 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, 1994). Native species were found to have a water use of 1.9 ac-ft/ac/yr in the Santa Ana Basin (Iverson, 1994). 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 would be between 40 to 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 may cause between 2,000 to 11,500 ac-ft of water loss due to evapo-transpiration.

8.2. Future With-Project Conditions Hydrology

Because Matilija Dam has less than 500 ac-ft of storage remaining, it does not affect the flood peaks in this area. For example, 10,000 cfs would fill a dry reservoir in approximately 36 minutes. The reservoir will already be full when the flood peak arrives and the reservoir will provide no attenuation of the flood peak.

However, removing the dam may increase the low flows in the area because less evaporation will occur in the reservoir area and more flow may reach the downstream river channel. In the previous section, it was estimated that Matilija Dam could cause up to 2,000 to 11,500 ac-ft ac-ft of water loss due to evaporation over the next 50 years. This equates to an average additional flow of between 0.05 to 0.3 cfs in the river. The flow is below 1.3 cfs 10 % of the time on average at the Matilija Hot Springs Gage, so increaing the flows by 0.3 cfs may have a measurable effect on the very lowest flows.

9. Future Conditions Groundwater Hydrology

9.1. Future Without-Project Conditions

Under Future Without-Project Conditions, no significant change to the groundwater hydrology is expected, if the groundwater pumping remains approximately the same. However, because of increased development in the Ventura Basin, it is likely that the groundwater extraction will increase and, on average, the groundwater elevations in the groundwater basins may decrease.

9.2. Future With-Project Conditions

For the With-Project Condition, the sediment transport modeling shows that the release of this material would not substantially change the composition of the Ventura River Bed (Figure 11.26 to Figure 11.28). The silts and clays would not permanently deposit onto the active riverbed. The only material that would deposit on the riverbed is cobble, gravel and some coarse 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).

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.

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. Figure 11.26 shows the change to the d_{16}. The d_{16} is the sediment 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 near current conditions. In most reaches, the d_{16} would be above 10 mm for all locations above River Mile 2. The d_{50} remains above 60 mm for all reaches above River Mile 2 after dam removal. The silts and clays would not permanently deposit onto the riverbed. 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 6.10). 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. In addition, 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, very little of the material in the disposal site sediment will 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.

10. Future Conditions Hydraulics

10.1. Future Without-Project Hydraulics

The flooding risk under Without-Project conditions is discussed in the following sections. Locations are identified by reach and RM. This section references results given in Section 11 “Future Conditions Channel Morphology, Sediment Transport, and Reservoir Sedimentation”, in which the future deposition in the Ventura River is discussed.

The increase or decrease in the Without-Project 100-yr flood elevations relative to the current condition are given in Figure 10.1. A moving average is also shown on the graph that is the average difference if flood elevations of the nearest seven cross-sections. The moving average is useful in spotting trends. The sediment model generally simulates the behavior of a reach better than the behavior of a specific cross section.

At any given cross section, changes between the current condition and the future without project condition less than 1 foot are not considered significant and the flood elevation would be considered essentially unchanged from the current condition. This is because the sediment model and survey accuracy should not be considered less than +/- 1 foot. Changes more than 2 feet are considered significant and the flood elevations are considered to have raised a significant amount. Changes that are between 1 to 2 feet would potentially be significant and will be dealt with on an individual basis.

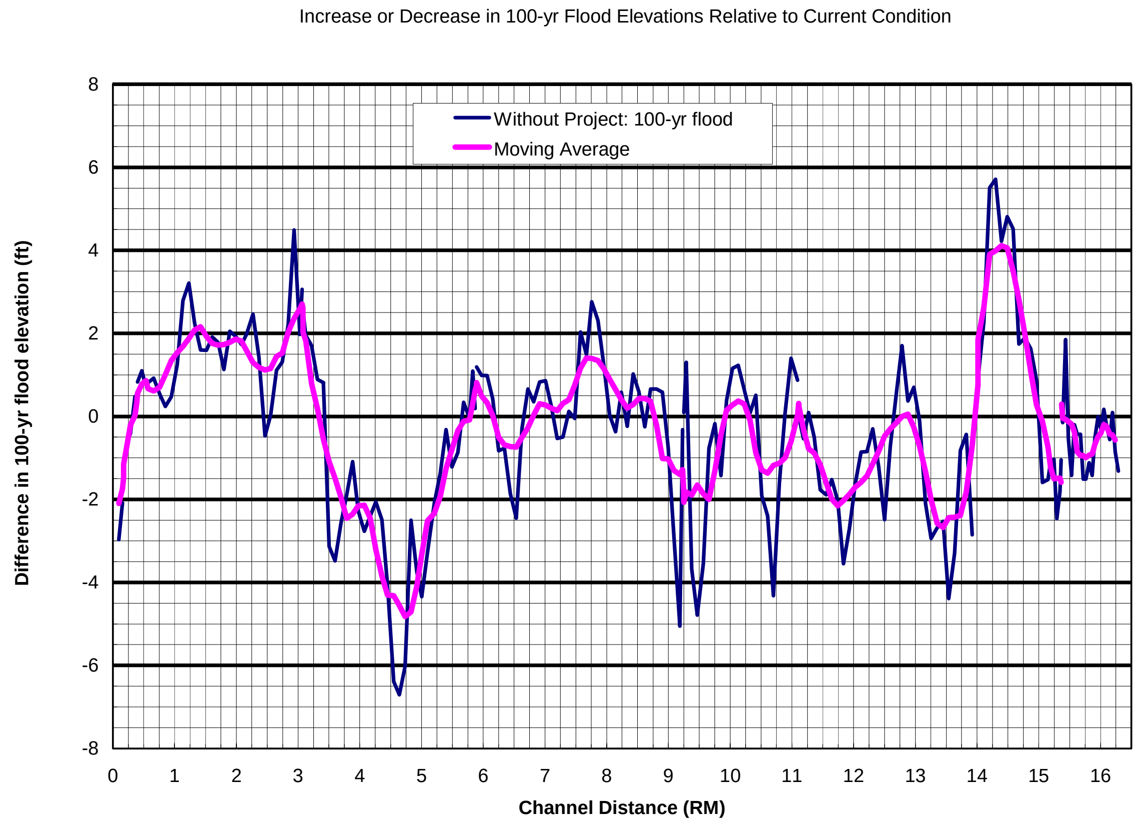


Figure 10.1. Change in 100-yr flood elevations between Current Condition and Future Without-Project Conditions.

Reach 6b – RM 16.4-15.0

Reach 6b begins immediately downstream of Matilija Dam and extends downstream to the canyon mouth. There is development at the "Matilija Hot Springs" facility and around the Camino Cielo Bridge.

Matilija Hot Springs: The "Matilija Hot Springs" facility is located at approximately RM 16.1. This reach is controlled by very large boulders, bedrock, and a concrete weir at the stream gage. The reach adjacent to the Hot Springs facility will remain stable for the near future. No significant change to the flood risk is expected.

Camino Cielo: The flood risk at this location is similar to the current condition. The 100-yr flood elevations upstream of the bridge are slightly higher than the current condition, but the increase is not considered significant. Downstream of the bridge, there was a slight decrease in flood elevations of 1 to 2 feet.

Reach 6a – RM 15-14.0

Reach 6a begins at the canyon mouth and extends downstream to Robles Diversion Dam.

Meiners Oaks Area: There are approximately 20 structures located along Oso Road and North Rice Road between RM 14.4 and 14.1 within Reach 6a. (There are additional structures within this community downstream of 14.1, but located in Reach 5.) All of these structures are constructed at grade. There is no functional levee, but most all of these structures are around 40 feet above the 100-year floodplain. There may be some flood risk caused by the flows originating from Cozy Dell Canyon, but these are not considered as part of this study. There is considerable increase in flood elevations in this area, but no structures are affected.

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. Because of the maintenance activities in the area of Robles Diversion, there should be no significant change to flood elevations in this area.

Reach 5 – RM 14.0 – 11.10

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

Continuation of Meiners Oaks Area: The flood elevations downstream of Robles Diversion are expected to continue to decrease. However, the risk of channel avulsions will remain and floods equal to or greater than the 50-yr flood could erode significant portions of the protective levee.

The flood impacts from Cozy Dell were not considered in this study because the project will not affect them. However, a complete flood risk analysis should consider the flows from Cozy Dell.

Burn Dump: The flood elevations in this area will remain relatively constant. However, the river is braided in this section and the plan form of the river will be altered after every major flood. The levee upstream of the Burn Dump will likely be eroded by floods and provide little flood protection.

The flood risk to the OVSD sewer lines that are along the east side of the river from RM 12.12 to Baldwin Road Bridge will remain similar to current conditions. The sewer line upstream of Baldwin Road will remain in the 100-yr floodplain.

Reach 4 – RM 11.10 – 7.93

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

Between Baldwin Road and Live Oak Acres: There is some lowering of the 100-yr floodplain expected from RM 10.8 to 10.4. This lowering will remove parts of some properties from the 100-yr floodplain on the west side of the river.

The Live Oak Drain enters the Ventura River at approximately RM 10.15. The 100-yr flood elevation is expected to rise approximately 1 foot in this area. This raise is relatively minor and is on the verge of being considered not significant. However, it may be necessary to plan for increased flood elevations along the Live Oak Drain.

Live Oak Acres: The Live Oak Levee is on the west bank of the Ventura River and extends from RM 9.25 to RM 10.15. It protects the populated area of Live Oak. The flood elevations in this reach are expected to remain stable or decrease in the future so that flood capacity is maintained at current levels.

The bank erosion in this area is expected to continue. The Live Oak Levee is a significant constriction to the flow at this location and will cause increased velocities in the main channel. The County repaired the Live Oak Levee in 2005 following a storm event and there will likely be more failures in the future. The County also placed groins along the East side of the river to prevent further erosion of the terrace at RM 9.6. The groins are of sufficient size to prevent erosion along that bank, but there presence may increase the likelihood of erosion along the Live Oak Levee.

The Santa Ana Bridge is expected to maintain its current level of flood capacity. The County has a maintenance program to remove sediment from underneath and upstream of the bridge to maintain flood capacity. The sediment model predicts that the bridge would still maintain 500-yr flood capacity without any sediment removal in this area. However, because of the uncertainty associated with sediment predictions, it is recommended that the sediment excavation program continue.

Downstream of Santa Ana Bridge: There are at least three residences located between RM 9.2 and 8.9 on the west side of the river. A levee high enough to protect these residences from the 100-yr flood exists, but high river flows could easily erode the riverbed material from which it was constructed. The levee is not assumed to protect the residences downstream of the Santa Ana Bridge. Therefore, the flood risk will remain similar to current conditions.

Reach 3 – RM 7.93-5.95

Casitas Springs: The Casitas Springs Levee runs from RM 6.50 to 7.67 along the east side of the river and protects the town of Casitas Springs. Under current conditions, the 500-yr flood is contained by the levee, but under future without-project conditions, the 500-yr flood exceeds the levee elevation at RM 7.67. All floods smaller than the 500-yr flood are contained by the Casitas Levee.

An additional flood risk is caused by the Fresno Drain that passes through Casitas Springs and through the Casitas Levee. The drain is open and near the elevation of the riverbed, rendering the Casitas Levee ineffective below RM 6.8. The 10-yr flood can inundate residences on the east side of the river below RM 6.8. The future flood risk will remain similar to the current conditions.

Several OVSD pipelines will continue to be in the 100-yr floodplain in this reach (Figure 4.19).

Foster Park Area: There is a community upstream of Casitas Vista Bridge located on the west side of the river opposite Foster Park from RM 6.4 to 6.3. There is a levee constructed of riverbed material that is highly vegetated and that will likely protect the community from the 10-yr flood. There was no evidence of flooding from the 2005 flood. The river is beginning to erode the upstream end of this levee and it is likely that flows larger than the 2005 flood would erode more of this levee and potentially flood the entire community. The flood mapping shows this community flooded for the 50-yr flood and larger.

Reach 2 – RM 5.95-0.6

From RM 5.8 to RM 3.3, the simulations predict that the river will degrade and flood capacity will increase. Therefore, the flood risk will remain the same or decrease in this reach under without-project conditions. It is expected that some reaches are controlled by bedrock and that further erosion is not possible. For example, documented bedrock near the OVSD treatment plant will prevent further erosion at that location (see Figure 11.15 in next section). Therefore, the flood risk to this facility should remain similar to current conditions.

The simulations show deposition below RM 3. However, as documented in the historical calibration section, the model over predicts deposition in this reach and it is possible that this reach remains relatively stable under without-project conditions.

10.2. Future With-Project Hydraulics

The flooding risk under With-Project conditions is discussion in the following sections. They are identified by reach and RM. This section references results given in Section 11 "Future Conditions Channel Morphology, Sediment Transport, and Reservoir Sedimentation", in which the future deposition under With-Project Conditions in the Ventura River is discussed. This section discusses the impacts predicted using the mean estimate for future flood elevations. Section 12.4 "Uncertainty Analysis for Flood Protection" discusses the uncertainty associated with the future predictions. The uncertainty with future predictions is large. For example, the estimated standard deviations of the water surface elevations are over 3 feet in the reach between Matilija Dam and Robles Diversion. This indicates that we are 95% confident that the flood elevations in the upper reaches are +/- 6 feet of the best estimate. This relatively large uncertainties in the future water surface elevations need to be considered in flood management strategies.

The increase or decrease in the With-Project 100-yr flood elevations relative to the current condition are given in Figure 10.2. As a comparison, the changes to flood elevations under Without-Project Conditions are also given. The line denoted by "With-Project High Deposition" are the flood elevations used as upper estimates on the amount of deposition possible under future scenarios. The water surface elevations changes for the reach between Matilija Dam and RM 13 are shown in Figure 10.3. This figure also shows the 100-yr flood elevations after one 100-yr flood occurs after dam removal.

At any given cross section, changes between the current condition and the future without-project condition less than 1 foot are not considered significant and the flood elevation would be considered essentially unchanged from the current condition. This is because the sediment model and survey accuracy should not be considered less than +/- 1 foot. Changes more than 2 feet are considered significant and the flood elevations are considered to have raised a significant amount. Changes that are between 1 to 2 feet would potentially be significant and will be dealt with on an individual basis.

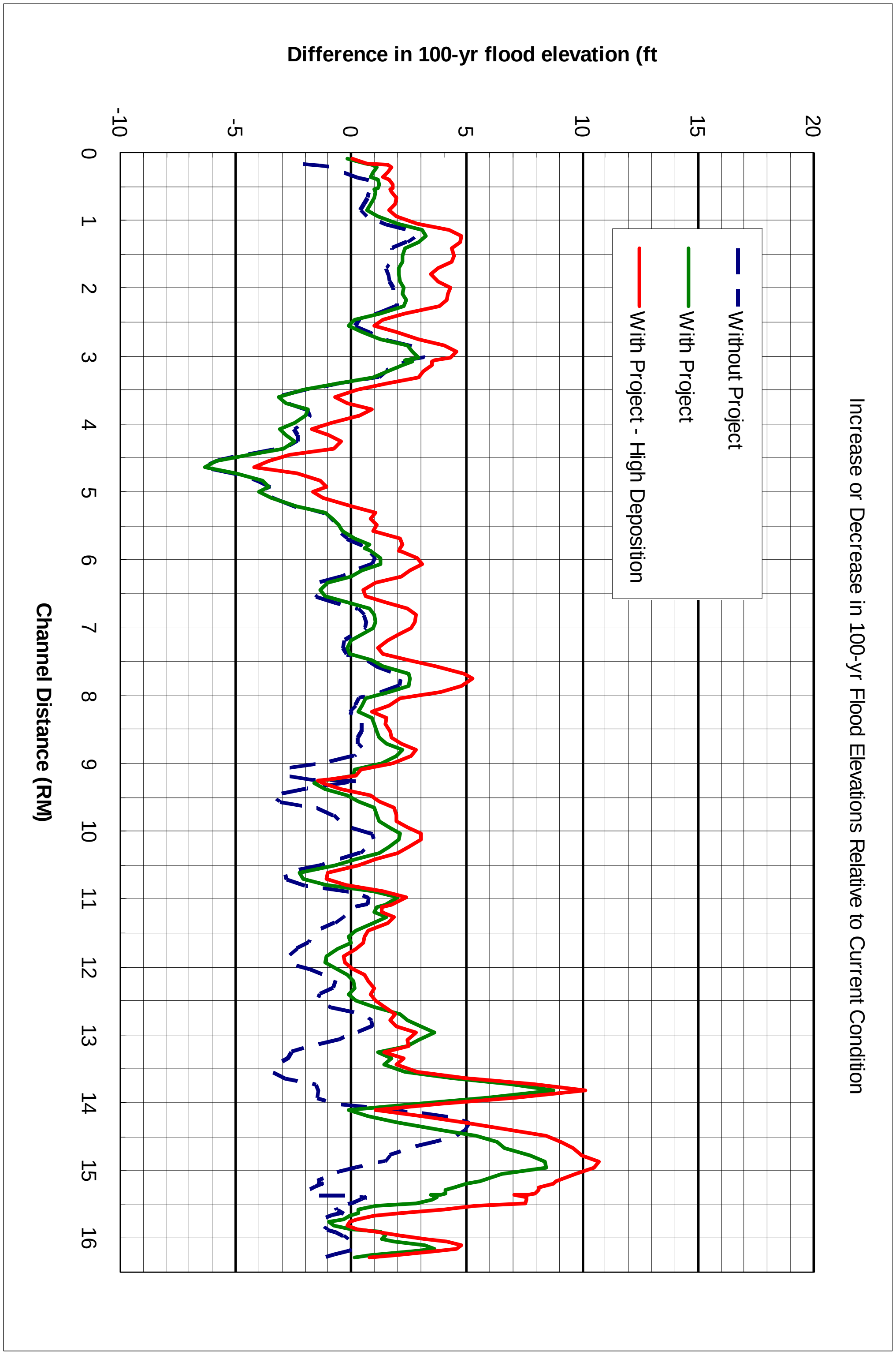


Figure 10.2. Change in 100-yr flood elevations between Current Condition and Future With-Project Conditions.

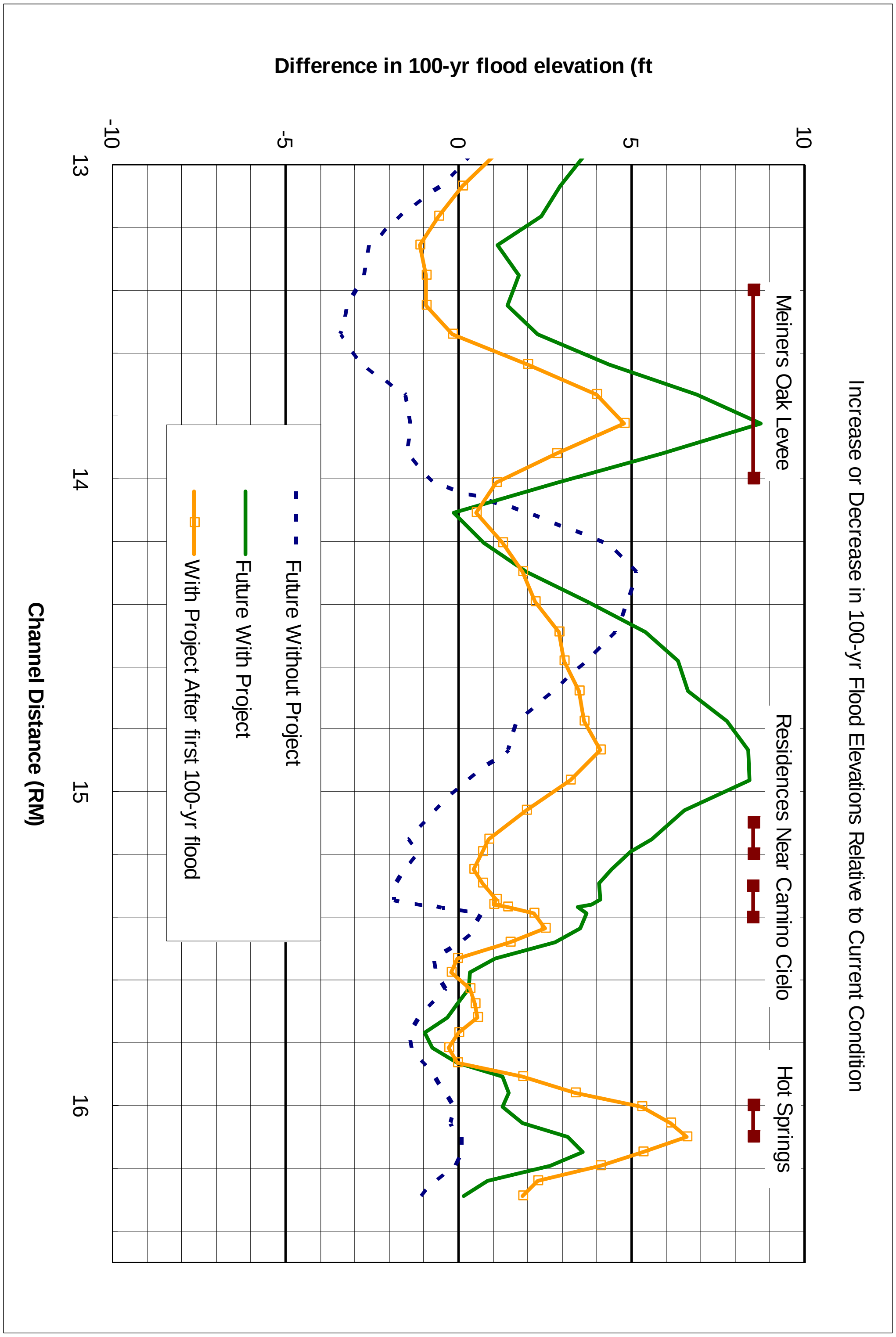


Figure 10.3. Change in 100-yr flood elevations between Current Condition and Future With-Project Conditions from RM 13 to Matilija Dam. Also shows 100-yr flood elevations after first 100-yr flood.

Reach 6b – RM 16.4-15.0

Reach 6b begins immediately downstream of Matilija Dam and extends downstream to the canyon mouth. There is development at the "Matilija Hot Springs" facility and around the Camino Cielo Bridge.

<u>Matilija Hot Springs</u>: This reach contains little development except the "Matilija Hot Springs" facility. This reach is controlled by very large boulders, bedrock, and a concrete weir at the stream gage. The Hot Springs property begins about 1000 feet downstream of the dam and extends for another approximately 1000 feet along the North side of the channel (RM 16.15 to 16.0). The best estimate under With-Project conditions is that the 100-yr flood elevations will increase by approximately 4 feet on average through this reach. The 10-yr flood will increase approximately 2 feet on average.

In this reach, the most critical time for flood risk is immediately following dam removal. If a 100-yr flood would occur immediately following dam removal the deposition could be slightly higher and the expected increase in the 100-yr flood plain would be approximately 6 feet. This would inundate most of the Hot Springs facility with about 3 feet of water. The 50-yr flood under this condition would inundate the facility with 1 foot of water.

<u>Camino Cielo</u>: There are several structures on the south side of the river and upstream of the Camino Cielo Bridge from RM 15.62 to 15.45 that are currently located near but slightly above the 100-yr and 500-yr floodplains. These structures remain above the 100-yr floodplain, but one structure located at RM 15.58 that would now inundated by the 500-yr flood under With-Project Conditions.

On the south side of the river downstream of the bridge, there is in one residence at cross section (XC) 15.1515 that is below the Current 10-yr flood elevation. The next property downstream is between XC 15.1515 and 15.0568 and is surrounded by the current 10-yr floodplain, and the structure is only 1 to 2 feet above the current 100-yr floodplain. The flood elevations are predicted to rise approximately 5 feet in this location and these structures would be inundated significantly more often.

North of the river and upstream of the Camino Cielo Bridge between XC 15.3675 and 15.3873, one structure that is currently located in the 10-yr floodplain. The 100-yr flood elevations at this location are expected to increase approximately 3 feet. The 10-yr flood elevations should also increase about 3 feet. The base of this structure would be under about 6 feet of water during the 100-yr flood.

Downstream of the Camino Cielo Bridge on the North side of the river, between XC 15.3409 and 15.2917 the property is encircled by the 100-yr flood but remains 3 feet above the 100-yr flood plain. The flood elevations are expected to increase

3 to 4 feet at this location. The 500-yr flood would inundate one structure at this location.

There is an orchard on the Northeast side of the river from XC 15.2462 to 14.9621. The flood elevations at this location are expected to increase approximately 4 feet near the orchard. Therefore, this orchard will be inundated more frequently and to greater depths under with-project conditions.

If the 100-yr flood would occur immediately after dam removal in this reach, the flood elevations for that first 100-yr flood will be less than those described above. For example, in the reach immediately around Camino Cielo, the flood elevations should only be 1 to 2 feet higher than under current conditions. Recall, however, that this is a best esimtae subject to considerable uncertainty.

Reach 6a – RM 15-14.0

Reach 6a begins at the canyon mouth and extends downstream to Robles Diversion Dam.

<u>Meiners Oaks Area</u>: There are approximately 20 structures located along Oso Road and North Rice Road between RM 14.4 and 14.1 within Reach 6a. (There are additional structures within this community downstream of 14.1, but located in Reach 5.) All of these structures are constructed at grade. There is no functional levee, but most all of these structures are around 40 feet above the 100-year floodplain. There may be some flood risk caused by the flows originating from Cozy Dell Canyon, but these are not considered as part of this study. There is considerable increase in flood elevations in this area, but no structures are affected.

<u>Robles Diversion</u>: 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. Because of the maintenance activities in the area of Robles Diversion, there should be no significant change to flood elevations in this area. Because of the highflow bypass allow more water through radial gates at a lower elevation, there should most likely be a decrease in flood elevations immediately upstream of Robles Diversion.

Reach 5 – RM 14.0 – 11.10

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

<u>Continuation of Meiners Oaks Area</u>: Under With-Project conditions, a protective levee will be constructed at this location to protect against flooding. The levee will also prevent erosion of the East bank of the river. The levee will extend from Robles Diversion (RM 14.0) to approximately RM 13.4. There will be a 5 to 10 foot increase in flood elevations under with-project conditions from Robles

Diversion to RM 13.6. This reach has experienced degradation after the construction of Matilija Dam and Robles Diversion because of the reduction in sediment supply. As shown in Figure 5.32, the 100-yr water surface elvation dropped 5 to 7 feet from 1970 to 2005. Re-supplying this reach with sediment will bring the river elevations back to pre-dam conditions. In some case, the rapid resupply may cause river bed elevations to slighltly exceed the pre-dam conditions. It needs to be recalled, however, that the uncertainty of this prediction is relatively high and the potential for increased flooding is addressed in the next section (Section 12.4 “Uncertainty Analysis for Flood Protection”).

It should be noted that the Cozy Dell drainage passes through this community on its East side and this drainage can cause substantial flooding. The flood impacts from Cozy Dell were not considered in this study because the project will not affect them. However, a complete flood risk analysis should consider the flows from Cozy Dell.

Potential Dispoal Area: A potential disposal site for the reservoir slurry material is located on the East Ventura floodplain, downstream of Meiners Oaks. This disposal site will not significantly affect flood elevations in the Ventura River, but could effect the flood flows coming from Cozy Dell. A channel of adequate sized and slope should be designed so that the Cozy Dell drainage maintains current capacity. Based upon the 2005 LIDAR survey, the lower portion of the Cozy Dell drain is approximately 40 feet wide at the top of bank and has a slope of approximately 0.008. The slope through the disposal area is approximately 0.015, and an equivalent 40 foot wide channel would have approximately 1.4 times the conveyance of the Cozy Dell drain. Therefore, the channel through the disposal area should not have to be greater than 40 feet across to maintain current flood capacity. In case the conveyance of the Cozy Dell drain is increased, it is suggested that a 40 foot top width be maintained for any channel design through the disposal area. In addition, the existing naturally formed channel downstream of the exit of Cozy Dell drain is approximately 40 wide and it is recommended that this natural formed channel be used to the extent possible.

Burn Dump: The flood elevations at this location are expected to remain relatively stable. The flood risk to the OVSD sewer lines that are along the east side of the river from RM 12.12 to Baldwin Road Bridge will remain similar to current conditions. The sewer line upstream of Baldwin Road will remain in the 100-yr floodplain.

Reach 4 – RM 11.10 – 7.93

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

Between Baldwin Road and Live Oak Acres:

The Live Oak Drain enters the Ventura River at approximately RM 10.15. The 100-yr flood elevation is expected to rise approximately 2 to 3 feet in this area. Future operation and maintenance of this drain need to accommodate the rise in flood elevations.

Live Oak Acres: The Live Oak Levee is on the west bank of the Ventura River and extends from RM 9.25 to RM 10.15. It protects the populated area of Live Oak. The height of this levee may be increased as part of the project. The flood elevations along the upstream end of the levee near RM 10.2 to 9.9 are expected to increase 2 to 3 feet. The elevations in the middle section between RM 9.9 and 9.5 will remain relatively stable. The future flood elevations were predicted to decrease along downstream section of this levee from RM 9.5 to 9.3. However, the model simulations likely overpredicted the erosion in this area and the flood elevations should remain relatively stable.

Previous analysis during the feasibility phase of the project suggested that the span of Santa Ana Bridge be increased (Appendix D of Matilija Dam Ecosystem Restoration Feasibility Study - Final Report). Even though the current analysis shows that the deposition is not large, it is still recommended that the bridge opening be increased. The current 100-yr flood is only 1-foot below the bridge opening. Even minor deposition could cause floods to back up against the bridge. This would cause flood elevations to increase significantly along the Live Oak Levee and perhaps cause it to be overtopped. This analysis assumes that the current bridge opening is increased on the left side by approximately 60 feet.

Downstream of Santa Ana Bridge: There are at least three residences located between RM 9.2 and 8.9 on the west side of the river. A levee high enough to protect these residences from the 100-yr flood exists, but it is constructed of riverbed material and may be eroded at higher flows. The levee is not assumed to protect the residences downstream of the Santa Ana Bridge. Therefore, the flood risk will remain similar to current conditions. If this bank fails, however, the integrity of the right downstream bridge abutment may be compromised. In addition, the abutment on the left side of the bridge failed during the 1998 flood (Figure 4.16). It is recommended that both the left and right banks of the downstream river channel be protected from erosion with riprap of sufficient size to remain stable during the 100-yr storm.

Reach 3 – RM 7.93-5.95

Casitas Springs: The Casitas Springs Levee runs from RM 6.50 to 7.67 along the east side of the river and protects the town of Casitas Springs. Under current conditions, the 500-yr flood is contained by the levee, but under future without-project conditions, the 500-yr flood exceeds the current levee elevation at RM 7.67. All floods smaller than the 500-yr flood are contained by the Casitas Levee.

An additional flood risk is caused by the Fresno Drain that passes through Casitas Springs and through the Casitas Levee. The drain is open and near the elevation of the riverbed, rendering the Casitas Levee ineffective below RM 6.8. The 10-yr flood can inundate residences on the east side of the river below RM 6.8. The future predictions show that the water surface elevations are within 1 foot of the current water surface elevations and very similar to the Without-project Future Conditions. Therefore, the project should not increase the flood risk of the houses surrounding Fresno Drain.

Several OVSD pipelines will continue to be in the 100-yr floodplain in this reach (Figure 4.19).

Foster Park Area: There is a community upstream of Casitas Vista Bridge located on the west side of the river opposite Foster Park from RM 6.4 to 6.3. There is a levee constructed of riverbed material that is highly vegetated and that will likely protect the community from the 10-yr flood. There was no evidence of flooding from the 2005 flood. The river is beginning to erode the upstream end of this levee and it is likely that flows larger than the 2005 flood would erode more of this levee and potentially flood the entire community. The flood mapping shows this community flooded for the 50-yr flood and larger.

Reach 2 – RM 5.95-0.6

From RM 5.8 to RM 3.3, the simulations predict that the river will degrade and flood capacity will increase. Therefore, the flood risk will remain the same or decrease in this reach under with-project conditions. It is expected that some reaches are controlled by bedrock and that further erosion is not possible. For example, documented bedrock near the OVSD treatment plant will prevent further erosion at that location (see Figure 11.15 in next section). Therefore, the flood risk to this facility should remain similar to current conditions.

The simulations show deposition below RM 3. However, as documented in the historical calibration section, the model over predicts deposition in this reach and it is possible that this reach remains relatively stable under without-project conditions.

11. Future Conditions Channel Morphology, Sediment Transport, and Reservoir Sedimentation

Estimates of future conditions relating to the channel morphology, sediment transport and reservoir sedimentation and/or erosion. First, the conditions in Matilija Reservoir are given for the without- and with-project conditions. Next, the general future predictions are given on a reach average basis using the reach definitions given in Table 1.3. Then results are given for specific locations.

11.1. Future Conditions in Matilija Reservoir

11.1.1. 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. 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 11.1. 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,

$$\mathit{Trap\ Efficency},\% = 95\left(1-\exp\left(-0.029\sqrt{S}\right)\right) \qquad \text{Eq 11.1}$$

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 11.1. The reservoir is predicted to have less than 50 ac-ft of storage by 2020.

Table 11.1. Projected deposition with dam in place.

Year	Dam Crest Elevation	Reservoir Storage (ac-ft)	Est. Trap Efficiency of Reservoir (%)	Est. Deposited Volume (yd^3)
2006	1095	470	45	6,500,000
2010	1095	350	33	6,900,000
2020	1095	110	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

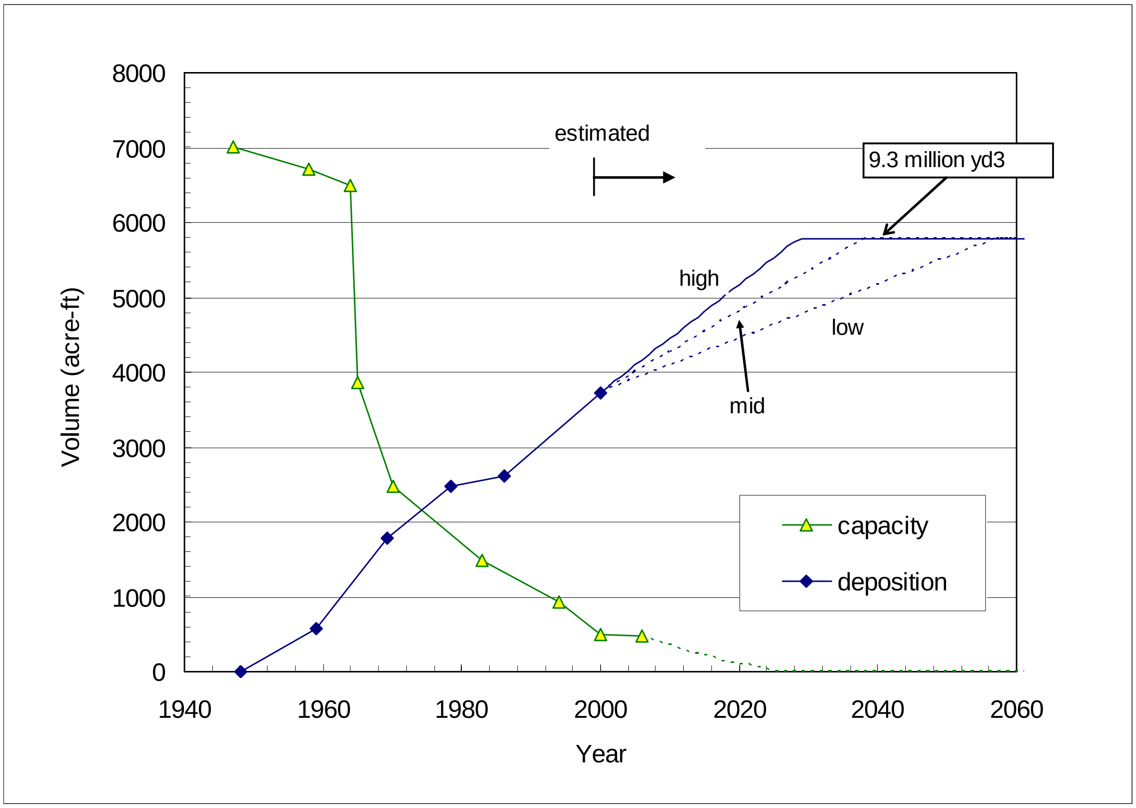


Figure 11.1. Historical and projected future deposition in Matilija reservoir.

The delta is continuing to progress into the reservoir and has become heavily vegetated. The first photo was taken in 1973 and shows a non-vegetated delta approximately 2,500 feet upstream from the dam (Figure 11.2). The next photo, taken in 1985, shows vegetation on the delta (Figure 11.3). The delta has progressed approximately 500 feet closer to the dam. The last photo, taken in 2001, shows the delta in the present location (Figure 11.4). The delta is heavily vegetated and has moved 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 11.1. 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 entered the upstream 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 11.2. 1973 Photograph of Matilija Delta. Note: The red circle is located at the same location in the following pictures of the Matilija Reservoir.

Figure 11.3. 1985 Photograph of Matilija Delta. Note: The red circle is located at the same location in each photo.

Figure 11.4. 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.

11.1.2. FUTURE WITH-PROJECT CONDITIONS IN MATILIJA RESERVOIR

The reservoir fines will be removed by slurry pipe and deposited on the downstream floodplains. A channel will be created through the reservoir deposits and the material removed from this excavation will be placed on the channel margins. An approximate rendering of the reservoir area immediately after dam removal is shown in Figure 11.5. Temporary revetments will be placed at the toes of the constructed channel to control the release of reservoir sediment. These revetments will be removed as sediment is eroded from the area and eventually all revetment should be removed from the reservoir area.

The simulated erosion volumes from the reservoir for the 50-yr hydrographs beginning in 1950, 1969, and 1991, are shown in Figure 11.6, Figure 11.7, and Figure 11.8, respectively. Most of the erosion from the reservoir will occur during the large flood events. For example, most of the reservoir erosion occurs the first year for the hydrograph beginning in 1969. This is because the largest storm on record occurs during the first year. For the 1950 hydrologic scenario, very little of the reservoir sediment is eroded during the first 10 years. This is because this is a very dry period with few storms.

The model predicts 1500 to 1700 ac-ft of sediment is eroded during the 50-year period following dam removal. This is approximately 60 % of the sediment remaining in the reservoir deposit after the reservoir fines are removed.

The current estimates of reservoir erosion rely on a one-dimensional sediment transport model (GSTAR-1D). Such a model does not calculate transverse variation of hydraulic or sediment transport variables and therefore cannot calculate the preferential erosion of one bank versus another. There will be additional work on the erosion of reservoir sediment and the results presented in this section are subject to revision pending that work.

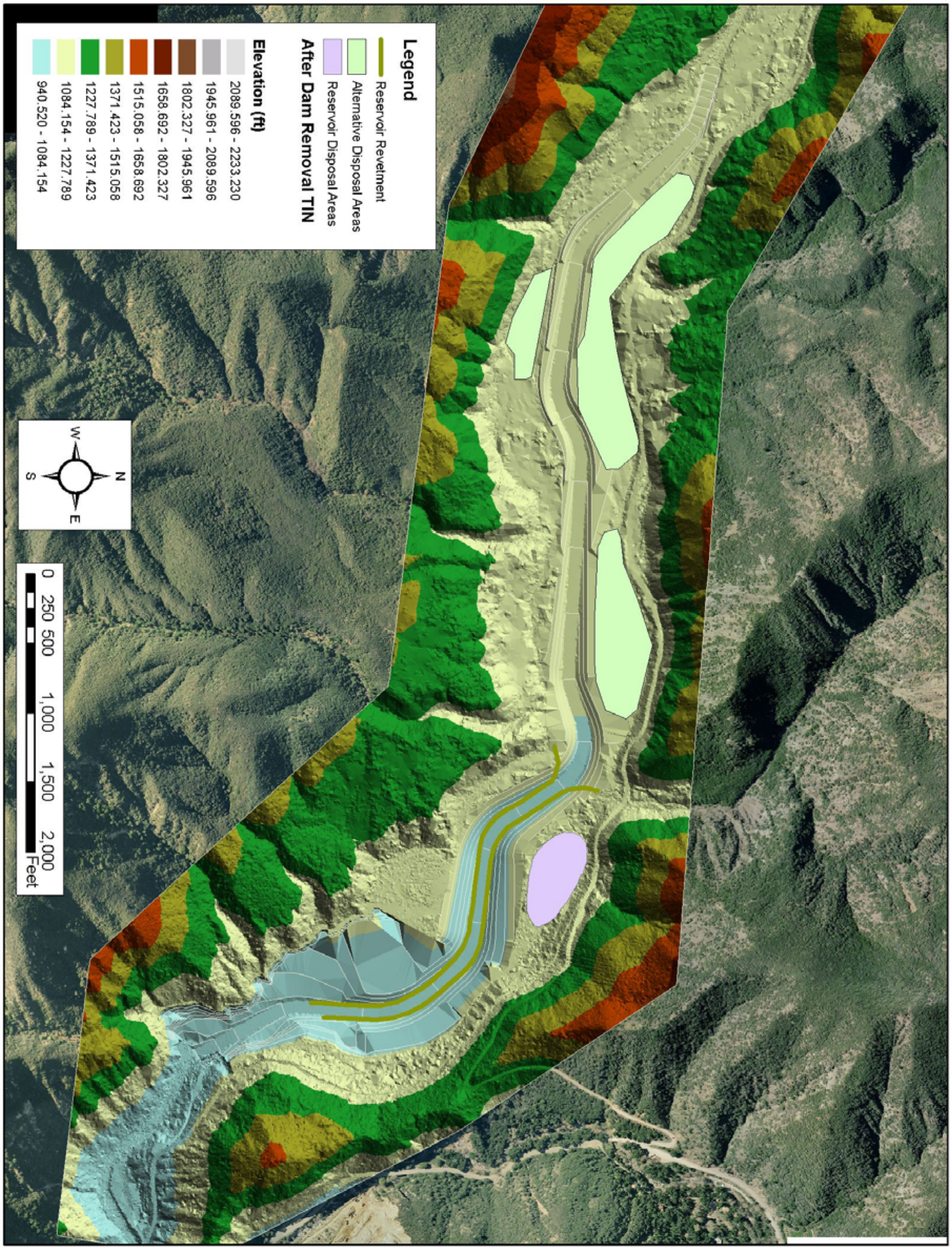


Figure 11.5. Approximate Rendering of Reservoir Area Immediately After Dam Removal.

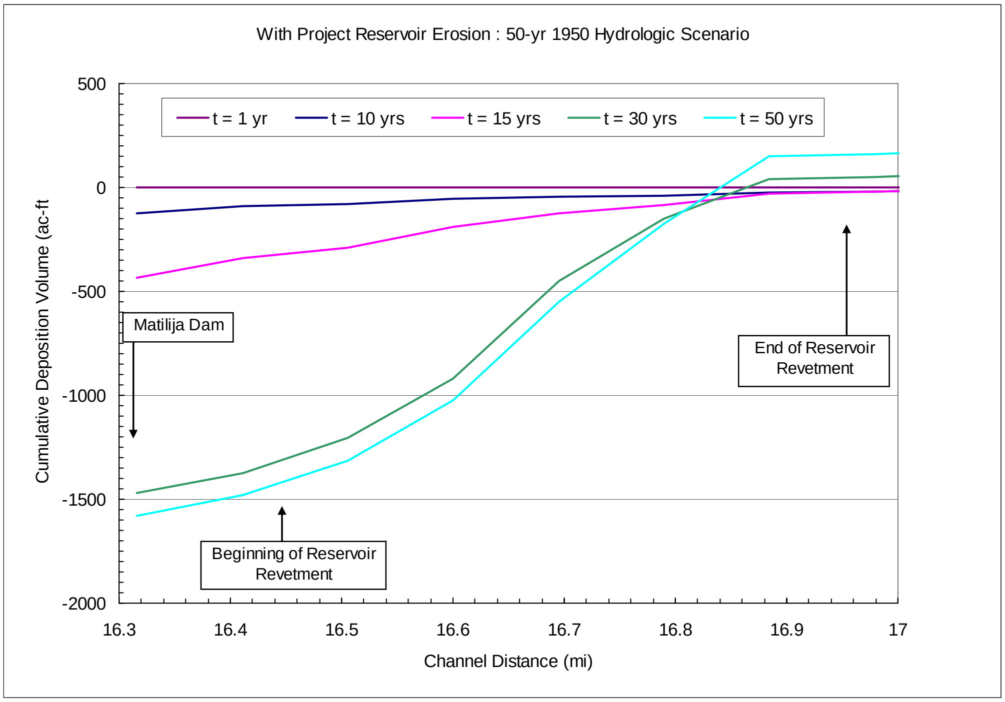


Figure 11.6. Erosion of Sediment from Matilija Reservoir under With-Project Conditions and 50-yr 1950 Hydrologic Scenario.

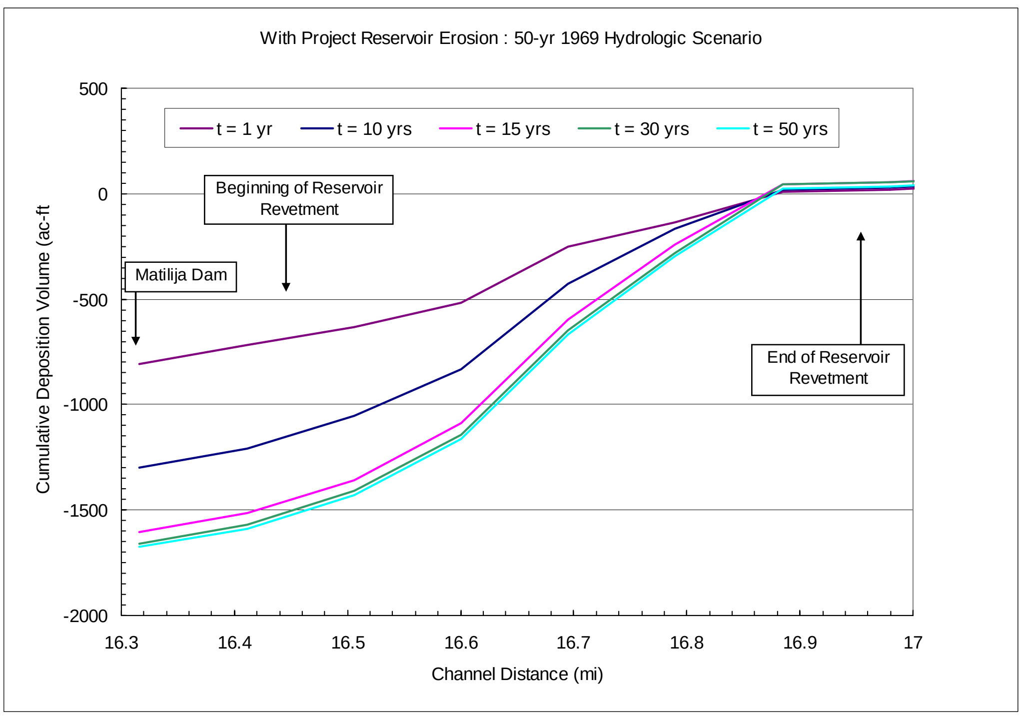


Figure 11.7. Erosion of Sediment from Matilija Reservoir under With-Project Conditions and 50-yr 1969 Hydrologic Scenario.

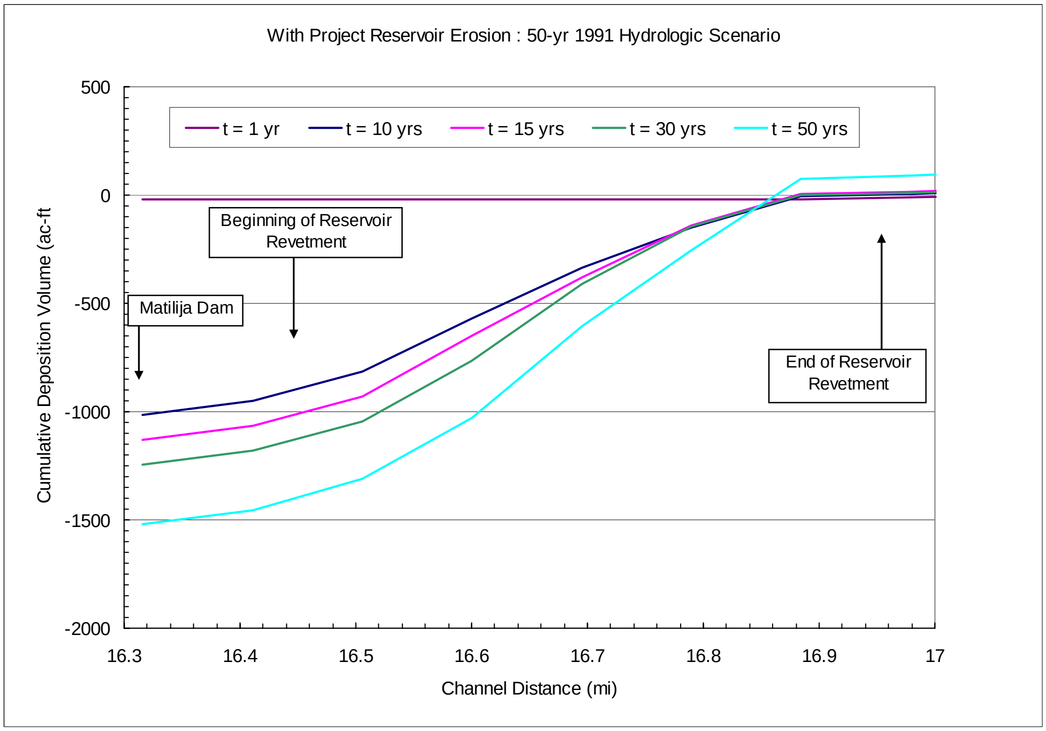


Figure 11.8. Erosion of Sediment from Matilija Reservoir under With-Project Conditions and 50-yr 1991 Hydrologic Scenario

11.2. Long Term Predictions of Erosion and Deposition in Ventura River

GSTAR-1D was used to simulate the future deposition and erosion in Matilija Creek and the Ventura River for the next 50 years, beginning in WY 2007. The three historical hydrographs described in Section 6.1.2 "Reconstructed hydrographs" were used to estimate future conditions. These will be referred to as the 1950, 1969, and 1991 hydrographs, indicating the year in which they were begun.

11.2.1. WITHOUT-PROJECT CONDITIONS

The volume of deposition and the elevation changes due to deposition are discussed below.

Volumes

The predicted cumulative deposition volume in Matilija Creek and Ventura River over the next 50 years is shown in Figure 11.9 for the 1991 50-yr hydrograph. The cumulative sediment volume was calculated by adding the volume of deposition for each cross section starting from the upstream end. The sum was begun at RM 20, almost 4 miles upstream of the dam. The sum was reset at Matilija Dam and at Robles Diversion, because these two structures cause significant deposition upstream of them. The trends are easier to see if the sum if reset to zero.

In the first 10 years for the 1991 50-yr hydrograph, the reservoir fills with almost 790 ac-ft of sediment (1.3 million yd^3). Over the following 40 years, 650 additional ac-ft (1.0 million yd^3) of sediment deposits in the reservoir. If it is assumed that the current reservoir holds 6.5 million yd^3, the total volume of sediment stored in the reservoir in fifty years will be 8.8 million yd^3, which is near the estimate in Table 11.1 (9.3 million yd^3).

The river downstream of Matilija Dam until Robles Diversion is essentially stable for the entire 50 years of simulation. This reach is controlled by large boulders, bedrock and a concrete weir at the stream gage.

Robles Diversion, however, creates an area of deposition upstream of it. CMWD has a program to excavate sediment in the basin behind Robles diversion after every significant flood. The simulation does not remove sediment from the basin and allows sediment to build up in the basin. About 60 ac-ft (100,000 yd^3) of sediment builds up behind the diversion. This is similar to the amount of sediment excavated after very large storms (see Table 1.5). CMWD has removed approximately 559,000 yd^3 (350 ac-ft) of material during the 42-year period from 1958 until 2000 (see Section 1.4.1). The model assumes that the diversion area fills with sediment and then allows sediment to start spilling over the top. In reality, CMWD excavates sediment from behind Robles Diversion and sediment does not pass over the top of the dam; however, some sediment is sluiced through the existing radial gages and some sediment is excavated from behind the dam and placed on its downstream side. It is likely that the sediment removal requirements at Robles Diversion will remain similar to the current conditions until the reservoir is completely full of sediment, which is predicted to be around 2038. After this time, the deposition at Robles Diversion may increase slightly.

Downstream of Robles Diversion the river was predicted to erode approximately 200 ac-ft of sediment between RM 14 until RM 10 over the next 50 years. The rate of erosion, however, is slower than in the previous years. From 1971 to 2001, for example, it was estimated that approximately 1000 ac-ft of sediment eroded in this reach (see Figure 6.21).

From RM 10 to 7.5, about 200 ac-ft of deposition occurs over 50 years. The river is relatively stable from RM 7.5 to 6, but erodes between RM 6 and 4. From RM 3 to the ocean, the model predicts deposition. However, this reach should remain relatively stable, as the model does not accurately simulate the estuary conditions.

The deposition figures for the 1950 and 1969 50-yr hydrographs are shown in Figure 11.10 and Figure 11.11. The 50-yr deposition predicted for these hydrographs are similar to the 1991 hydrograph (Figure 11.12). However, the rates at which the final 50-yr values are approached are different. The timing of the deposition volumes is dominated by the 1969 storm. For the 1969 hydrograph, it occurs in the first year and therefore, the change in the deposition volumes occurs quickly. For the 1950 50-yr hydrograph, the first 10 years is essentially dry and therefore little deposition occurs. In the next 10 years of the 1950 hydrograph, the change is large because the 1969 storm occurs between years 10 and 20.

The sediment loads passing each cross section over the 50-year simulations are shown in Figure 11.13. The sediment loads are affected by the structures in the river and the tributaries. The sediment load from RM 20 to 16.3 gradually decreases because of the deposition induced by Matilija Dam. There is an increase at RM 15.8 resulting from the supply of North Fork Matilija Creek. Robles Diversion reduces sediment loads at RM 14 and then downstream of Robles, the sediment loads increase again due to the erosion of material from the channel. At RM 8, the sediment loads increase substantially due to San Antonio Creek inputs.

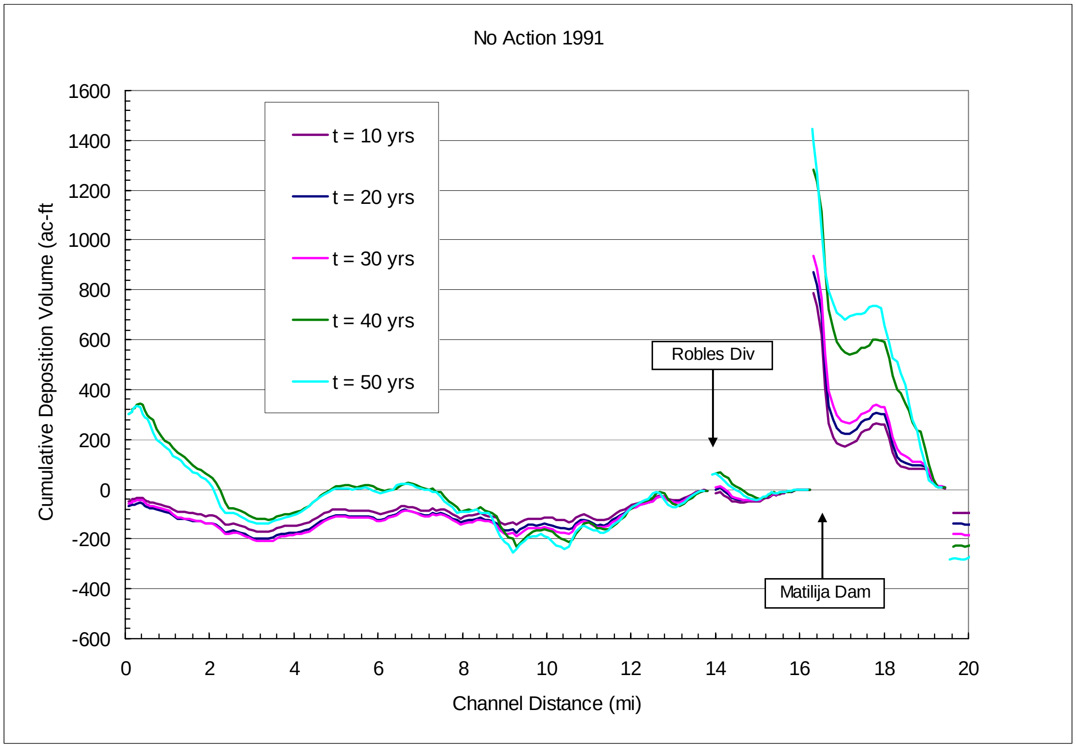


Figure 11.9. Cumulative Deposition for 1991 historical 50-yr hydrograph for Without Project Conditions.

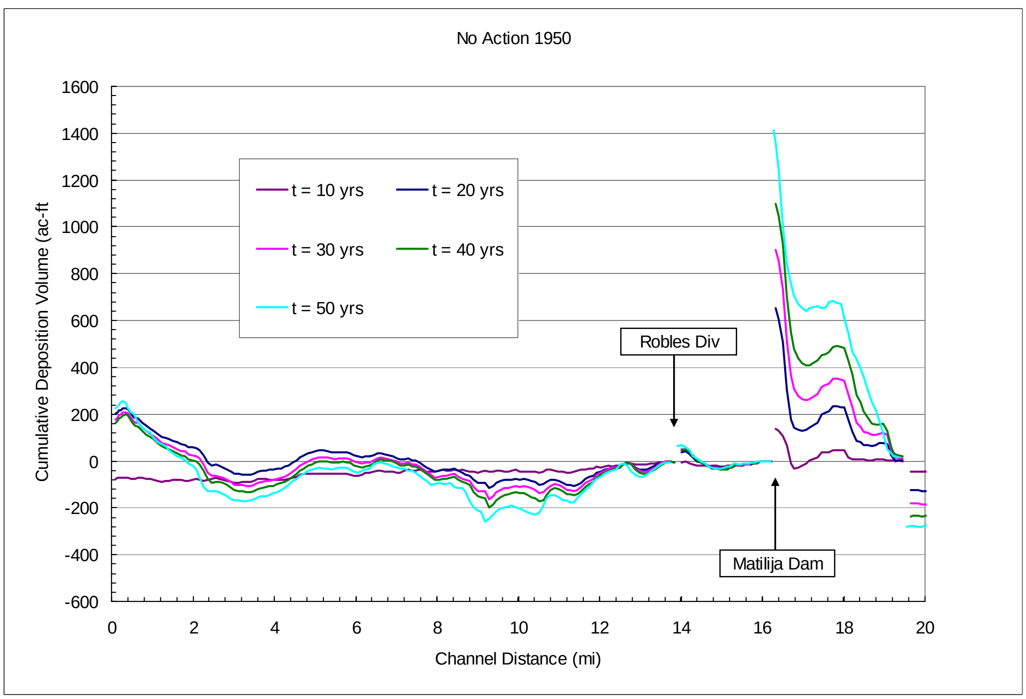


Figure 11.10. Cumulative Deposition for 1950 historical 50-yr hydrograph for Without Project Conditions.

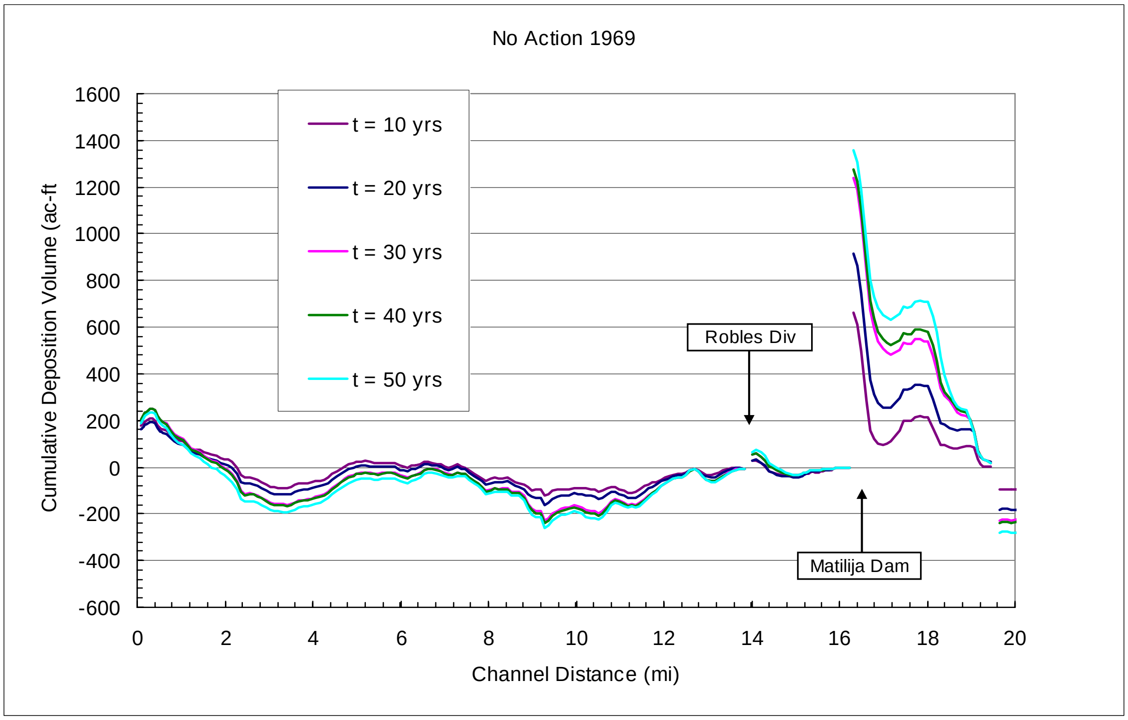


Figure 11.11. Cumulative Deposition for 1969 historical 50-yr hydrograph for Without Project Conditions.

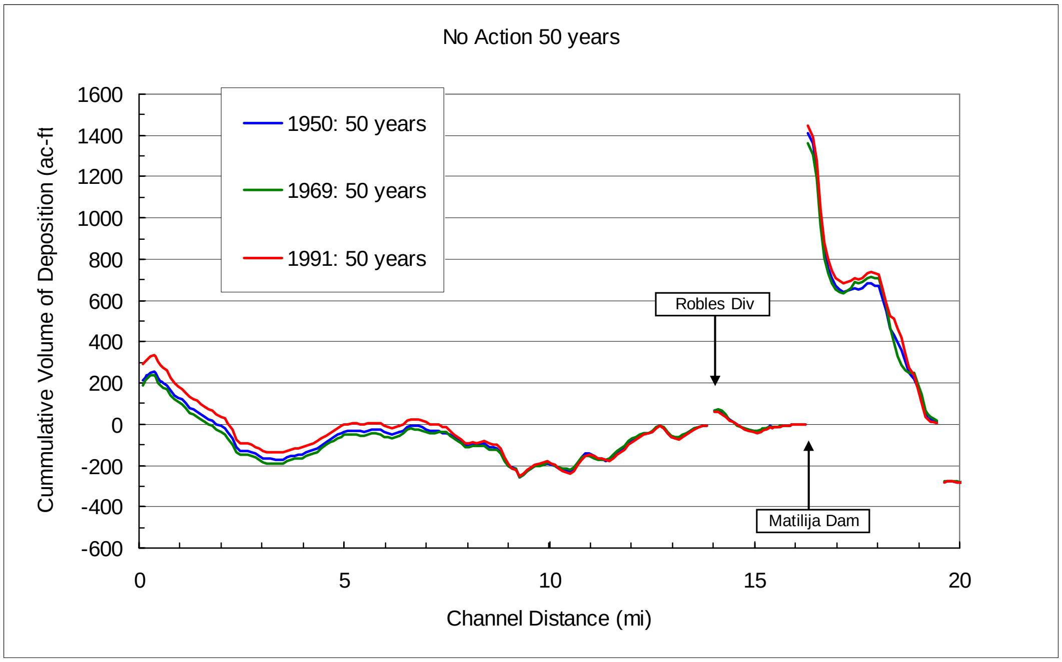


Figure 11.12. Cumulative Deposition in 50 years for Without-Project Conditions for different hydrographs.

Figure 11.13. Cumulative Sediment Loads at end of 50 year Simulation for Without-Project Conditions for different hydrographs.

Average Bed elevation Changes

The changes to the average bed elevation at the end of the 50-yr simulation for each hydrology are given in Figure 11.8. The 50-yr values are essentially the same between the three hydrographs. The average values of deposition by reach are given in Table 11.4 to Table 11.4 for various years into the simulation. The final results at year 50 are essentially the same between the three hydrographs, however, the results at year 1, 3 and 10 may be different depending upon the hydrology in the first 10 years.

Most all reaches show less than 1 foot of average deposition or erosion during the 50 year time period. Exceptions to this include reach 6a, but this is entirely due to the presence of Robles Diversion Dam creating a backwater in this reach. Another exception to this is in reach 6b, but the erosion predicted in this reach is likely over-predicted because bedrock controls were not considered.

<u>Reach 6b</u>: This reach is controlled by bedrock and large boulders. A fixed bottom was assumed through the majority of this reach and therefore no significant erosion is predicted. Reach 6b will likely be stable for the foreseeable future.

The Camino Cielo Reach (RM 15.4 to 15.2) will remain stable for the next 50 years. Large boulders and the Camino Cielo Bridge prevent this reach from eroding. It was assumed that this bridge remains at the same location and has the same hydraulic properties for the next 50 years.

Reach 6a: Reach 6a extending from RM 15 to 14.0 will experience deposition primarily caused by Robles Diversion Dam. The model does not simulate the excavation of sediment behind Robles and therefore sediment continues to build up throughout the simulation. There is approximately 5 to 7 feet of deposition in the reaches upstream of Robles.

Reach 5: Reach 5 extends from Robles Dam to Baldwin Road and this reach will continue to degrade under without-project conditions. However, the rate of degradation is expected to slow. The largest amount of erosion will occur just downstream of Robles Diversion with up to 3 feet of degradation. Near Baldwin Road, the river will be nearly stable. The average bed change throughout the entire reach over the next 50 years is -0.9 feet.

Reach 4: Reach 4 extends from Baldwin Rd to the confluence with San Antonio Creek (RM 11.11 to 7.86). The river remains relatively stable at Baldwin Rd, but from RM 10.9 to 10.4, the river shows up to 3 feet of erosion. The erosion predicted by the model could be due to the fact the the 2005 flood straightened the river at this location and the straigtening will cause the river to incise slightly at this location.

Live Oak Drain enters the Ventura River from the west side just upstream of Live Oak Acres at approximately RM 10.15 (Figure 4.11). Live Oak Drain has a bottom elevation of approximately 457.5 feet where it crosses under Burnham Road. It was designed to carry the 100-yr flood of approximately 890 cfs at a flow depth of approximately 5 feet at a slope of 0.0009. However, the designed assumed a elevation of 456.5 feet at the drain exit. Since that time, the drain exit has aggraded to 458 feet. Therefore, there is now essentially no slope to the drain from Burnham Road to the Ventura River, a distance of approximately 860 feet. The 100-yr flood elevation of the Ventura River at this location is approximately 462 feet. The levee elevation along the drain is approximately 469.5 feet. Because there is no slope to the drain, it is likely that it will continue to experience aggradation.

There is some deposition expected just upstream of the Live Oak Levee because of the constriction caused by the levee will create some backwater upstream of the levee (RM 10.1). The deposition is estimated to be 2 feet or less. This will tend to accelerate deposition in the Live Oak Drain.

The County placed groins in 2005 starting approximately 1200 feet upstream of the bridge in the river on the East Bank of the Ventura River upstream of Santa Ana Bridge to protect the toe of the high bank. There was concern that this bank protection would be undermined, but this is unlikely. The simulated erosion is mostly downstream of the groin area. It is suggested that no additional measures be taken to prevent erosion of the groins.

In the immediately vicinity of Santa Ana Bridge and for approximately 1000 feet upstream of the bridge, the model predicts the average bed elevation will decrease up to 5 feet. However, this erosion is not primarily vertical, but it is a widening of the cross section and a levelling off of the main channel islands (see Figure 11.22). This is very near to where the County has seen bank erosion in the past (Figure 4.12). The levee material along the West Bank is not of sufficient size to prevent erosion and the County will likely have to continue to repair the bank protection near where the erosion occurred in 2005.

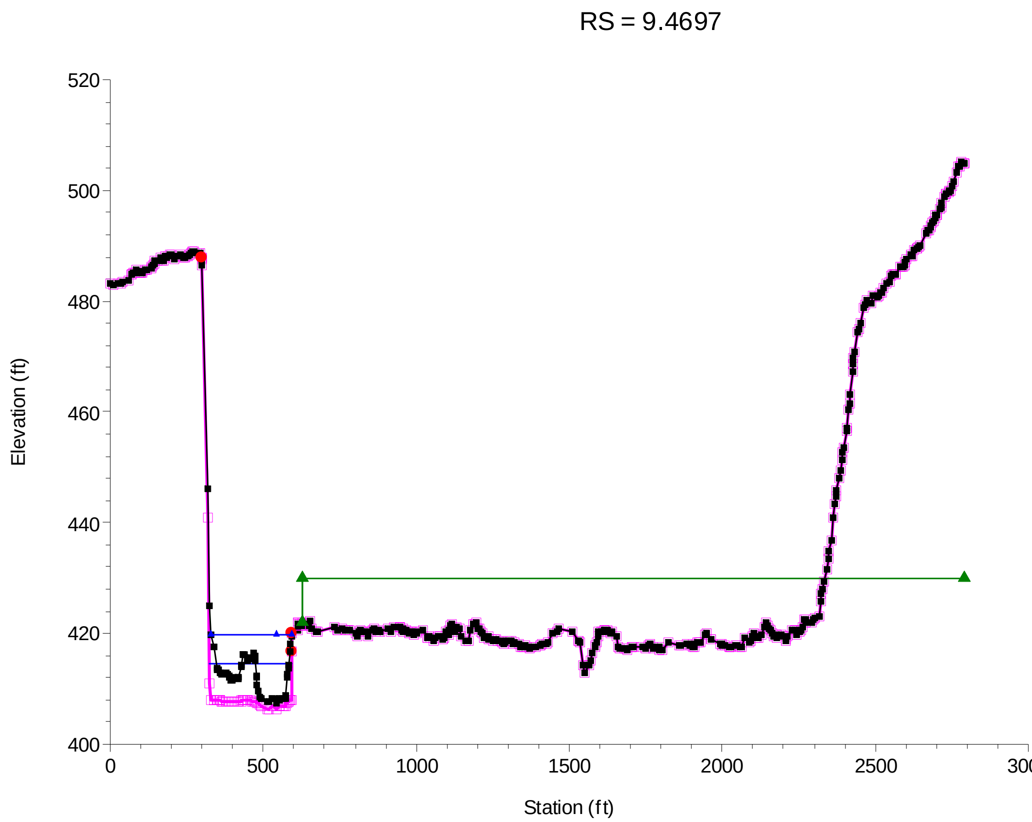


Figure 11.14. Bed Elevation Changes upstream of Santa Ana Bridge under Without-Project Conditions.

<u>Reach 3</u>: Reach 3 extends from San Antonio Creek to Casitas Vista Road Bridge. The upper portion of this reach from RM 7.8 to 7.5 along the Casitas Springs levee will experience 2 to 3 feet of deposition. The lower portion of this reach from RM 7.5 to 6.6 will remain relatively stable. There is up to 2 feet of erosion predicted in the reach from RM 6.5 to 6.1. At the Casitas Vista Road Bridge, some deposition is likely and expected to be around 2 to 3 feet.

The Casitas Springs reach may see some deposition, especially along the upper end of the Casitas Levee from RM 7.8 to 7.5 with 2 to 3 feet of deposition possible over the next 50 years. The 20-yr flood is already within 2 feet of the levee crest elevation along this reach hence 2 or 3 feet of deposition would significantly affect the flood risk of this community. There is a critical need to improve the entire Casitas Levee under the Without Project Conditions.

Fresno Drain enters the Ventura River at approximately RM 6.8. The river should remain relatively stable in this reach with perhaps slight aggradation of approximately 1 foot. However, even a riverbed elevation rise of 1 foot would cause the 20-yr flood and perhaps the 10-yr flood to exceed the Casitas Levee elevation from RM 7.1 to RM 6.8.

The reach adjacent to the Coyote Creek Levee will likely experience erosion of approximately 1 to 2 feet. However, this may not reduce flood impacts significantly because the greatest danger to this community is erosion of the protective levee. The 2005 flood pushed the river next to the levee and started to erode its upstream end. Local residences placed concrete blocks at the location of this erosion, but these would likely prove ineffective against storms that are larger than the 10-yr or 20-yr storm.

There is 2 feet of erosion expected upstream of Casitas Vista Road Bridge near Foster Park. At the Casitas Vista Road Bridge, some deposition is likely and expected to be around 2 feet. Several rock groin structures were installed in 2005 in the Ventura on the east side of river to protect Foster Park from erosion. These groins may protect the park from future erosion, but we did not perform an analysis to determine if the rock size was sufficient to stabilize the bank.

Reach 2: Reach 2 extends from Casitas Vista Road Bridge to Main St Bridge. This reach is expected to continue to degrade from RM 5 to RM 3. However, some sections of this reach may be controlled by bed rock that will prevent future degradation. The OVSD facility at RM 5 is currently inundated by the 500-yr flood and the 100-yr flood is within a few inches of inundating the pond elevation and within 2 feet of inundating their buildings. This reach is expected to degrade or remain stable in the next 50 years. There is bedrock exposed in many parts of this reach that will likely prevent any significant erosion (Figure 11.15).

The model results below RM 3 show that there is deposition, but the deposition is likely overpredicted. When simulating the period 1971 to 2001, the model predicted that below RM 2 there was significant deposition, but the field measurements show that this reach was stable or eroded during this period. Therefore, the model results below RM 3 may show too much deposition.

Figure 11.15. Exposed Bedrock along West side of Ventura River near OVSD Facility at RM 5.

Reach 1: Reach 1 consists of the estuary. The model results show slight deposition in the upper portion of this reach and slight erosion in the lower part of this reach. Overall, this reach will remain relatively stable. However, the size of the sand bar at the river mouth is very sensitive to the previous storm history. The large flucuations in the coastlin are shown in Figure 5.38. This variation is expected to continue and the sand bar will grow as storms occur in the watershed and decay as ocean waves erode it.

Average Values by Reach

The average values of deposition by reach are given in Table 11.2 to Table 11.4 for various years into the simulation. The final results at year 50 are essentially the same between the three hydrographs, however, the results at year 1, 3 and 10 may be different depending upon the hydrology in the first 10 years.

Most all reaches show less than 1 foot of average deposition or erosion during the 50 year time period. Exceptions to this include reach 6a, where large amounts of deposition occur upstream of Robles Diversion and downstream of Matilija Canyon.

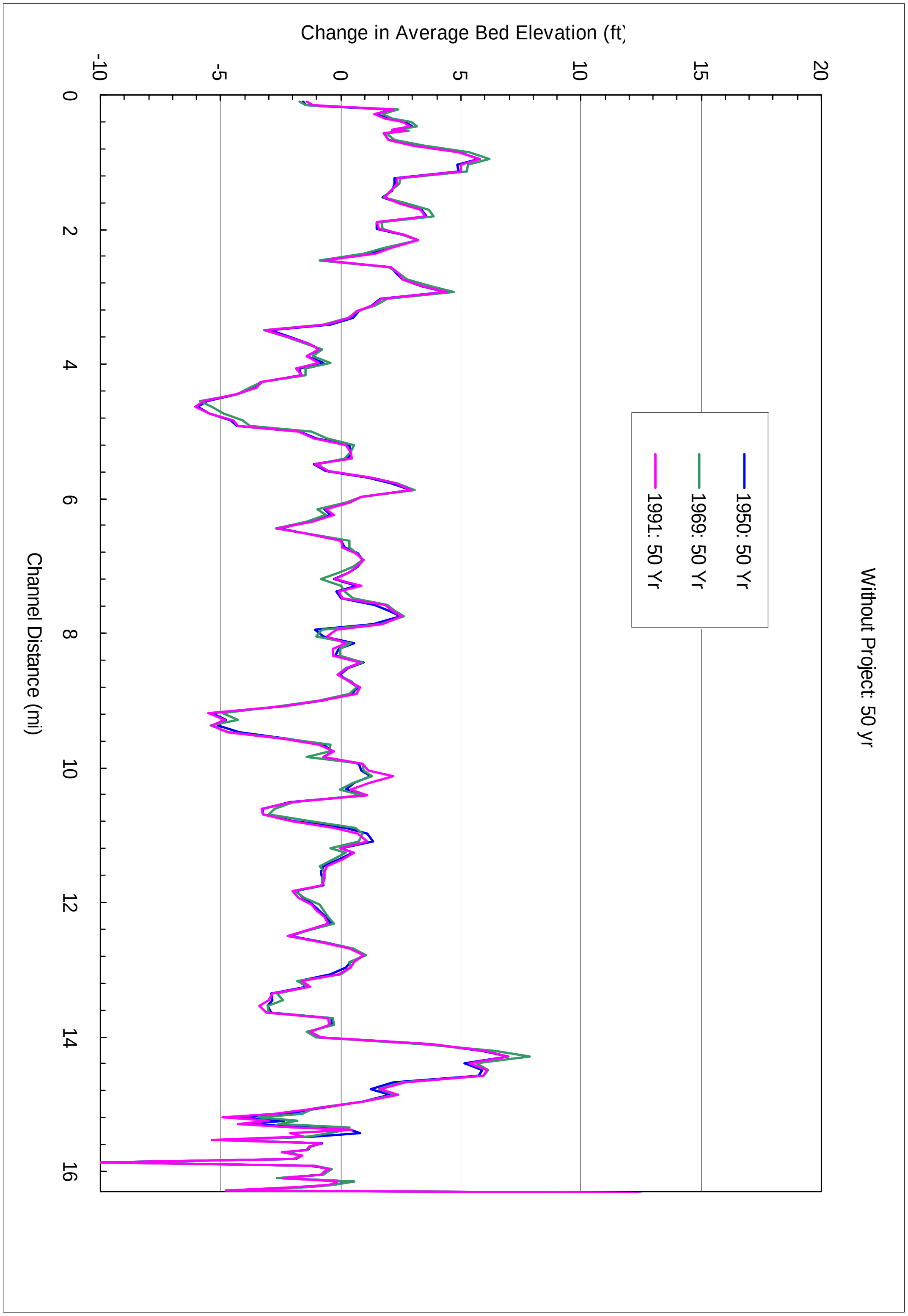


Figure 11.16. Average bed elevation changes at the end of the 50-yr simulations for the 1950, 1969, and 1991 50-yr hydrographs for Without Project Conditions.

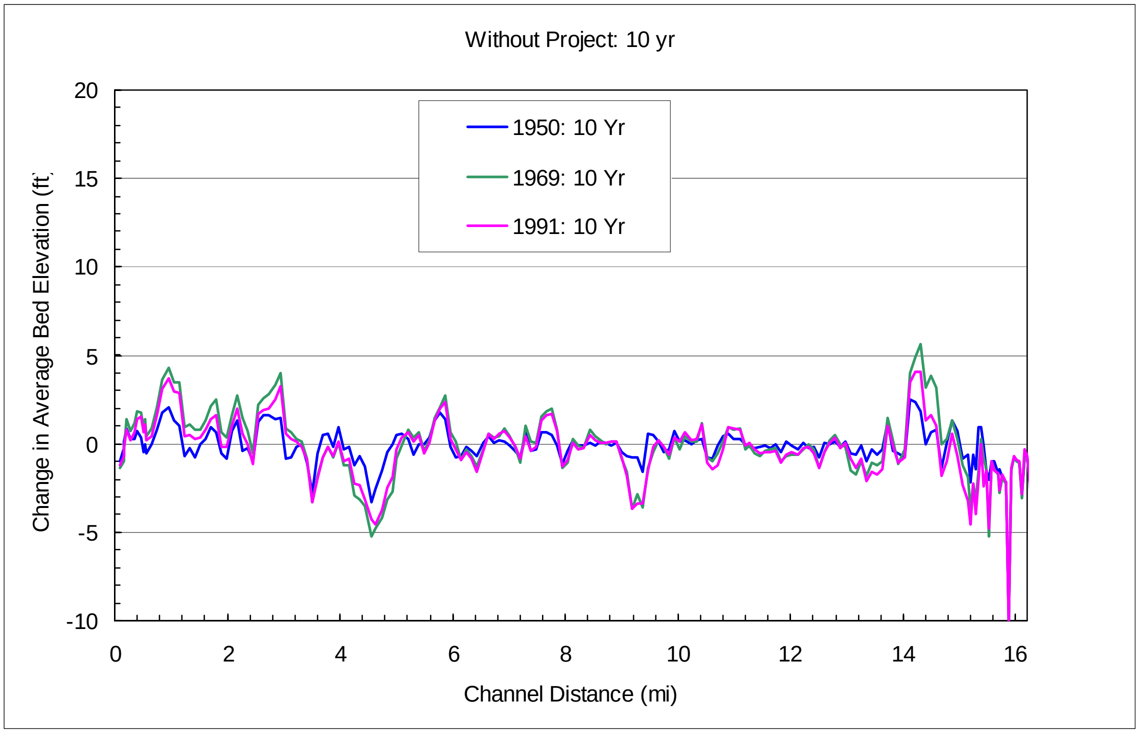


Figure 11.17. Average bed elevation changes at the end of the 10-yr simulations for the 1950, 1969, and 1991 50-yr hydrographs for Without Project Conditions.

Table 11.2. Table of Reach Average Deposition for 1950 50-yr Hydrograph for Without-Project Conditions.

Average Deposition (ft)	**Year**				
Location	**1**	**3**	**10**	**20**	**50**
Reach 6b	0.0	0.0	-1.4	-1.9	-1.8
Reach 6a	0.0	0.0	0.6	2.5	3.5
Reach 5	0.0	0.0	-0.2	-0.5	-0.8
Reach 4	0.0	0.0	-0.1	-0.4	-0.8
Reach 3	0.0	0.0	-0.1	0.3	0.2
Reach 2	0.0	0.0	0.0	0.3	0.5
Reach 1	0.0	0.0	0.1	0.4	0.9

Table 11.3. Table of Reach Average Deposition for 1969 50-yr Hydrograph for Without-Project Conditions.

Average Deposition (ft)	Year				
Location	**1**	**3**	**10**	**20**	**50**
Reach 6b	-1.7	-1.8	-2.1	-2.0	-1.7
Reach 6a	1.8	1.8	2.4	2.6	3.7
Reach 5	-0.3	-0.3	-0.5	-0.6	-0.8
Reach 4	-0.1	-0.2	-0.4	-0.5	-0.8
Reach 3	0.3	0.3	0.2	0.2	0.2
Reach 2	0.4	0.4	0.3	0.4	0.7
Reach 1	0.5	0.5	0.4	0.6	1.0

Table 11.4. Table of Reach Average Deposition for 1991 50-yr Hydrograph for Without-Project Conditions.

Average Deposition (ft)	Year				
Location	**1**	**3**	**10**	**20**	**50**
Reach 6b	-1.1	-1.8	-2.3	-2.0	-2.1
Reach 6a	0.2	0.3	1.1	1.6	3.6
Reach 5	0.0	-0.2	-0.5	-0.6	-0.9
Reach 4	0.0	-0.1	-0.5	-0.5	-0.8
Reach 3	0.0	-0.1	0.1	0.0	0.2
Reach 2	0.1	0.1	0.1	0.2	0.5
Reach 1	0.2	0.1	0.3	0.4	0.9

11.2.2. WITH-PROJECT CONDITIONS

The volume of deposition and the elevation changes due to deposition are discussed below. The simulation results presented in this section assumed that the erosion protection in the reservoir was designed to be at the 2-yr flood elevation.

Volumes

The predicted cumulative deposition volume in Matilija Creek and Ventura River over the next 50 years for the 1991 hydrograph is shown in Figure 11.18. Most all of the erosion in Matilija Reservoir occurs within the first 10 years. Several storms exceed the 2-yr flood protection and erode the relatively fine reservoir sediment quickly. Approximately 1500 ac-ft (2.4 million yd^3) of sediment is eroded from the reservoir. This is approximately 60% of the sediment remaining in the reservoir region after the reservoir fines are removed by hydraulic means.

The reach immediately downstream of the dam called Matilija Canyon is relatively steep and only a small amount of deposition occurs between the dam and RM 15.5. Downstream of the Canyon, the river becomes wider and the river slope decreases. In addition, Robles Diversion Dam causes deposition in the channel immediately upstream. CMWD has a program to excavate sediment in the basin behind Robles diversion after every significant flood. The simulation does not remove sediment from the basin and allows sediment to build up in the basin. In the reach between RM 15.2 and Robles Diversion, approximately 400 ac-ft of sediment is deposited over the course of 50 years for an average of (8 ac-ft/yr). As a comparison CMWD has removed approximately 350 ac-ft (559,000 yd^3) of material during the 42-year period from 1958 until 2000 for an average of 7 ac-ft/yr (see Section 1.4.1). Based upon the deposition predicted in this reach, it is likely that the amount of material excavated at Robles Diversion by CMWD to maintain operations will increase.

Significant deposition also occurs from Robles Diversion to RM 12.5. There is approximately 400 to 450 ac-ft deposited in this reach over 50 years. The reach from RM 12.5 until RM 10.8 is relatively stable with little deposition or erosion for the entire simulation period. There is a continual deposition trend from RM 10.2 to RM 6 during the 50-year simulation. Approximately 500 ac-ft of sediment is deposited in this reach. If it is assumed that the volume is deposited uniformly in the reach and the average width is 800 feet, the sediment deposition thickness is 1.3 feet.

The deposition volumes resulting from the 1950 hydrograph and 1969 hydrograph are shown in Figure 11.19 and Figure 11.20, respectively. The final 50-yr volumes are similar between all hydrographs (Figure 11.21). However, the results at 10-yr can be significantly different between the hydrographs. For example, at year 10 of the 1950 hydrograph little deposition or erosion has taken place through the reach.

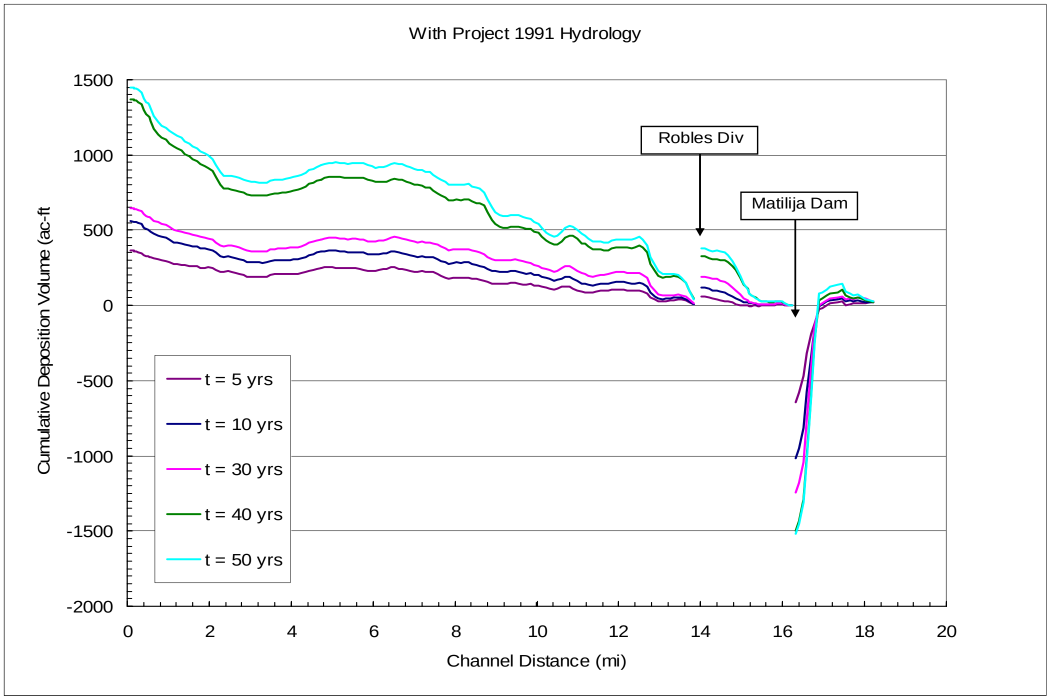


Figure 11.18. Cumulative Deposition for 1991 historical 50-yr hydrograph for With- Project Conditions.

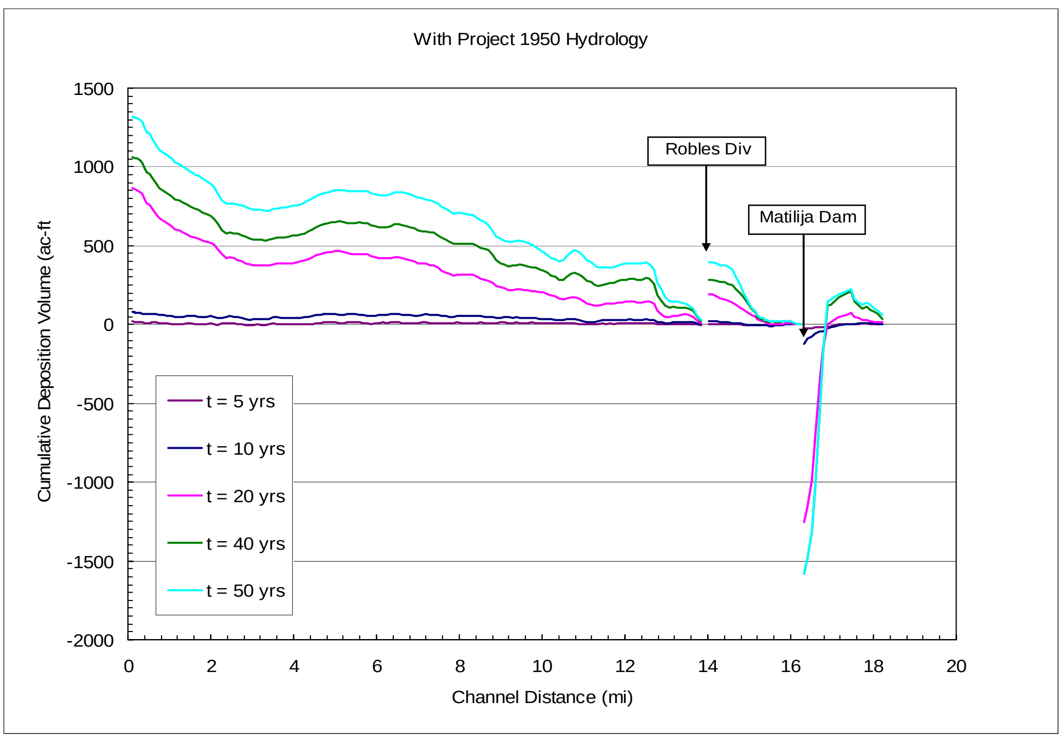


Figure 11.19. Cumulative Deposition for 1950 historical 50-yr hydrograph for With- Project Conditions.

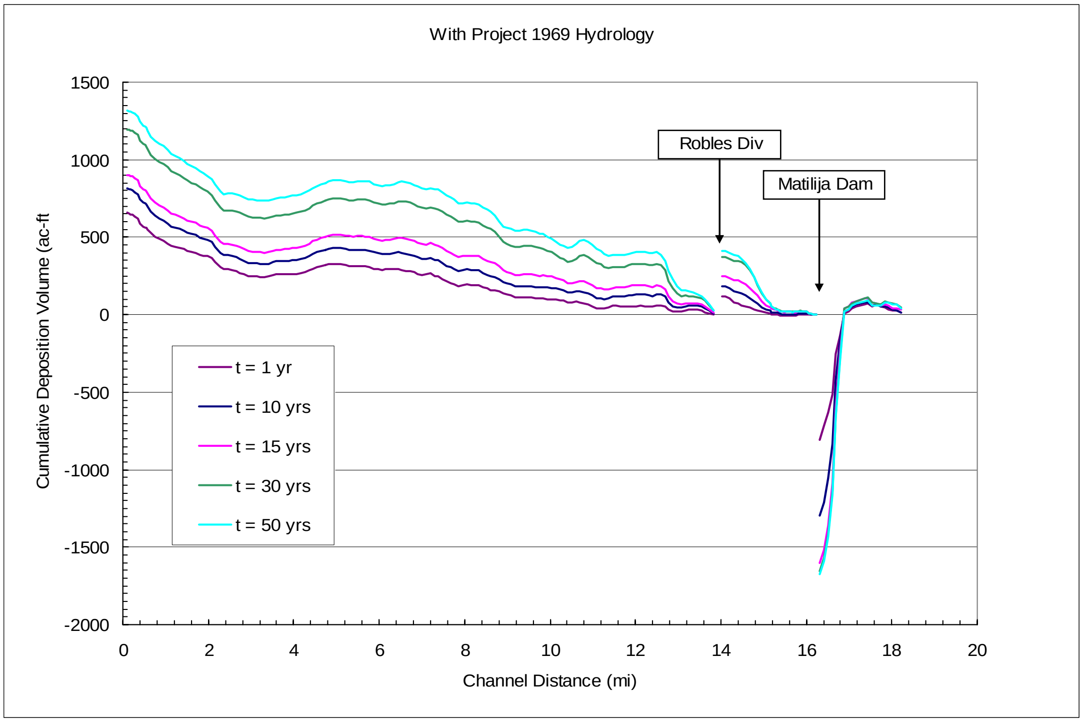


Figure 11.20. Cumulative Deposition for 1969 historical 50-yr hydrograph for With- Project Conditions.

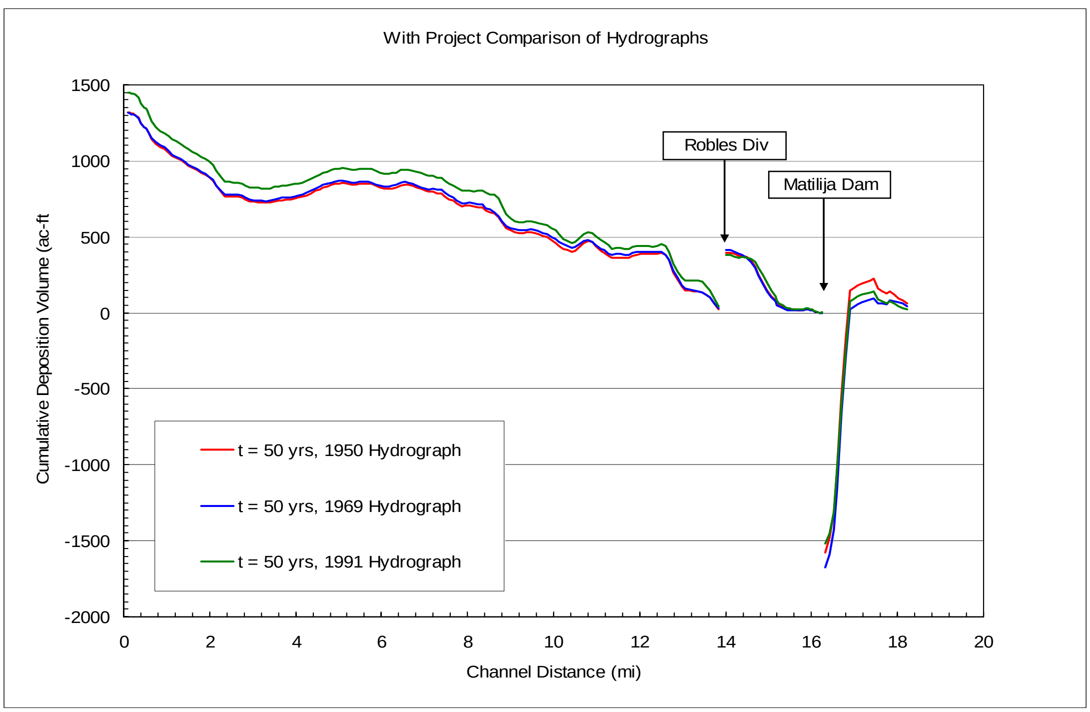


Figure 11.21. Cumulative Deposition in 50 years for With-Project Conditions for different hydrographs.

Average Bed elevation Changes

The changes to the average bed elevation at the end of the 50-yr simulation are given in Figure 11.24. The 50-yr values are essentially the same between the three hydrographs.

<u>Reach 6b</u>: The section of Reach 6b extending from the dam to RM 15.7 shows relative stability erosion. This reach is controlled by bed rock and large boulders, and most of the sediment eroded from the reservoir will pass through this reach. There is the possibility, however, that there will be temporary storage of sediment at this location.

<u>Reach 6a</u>: Reach 6a extending from RM 15 to 14.0 will experience deposition caused by the decrease in river slope in this area and Robles Diversion Dam. The model does not simulate the excavation of sediment behind Robles and therefore sediment continues to build up throughout the simulation. Up to 10 feet of deposition occurs in this reach.

<u>Reach 5</u>: Reach 5 extends from Robles Dam to Baldwin Road. Significant deposition occurs in the section between Robles Diversion and RM 12.5. This reach has experienced erosion during the period from 1970 to 2000, and it is likely that the re-supply of sediment will fill the eroded areas.

The reach from RM 12.5 to Baldwin Road is expected to remain relatively stable.

<u>Reach 4</u>: Reach 4 extends from Baldwin Rd to the confluende with San Antonio Creek (RM 11.11 to 7.86). The river remains relatively stable at Baldwin Rd, but from RM 10.9 to 10.4, the river shows up to 4 feet of erosion. The erosion predicted by the model could be due to the fact the the 2005 flood straightened the river at this location and the straigten will cause the river to incise slightly at this location.

The model predicts 1 to 3 feet of deposition for the reach just upstream of Live Oak Levee and along most of Live Oak Levee from RM 10.4 to 9.5. The deposition in this reach will increase flood elevations along the Live Oak levee and appropriate levee improvments should be designed.

Live Oak Drain enters the Ventura River from the west side just upstream of Live Oak Acres at approximately RM 10.15 (Figure 4.11). Live Oak Drain has a bottom elevation of approximately 457.5 feet where it crosses under Burnham Road. It was designed to carry the 100-yr flood of approximately 890 cfs at a flow depth of approximately 5 feet at a slope of 0.0009. However, the designed assumed a elevation of 456.5 feet at the drain exit. Since that time, the drain exit has aggraded to 458 feet. Therefore, there is now essentially no slope to the drain from Burnham Road to the Ventura River, a distance of approixmatley 860 feet. The 100-yr flood elevation of the Ventura River at this location is approximately 462 feet. The levee elevation along the drain is approximately 469.5 feet. Because there is no slope to the drain, it is likely that it will continue to experience aggradation. Furthermore, there is 1 to 2 of deposition expected near the Live Oak Drain under future with-project conditions. This will tend to accelerate the deposition in the Drain.

The County placed groins in 2005 starting approximately 1200 feet upstream of the bridge in the river on the East Bank of the Ventura River upstream of Santa Ana Bridge to protect the toe of the high bank. There was concern that this bank protection would be undermined, but this is unlikely. The simulated erosion is modely downstream of the groin area. It is suggested that no additional measures be taken to prevent erosion. It is more likely that the reach where goins are located will aggrade.

In the immediately vicinity of Santa Ana Bridge and for approximately 1000 feet upstream of the bridge, the model predicts the average bed elevation will decrease. However, this erosion is not primarily vertical, but it is a widening of the cross section and a levelling off of the main channel islands (see Figure 11.22). This is very near to where the County has seen bank erosion in the past (Figure 4.12). Therefore, it is expected that the bank erosion that occurred in the past will continue in the future. The levee material along the West Bank is not of sufficient size to prevent erosion and the County will likely have to continue to repair the bank protection near where the erosion occurred in 2005.

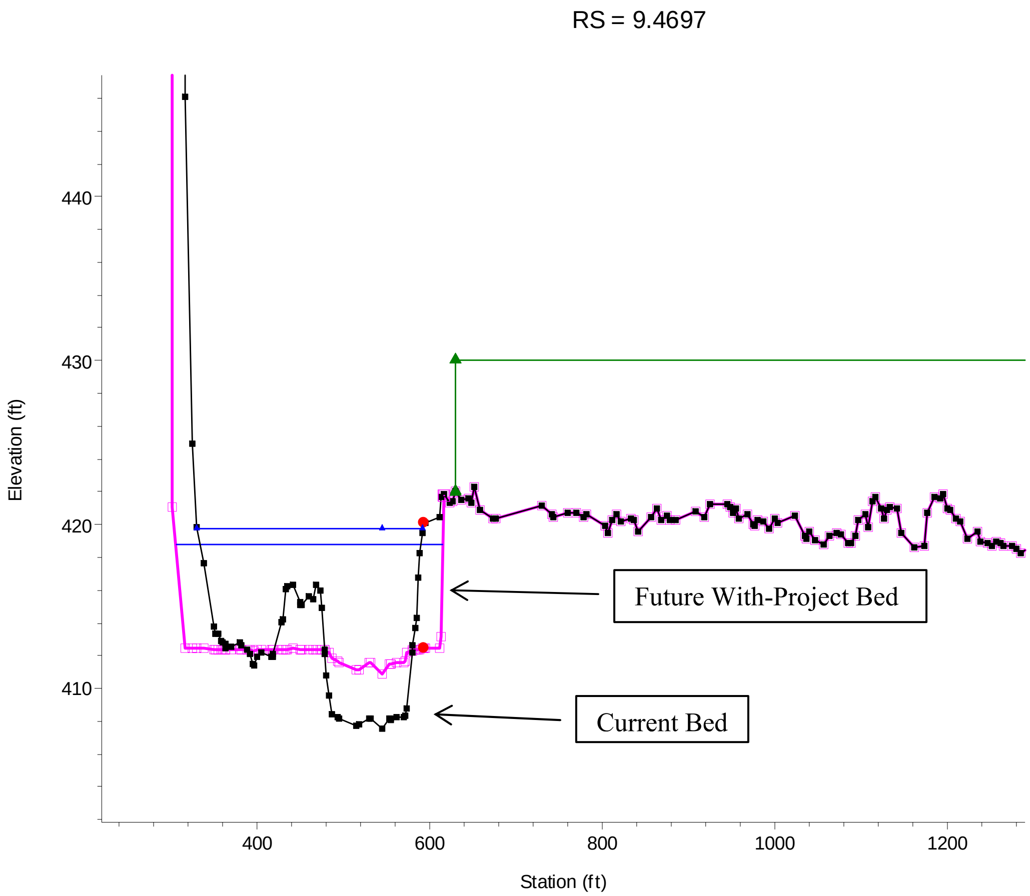


Figure 11.22. Bed Elevation Changes upstream of Santa Ana Bridge under With-Project Conditions.

The East side of the Santa Ana Bridge will be widened by approximatley 60 feet under project conditions. The proposed new bank line is shown in Figure 11.23. The figure also shows the historical channel migration zone. This is the zone in which the main channel has been located since 1947. The location of the groins constructed in 2005 is also shown in the figure. The widening of the bridge will tend to reduce velocities in the area under the bridge and and downstream of the bridge. It will also lower water surfaces upstream of the bridge. The effect of the bridge on channel hydraulics is strongest downstream of RM 9.47. The groins are located from RM 9.47 to about RM 9.7, and therefore the bridge reconstruction should have little effect on their performance.

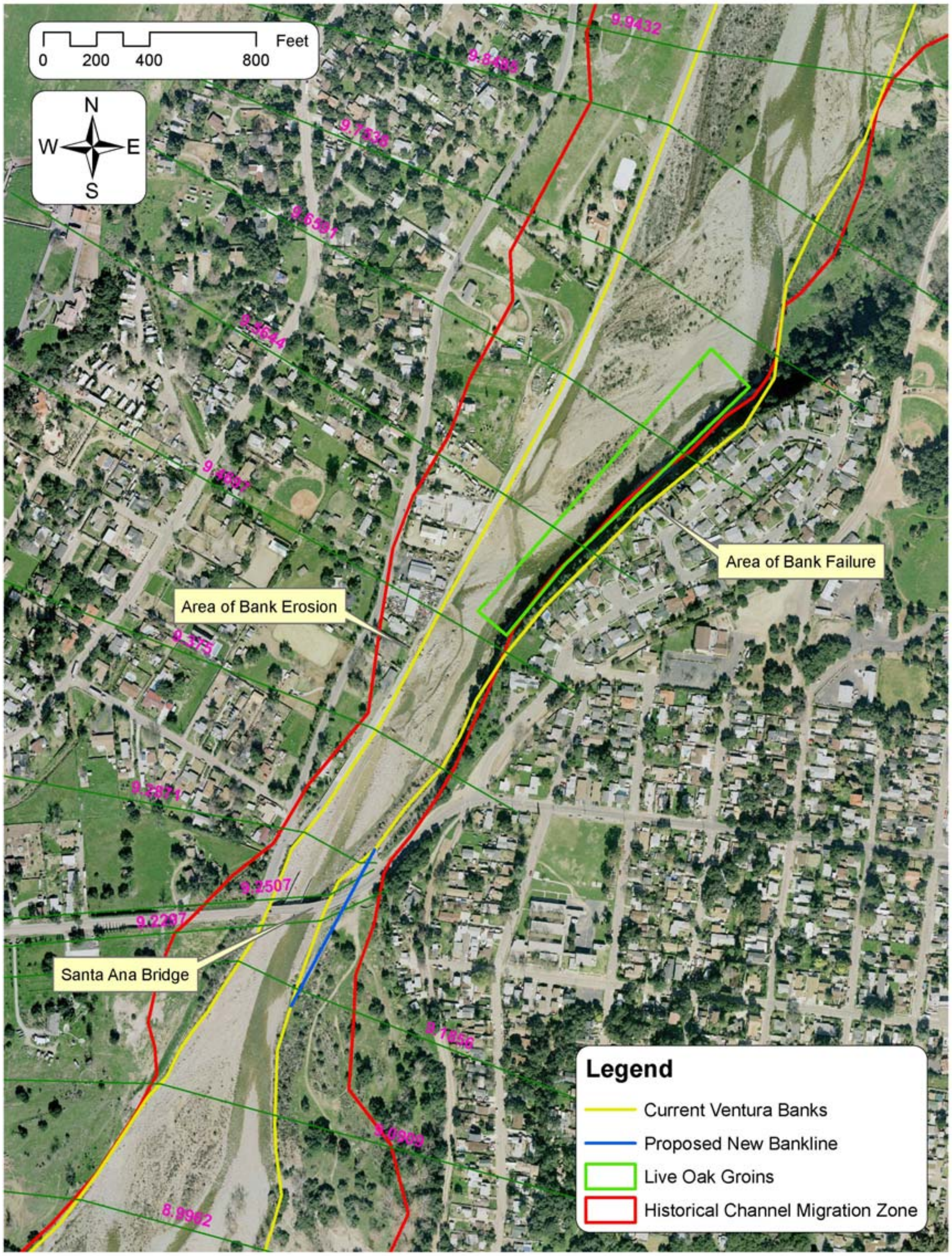


Figure 11.23. Proposed New Bank Link in Vicinity of Santa Ana Bridge. Groins Constructed in 2005 are also Shown.

Reach 3: Reach 3 extends from San Antonio Creek to Casitas Vista Road Bridge. The upper portion of this reach from RM 7.8 to 7 along the Casitas Springs levee will experience 2 to 3 feet of deposition. The Ventura river just upstream and downstream of the Casitas Vista Road Bridge will also likely experience deposition of 2 to 3 feet.

Reach 2: Reach 2 extends from Casitas Vista Road Bridge to Main St Bridge. Overall, this reach is expected to continue to degrade. The model results below RM 3 show that there is deposition, but the deposition is likely overpredicted. When simulating the period 1971 to 2001, the model predicted that below RM 2 there was significant deposition, but the field measurements show that this reach was stable or eroded during this period. Therefore, the model results below RM 3 may show too much deposition.

Reach 1: Reach 1 consists of the estuary. The model results show slight deposition in the upper portion of this reach and slight erosion in the lower part of this reach. Overall, this reach will remain relatively stable. However, the size of the sand bar at the river mouth is very sensitive to the previous storm history. The large flucuations in the coastline are shown in Figure 5.38. This variation is expected to continue and the sand bar will grow as storms occur in the watershed and decay as ocean waves erode it.

Average Values by Reach

The average values of deposition by reach are given in Table 11.5 to Table 11.7 for various years into the simulation. The final results at year 50 are essentially the same between the three hydrographs, however, the results at year 1, 3 and 10 may be different depending upon the hydrology in the first 10 years.

Most all reaches show less than 1 foot of average deposition or erosion during the 50 year time period. Exceptions to this include reach 6a, where large amounts of deposition occur upstream of Robles Diversion and downstream of Matilija Canyon.

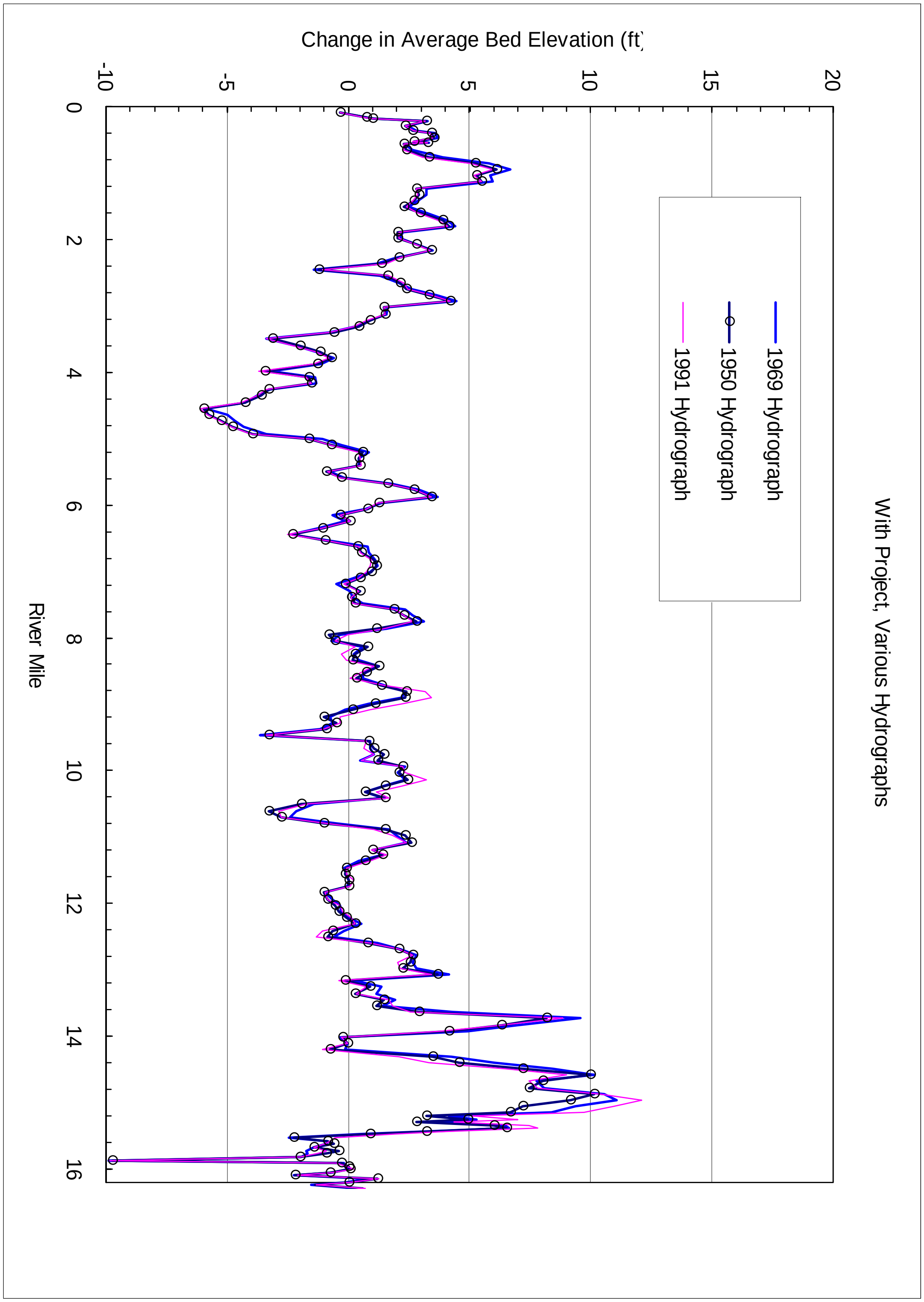


Figure 11.24. Average bed elevation changes at the end of the 50-yr simulations for the 1950, 1969, and 1991 50-yr hydrographs under With-Project Conditions.

Table 11.5. Table of reach average deposition for 1950 50-yr hydrograph under With-Project Conditions.

Average Deposition (ft)	**Year**				
Location	**1**	**3**	**10**	**20**	**50**
Reach 6b	-0.5	-0.5	-0.1	0.5	0.4
Reach 6a	2.6	2.6	3.4	4.2	3.6
Reach 5	0.1	0.1	0.3	0.6	0.5
Reach 4	-0.1	-0.1	-0.3	-0.5	0.3
Reach 3	0.5	0.5	0.5	0.5	0.4
Reach 2	0.7	0.6	0.7	0.7	0.6
Reach 1	1.4	1.4	1.6	1.6	1.6

Table 11.6. Table of reach average deposition for 1969 50-yr hydrograph under With-Project Conditions.

Average Deposition (ft)	**Year**				
Location	**1**	**3**	**10**	**20**	**50**
Reach 6b	-0.5	-0.5	-0.1	0.5	0.9
Reach 6a	2.6	2.6	3.4	4.2	4.9
Reach 5	0.1	0.1	0.3	0.6	1.0
Reach 4	-0.1	-0.1	-0.3	-0.5	0.4
Reach 3	0.5	0.5	0.5	0.5	0.6
Reach 2	0.7	0.6	0.7	0.7	0.7
Reach 1	1.4	1.4	1.6	1.6	1.7

Table 11.7. Table of reach average deposition for 1991 50-yr hydrograph under With-Project Conditions.

Average Deposition (ft)	**Year**				
Location	**1**	**3**	**10**	**20**	**50**
Reach 6b	-0.5	-0.5	-0.1	0.5	0.5
Reach 6a	2.6	2.6	3.4	4.2	3.6
Reach 5	0.1	0.1	0.3	0.6	0.8
Reach 4	-0.1	-0.1	-0.3	-0.5	0.3
Reach 3	0.5	0.5	0.5	0.5	0.3
Reach 2	0.7	0.6	0.7	0.7	0.4
Reach 1	1.4	1.4	1.6	1.6	1.5

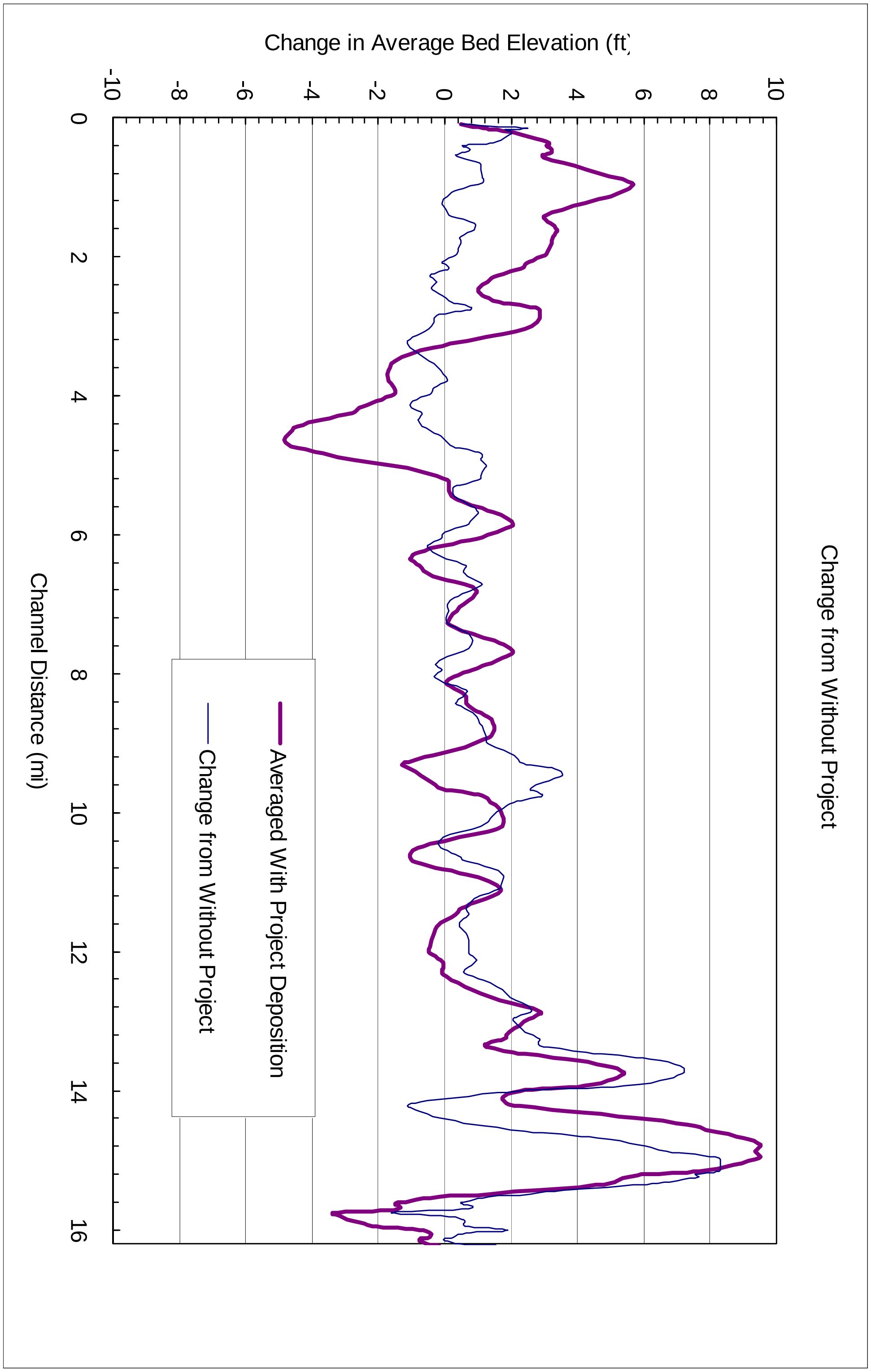


Figure 11.25. Average bed elevation changes at the end of the 50-yr simulations and a comparison with Without Project Future Conditions.

11.3. Future With-Project Conditions Bed Material

The increase in sediment loads in the Ventura River after dam removal may alter the bed characteristics. As the fine sediment concentrations increase in the Ventura River, more fine sediment may be stored into the bed. Much of this fine sediment will be stored temporarily and subsequent flows will remobilize the fine sediment and carry it to the ocean. Some of the fine sediment will be incorporated into the bed and increase the fraction of fine sediment in the bed.

There is limited information on the effect of fine sediment on coarse sediment, but one instructive set of laboratory experiments was performed by Schächli, U. (1992). In this work, fine sediment (silt and clay) was introduced into a flow over a static coarse bed. The increase in fine sediment in the bed was then measured, and the hydraulic conductivity of the bed material was measured. The fine sediment filled some of the pore space and decreased the hydraulic conductivity. The flow was then started again and the flow rate was increased to mobilize the hydraulic conductivity of the coarse bed. The fine silts and clays that had intruded into the bed were remobilized and the hydraulic conductivity of the bed returned to near the original values.

Much of the fine (silt and clay sized) sediment behind Matilija Dam will be removed by hydraulic slurry and deposited on the downstream floodplain. The remaining fine sediment in the reservoir region will be stabilized so that it will be mobilized at only flows that overtop the revetments. When this occurs, the flows will be large enough to mobilize the bed in the river channel downstream of the dam.

It is also important to realize that the current sediment concentrations in the Ventura River are very high. Typical sediment suspended sediment concentrations during floods are near 20,000 mg/l. While the project will increase the sediment concentrations in the Ventura River, the increase is incremental.

The model simulations predict that there will be very little change to the surface bed material in the Ventura River following dam removal (Figure 11.26, Figure 11.27, Figure 11.28). Figure 11.26 shows the d_{16} in the entire Ventura River for various years following dam removal. The initial and final values for the d_{16} are very similar for the entire river. At locations downstream of RM 3, however, the d_{16} does become finer in the first few years following dam removal as the fines are flushed from the upper reaches.

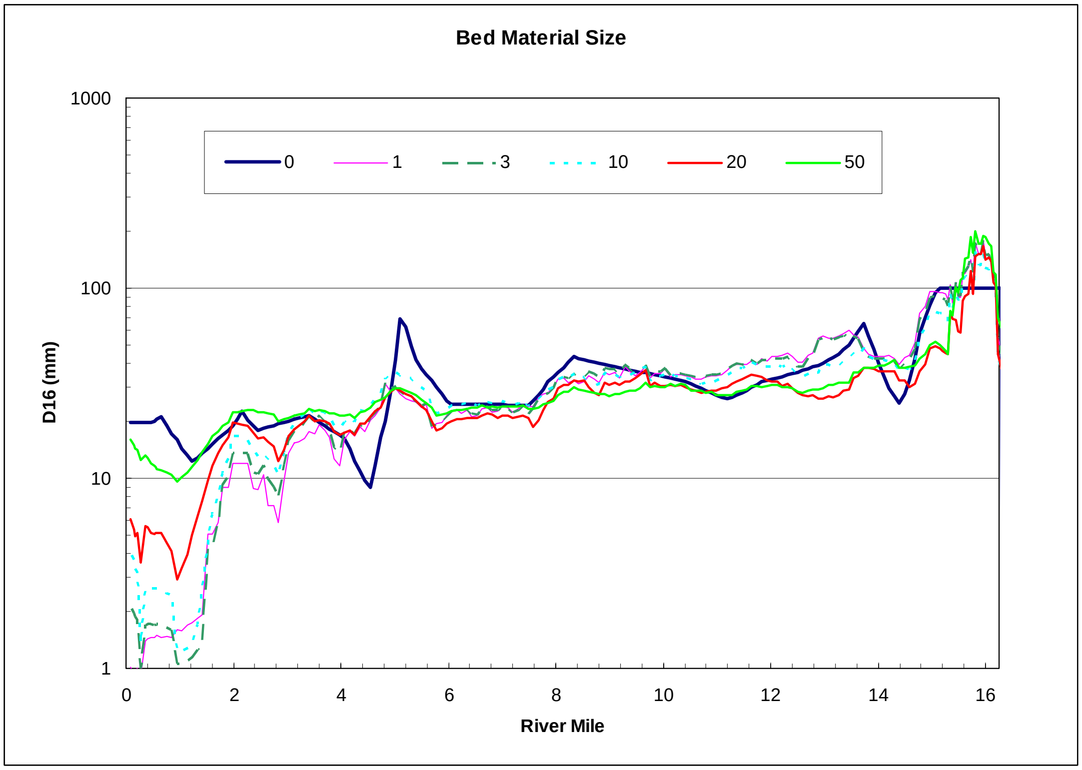


Figure 11.26. Bed Material Size following Dam Removal. D16 is the diameter of which 16% of the bed material is finer.

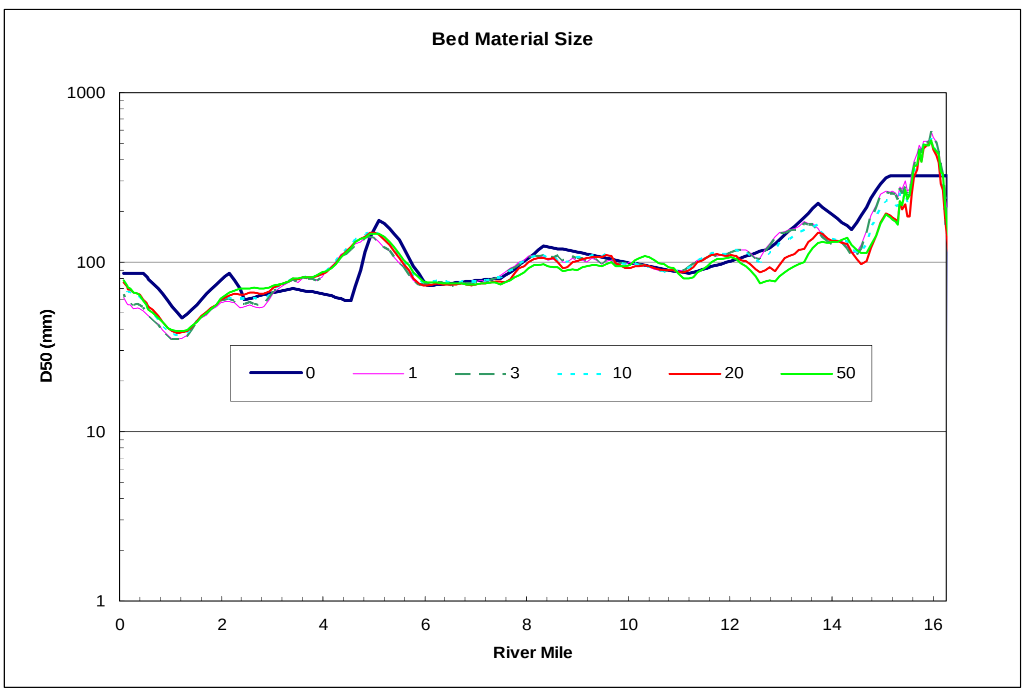


Figure 11.27. Bed Material Size following Dam Removal. D50 is the diameter of which 50% of the bed material is finer.

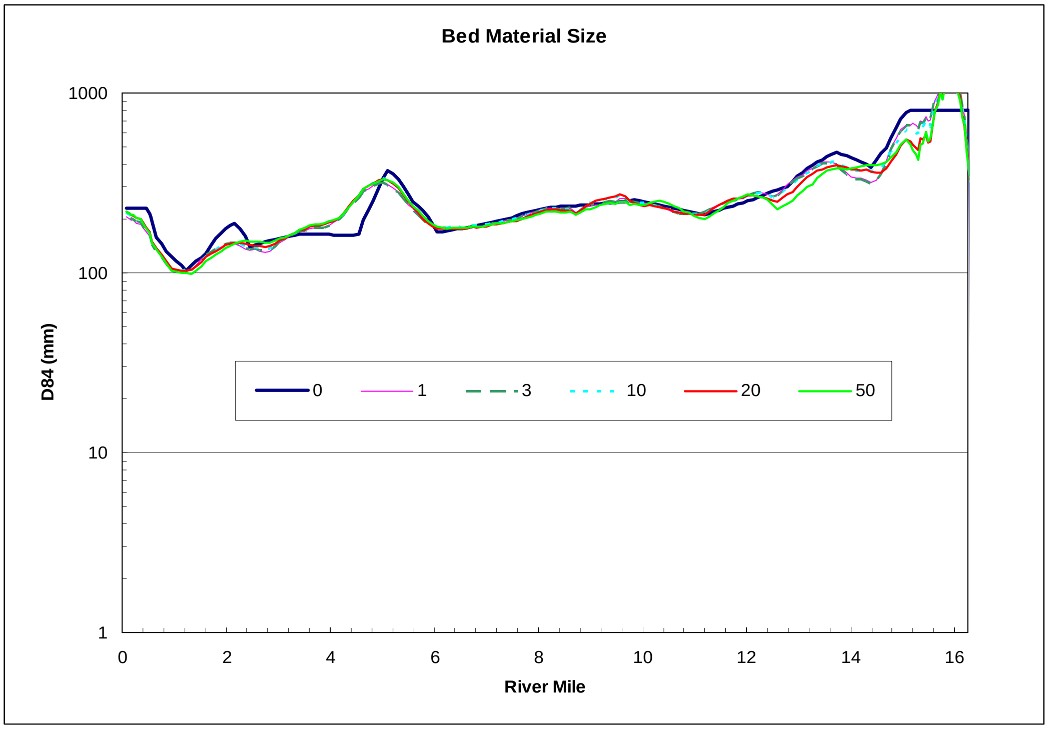


Figure 11.28. Bed Material Size following Dam Removal. D84 is the diameter of which 84% of the bed material is finer.

11.4. Design Storm Predictions for With-Project Conditions

In addition to analyzing long-term changes to the river system, single event floods were simulated as if they occurred immediately after dam removal. The results from simulating two consecutive 100-yr floods are shown in Figure 11.29. In general, the deposition values following two 100-yr storms are similar to the final deposition results of the 50-year simulation. This comparison was made to ensure that the river geometry used to predict with-project flood elevations, had higher bed elevations than those occurring during the 100-yr flood.

At locations where deposition was predicted, the 50-yr hydrograph deposition estimate was usually higher than the deposition estimated after two 100-yr storms. Exceptions to this include at RM 7.3, where the 100-yr storm deposition was approximately 1 foot higher than the 50-yr hydrograph value. Other exceptions include RM 10.4, where the 100-yr value was up to 2 feet higher than the 50-yr hydrograph value.

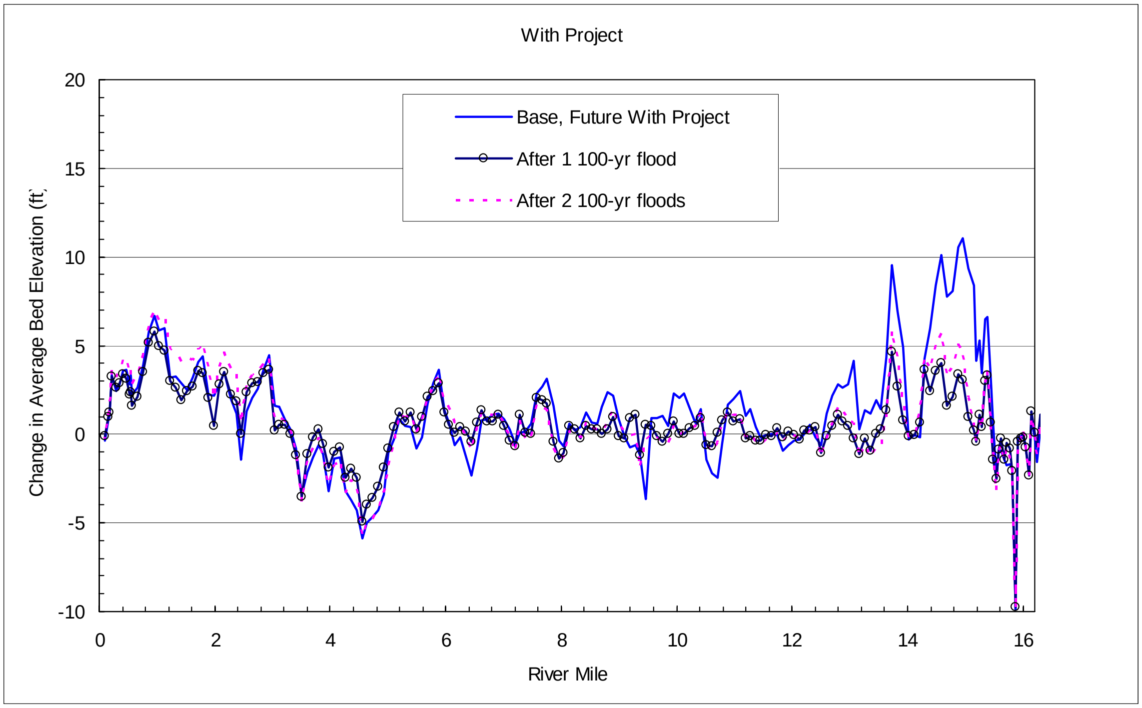


Figure 11.29. Average bed elevation changes at the end of the 100-yr storm and at the simulation end of the 1969 50-yr hydrograph.

12. Sensitivity and Uncertainty Analysis for With-Project Coniditions

The predictions of numerical models of natural systems are subject to uncertainties. This section discusses many of the sources of these uncertainties and estimates their value. It first compares the GSTAR-1D results to a simplified method to predict sediment aggradation following dam removal. Then several different stochastic hydrologic scenarios are used to estimate the uncertainties due to hydrologic variability. Next, the sensitivity to several model parameters is investigated. The uncertainties are evaluated based upon the hydrologic and sensitivity analysis.

12.1. Comparison to Sediment Wave Model

This section describes an analytical model of the movement of the sediments behind Matilija Dam. The model is much simpler than the numerical model (GSTAR-1D). 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. (2006), who extended the analytical description of aggradation of Soni et al. (1980) to describe downstream aggradation following dam removal. Greimann et al. verified the approach 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 12.1. The sediment accumulation sits on top of the original bed material that is at a stable and uniform slope.

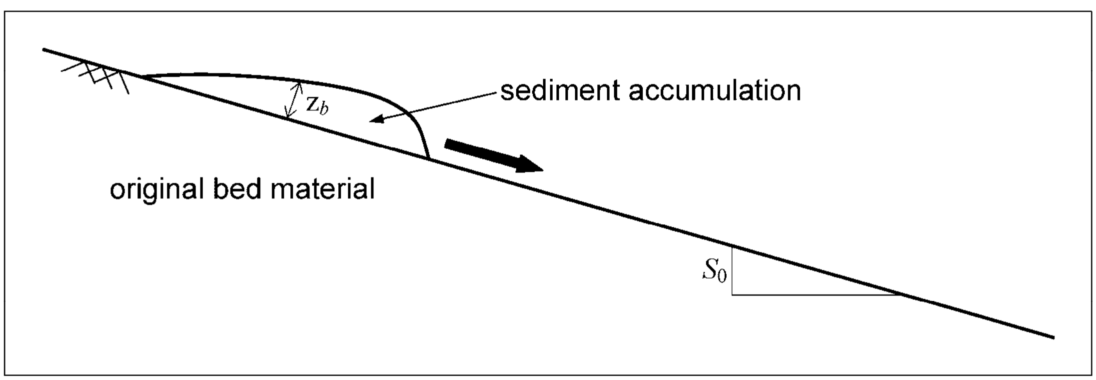


Figure 12.1. Schematic of idealized representation of the movement of a sediment accumulation, from Greimann et al. (2006).

The following equation was derived by Greimann et al. (2006),

$$\frac{\partial z_b}{\partial t} + u_d \frac{\partial z_b}{\partial x} = K_d \frac{\partial^2 z_b}{\partial x^2} \qquad \text{Eq 12.1}$$

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,

$$u_d = \frac{\left(G_d^* - G_0^*\right)}{h_d(1-\lambda)} \qquad \text{Eq 12.2}$$

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),

$$K_d = \frac{\left(b_d G_d^* + b_0 G_0^*\right)}{6 S_0 (1-\lambda)} \qquad \text{Eq 12.3}$$

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

$$G_d^* = a_d U^{b_d}, \qquad G_0^* = a_0 U^{b_0} \qquad \text{Eq 12.4}$$

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,

$$z(x,t) = \sum_{i=1}^{N-1} \frac{\left(z_{1i} + z_{1i+1}\right)}{4} \left[\begin{array}{l} \operatorname{erf}\left(\dfrac{x - u_d t - x_i}{2\sqrt{K_d t}}\right) - \operatorname{erf}\left(\dfrac{x - u_d t - x_{i+1}}{2\sqrt{K_d t}}\right) - \\ \operatorname{erf}\left(\dfrac{x + u_d t + x_i}{2\sqrt{K_d t}}\right) + \operatorname{erf}\left(\dfrac{x + u_d t + x_{i+1}}{2\sqrt{K_d t}}\right) \end{array} \right] \qquad \text{Eq 12.5}$$

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.

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 can be used 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 12.1. 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 12.1. 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 12.2. 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.

Most of the sand will also pass through the Ventura River and not deposit in the river channel. As a conservative estimate, it was assumed that half of the sand would travel as bed load in the system.

Table 12.2. Parameters used in analytical model of Matilija Dam removal.

Parameter	Value for Matilija Dam
S_0	0.01
G_d^* (ft^2/s)	0.02
G_0^* (ft^2/s)	0.01
λ	0.4
b_d	5
h_d (ft)	22

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. These results can be compared to the GSTAR-1D results presented in previous sections.

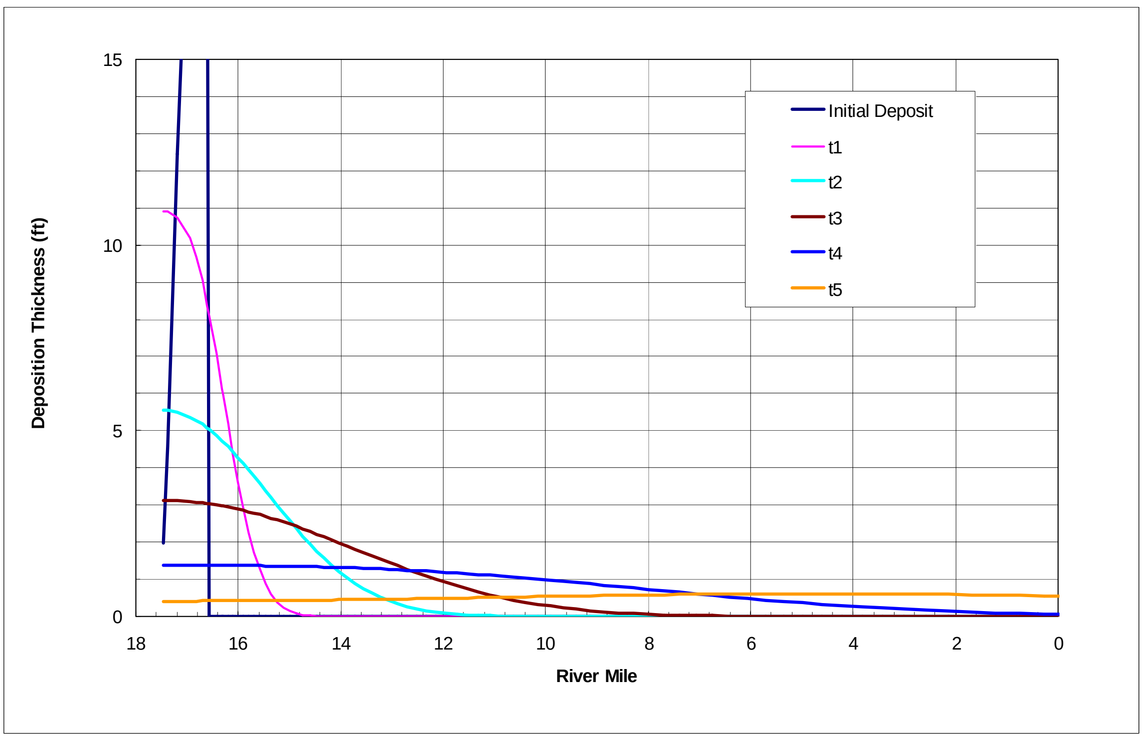


Figure 12.2. Simulation of Deposition Thickness using Sediment Wave Model.

12.2. Hydrologic Uncertainty

To estimate the hydrologic uncertainty due to hydrologic variability, the stochastic hydrologic scenarios described in Section 6.1.3 were simulated using GSTAR-1D. There were 40 additional simulations performed. The resulting range in average river bed elevation change (Δz_{ba}) is shown in Figure 12.3. The mean, maximum and minimum change in Δz_{ba} are also shown. In addition, with 40 different simulations it is possible to calculate the standard deviation of Δz_{ba}. The standard deviation varies between 0.2 feet and 1.6 feet (Figure 12.4).

To equate the change in average bed elevation change to a change in the 100-yr flood elevations a comparison between the two for the 1969 Hydrograph is shown (Figure 12.5). For most of the reach, and especially above RM 7, there is nearly a one-to-one correspondence between average bed elevation change and change to the 100-yr flood elevation. Therefore, it is reasonable to use the average bed elevation change as a measure for the change to the 100-yr flood elevations.

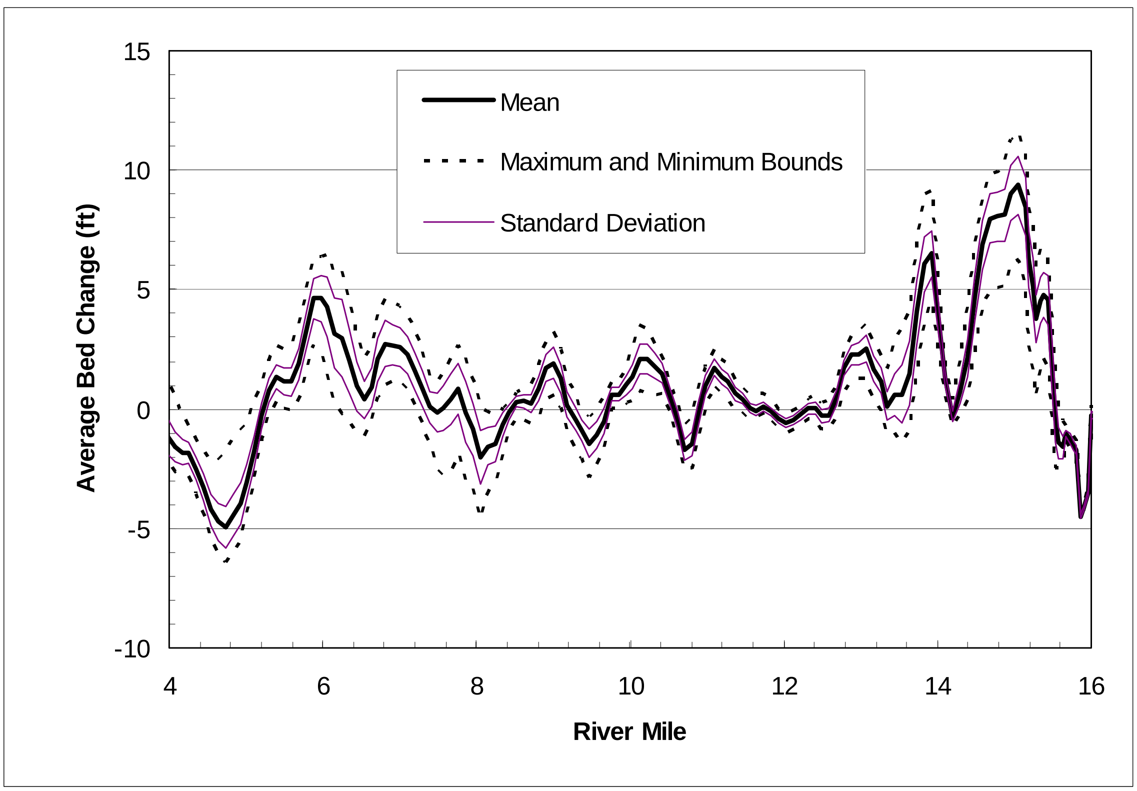


Figure 12.3. Average Bed Deposition for the Range of Stochastic 50-yr Hydrographs under With-Project Conditions

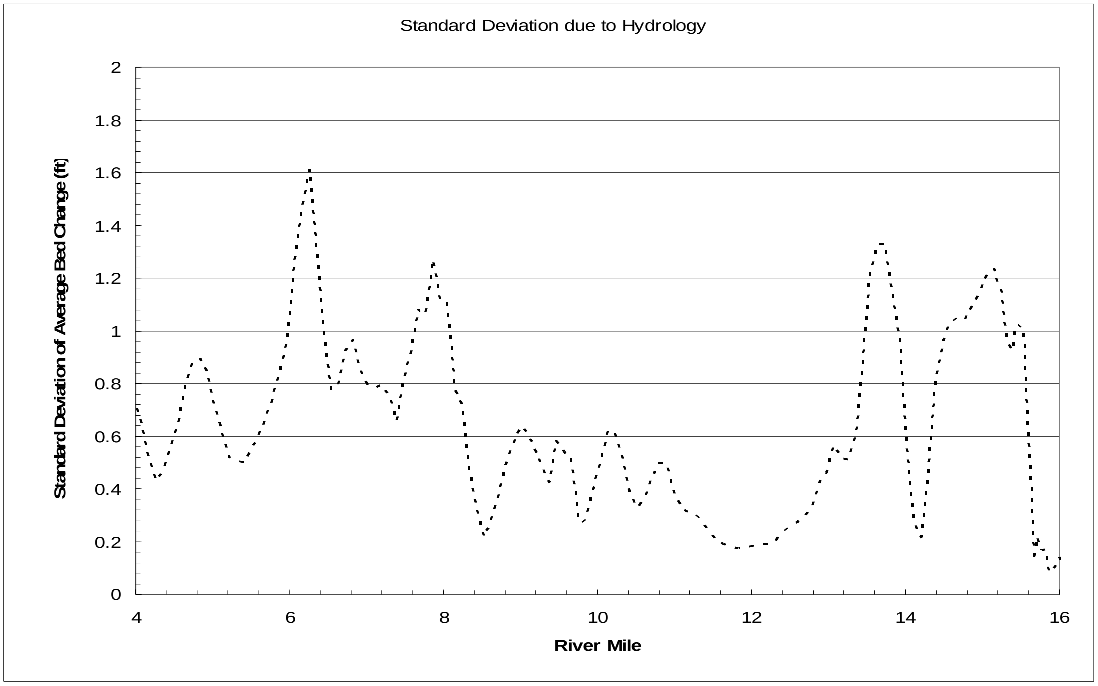


Figure 12.4. Standard Deviation in the Average Bed Elevation Changes due to Hydrologic Variability.

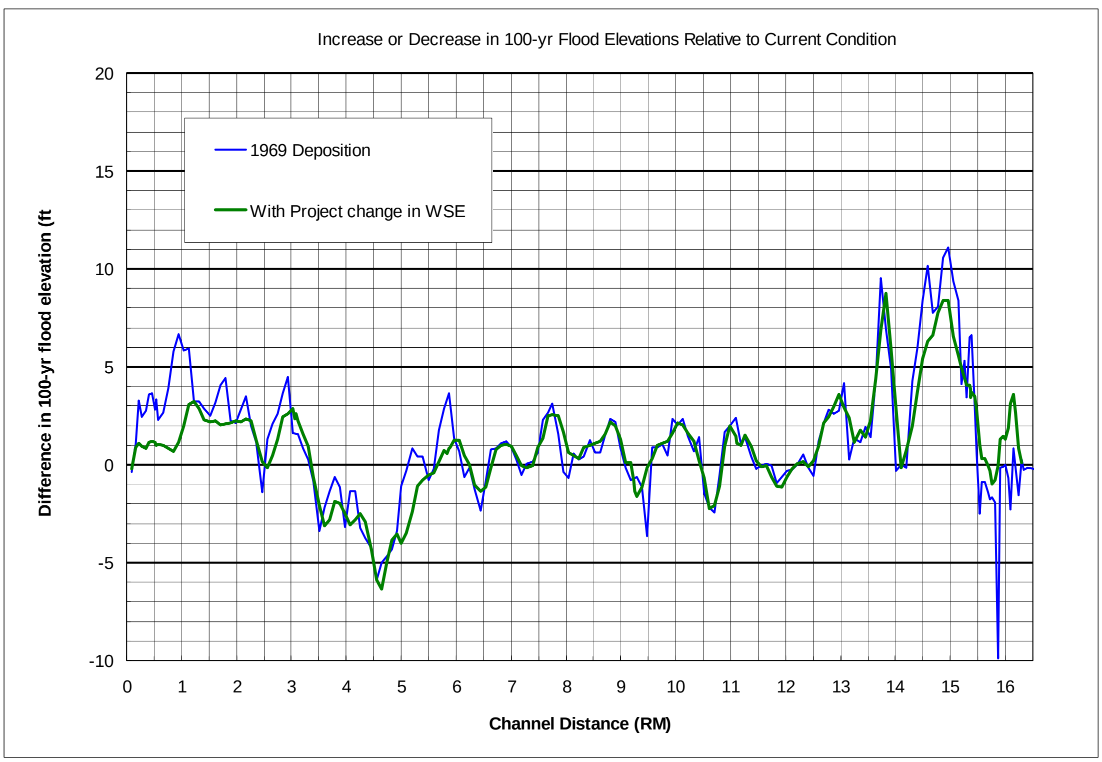


Figure 12.5. Comparison between Average Bed Elevation Change and Change to the 100-yr Water Surface Elevation Change.

12.3. Sediment Model Uncertainty and Sensitivity Analysis

A sensitivity analysis was performed on GSTAR-1D to determine the range of possible river response following the removal of Matilija Dam.

12.3.1. Sensitivity to Sediment Loads

Some of the most important inputs into the sediment model are the sediment loads from Matilija Creek, North Fork Matilija Creek, and San Antonio Creek. While, the suspended sediment load has been measured at North Fork Matilija Creek and San Antonio Creek, there is some scatter in the suspended load measurements, particularly at North Fork Matilija Creek. Furthermore, no significant bed load measurements have been performed at any of the tributaries. The bed load rates at North Fork Matilija and San Antonio Creeks used in the model were based upon computations and not measurements. There is considerable uncertainty in computations in bed load rates and the true bed load rate could vary by a factor of two from the computed one.

The reservoir deposition provides a record of sediment transport in Matilija Creek. The sediment volumes by size class were used to reconstruct a sediment rating curve for Matilija Creek. There is some uncertainty in the volume estimate by size class and in the procedure used to reconstruct the sediment rating curve. Therefore, there is some uncertainty in the sediment rating curve at Matilija Creek.

To estimate the effect of this uncertainty the sediment loads at these three Creeks were increased by a factor of two. Increasing the sediment loads should increase the deposition occurring in the Ventura River and help to bound the upper estimate of deposition. The effect of sediment loads on the 100-yr water surface was shown previously in Figure 10.2 (p. 210). Increasing the sediment loads raise water surfaces 1 to 3 feet along much of the Ventura River. The effect of increasing the sediment loads on the volume of deposition is shown in Figure 12.6. Increasing the sediment loads increase the amount of deposition by almost of factor of two. Most all the deposition occurs within the first 10 years. This is partly because the 1969 hydrograph was used, and the largest storm of record occurs in the first year of that hydrograph.

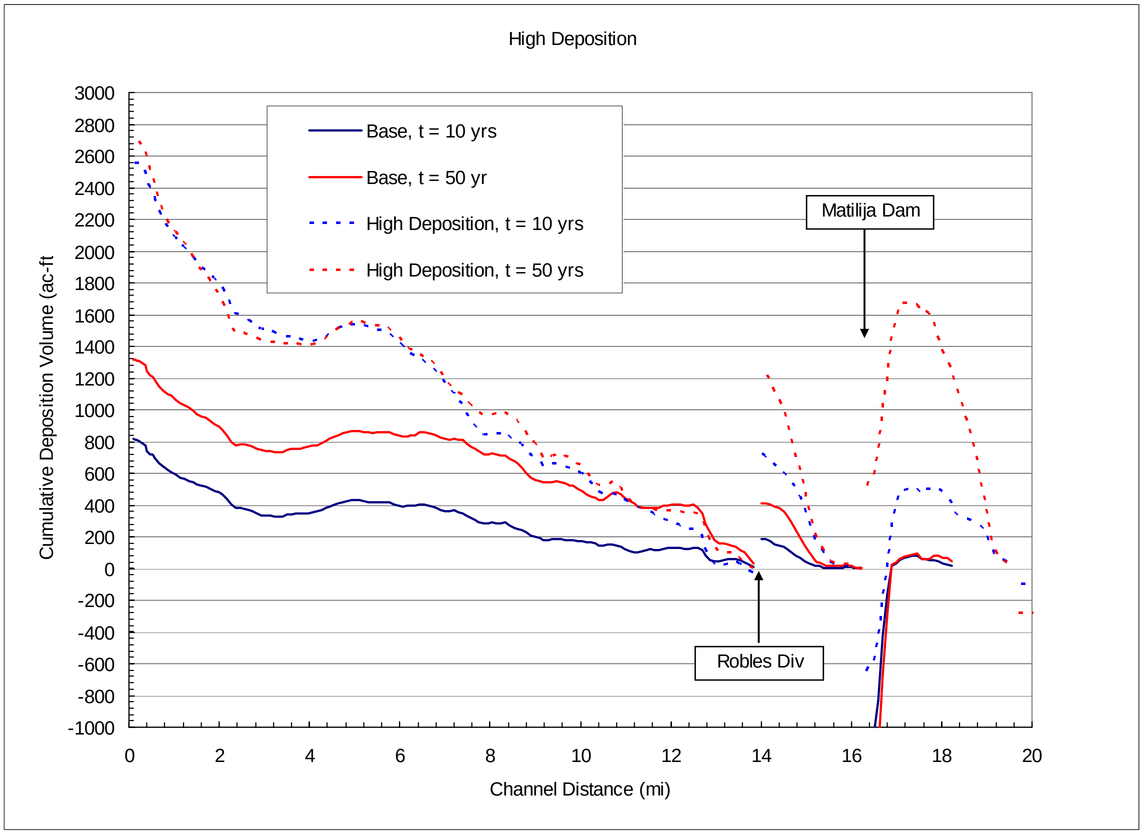


Figure 12.6. Sensitivity of Volume of Sediment Deposition to Incoming Sediment Loads.

12.3.2. Sensitivity to Bank Protection in Matilija Reservoir

After the dam is removed and reservoir fines are removed by slurry pipe, a channel will be created through the remaining sediments. This channel will be temporarily protected by channel revetment such as soil cement or riprap. The revetment will be designed to a certain elevation, above which water will begin to erode the banks of the reservoir and transport reservoir sediment downstream. Two different level of bank revetment were simulated: 1) No Bank protection, all the reservoir sediment is free to erode, and 2) 2-year Bank Protection: the revetment reaches to the 2-yr flood elevations within the reservoir. The base future with-project simulations include 2-yr protection.

The long-term deposition in the Ventura River is not sensitive to the bank protection in the reservoir (Figure 12.7). The revetment primarily protects the sands and silts of the reservoir. Because the Ventura River has a large capacity to transport sands and silts, only a small portion of the sands and silts from the reservoir will deposit in the channel, Most of the sands and silts picked up in the reservoir will be washed out to the ocean. The remainder of the sand and silt may be deposited on the floodplain, but very little of it is expected to remain in the main channel.

Additional work will be done on the effect of bank protection in the reservoir. The difference in bank protection will most likely affect the suspended sediment concentrations more than the deposition volumes. The one-dimensional model employed in this analysis is not considered detailed enough to estimate differences in bank protection measures in the reservoir area.

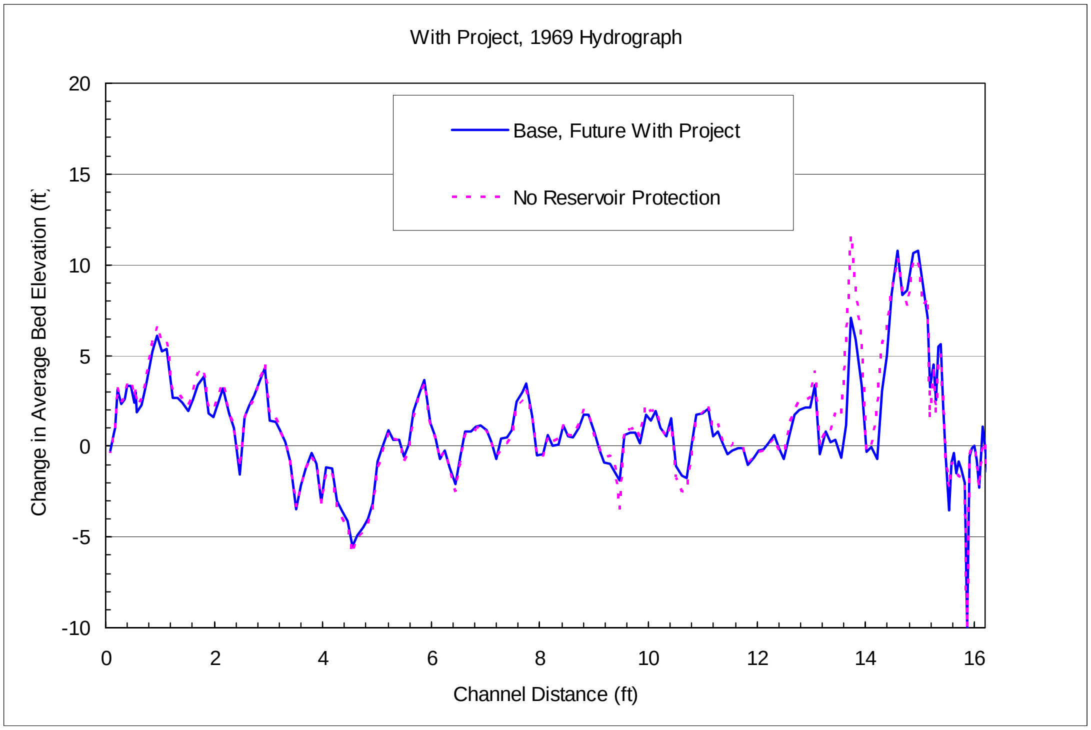


Figure 12.7. Sensitivity of Sediment Model to Various Bank Protection Measures in Reservoir.

12.3.3. SENSITIVITY TO ACTIVE LAYER THICKNESS

The active layer thickness controls the thickness over which sediment exchange between the bed and flow takes place. Sediment is added to the active layer when there is erosion and sediment is added to the active layer when there is deposition. The active layer thickness remains constant during the simulation. The active layer was varied between 1 foot and 10 feet. The variation of the active layer did not significantly affect the average bed elevations downstream of Matilija Dam. Therefore, the simulation is considered not sensitive to the active layer thickness.

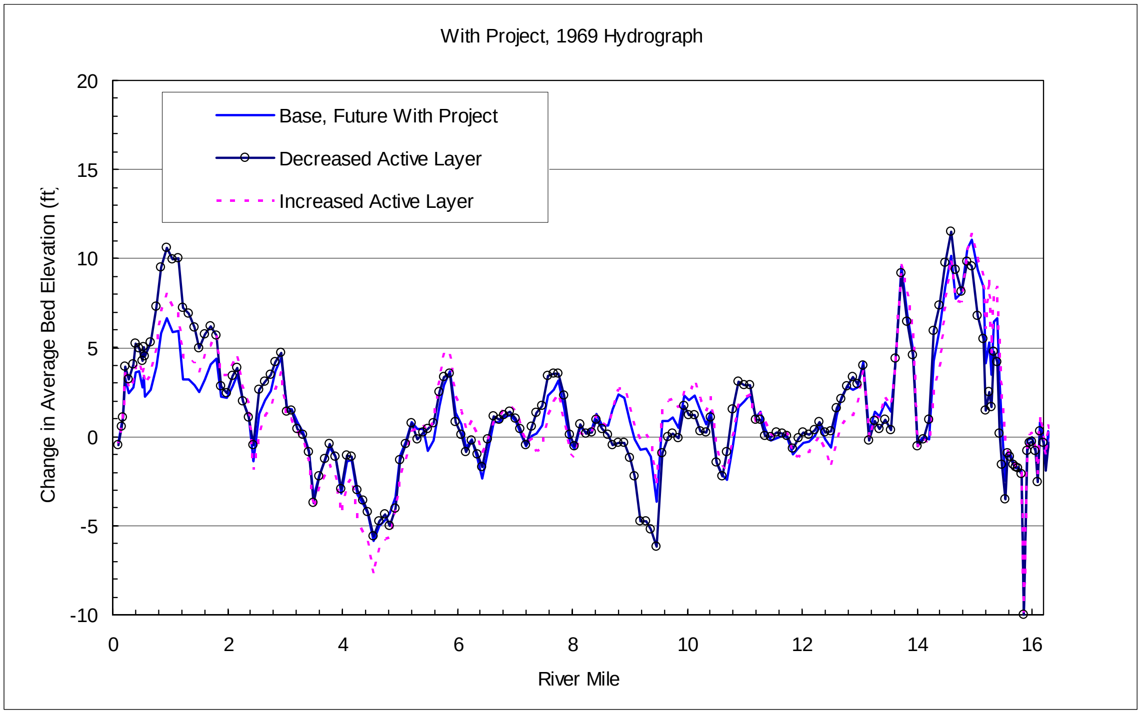


Figure 12.8. Affect of Active Layer on Average Bed Elevations for 1969 Hydrograph.

12.3.4. Sensitivity to Manning's Roughness Coefficient

The Manning's Roughness Coefficient was decreased and increased by 25 % and the results are shown in Figure 12.4. Decreasing the roughness decreased the deposition in the upper reaches significantly. In particular, the deposition at RM 15 decreased by approximately 5 feet. There is some uncertainty regarding the roughness in the reach from the dam to Robles Dam. There were no water surface elevations available in this area for calibration and therefore there is no check on the Manning's roughness coefficient. It is possible that the lower roughness values are more representative of the actual condition, but higher values are used to so that the deposition estimates in this reach are more conservative.

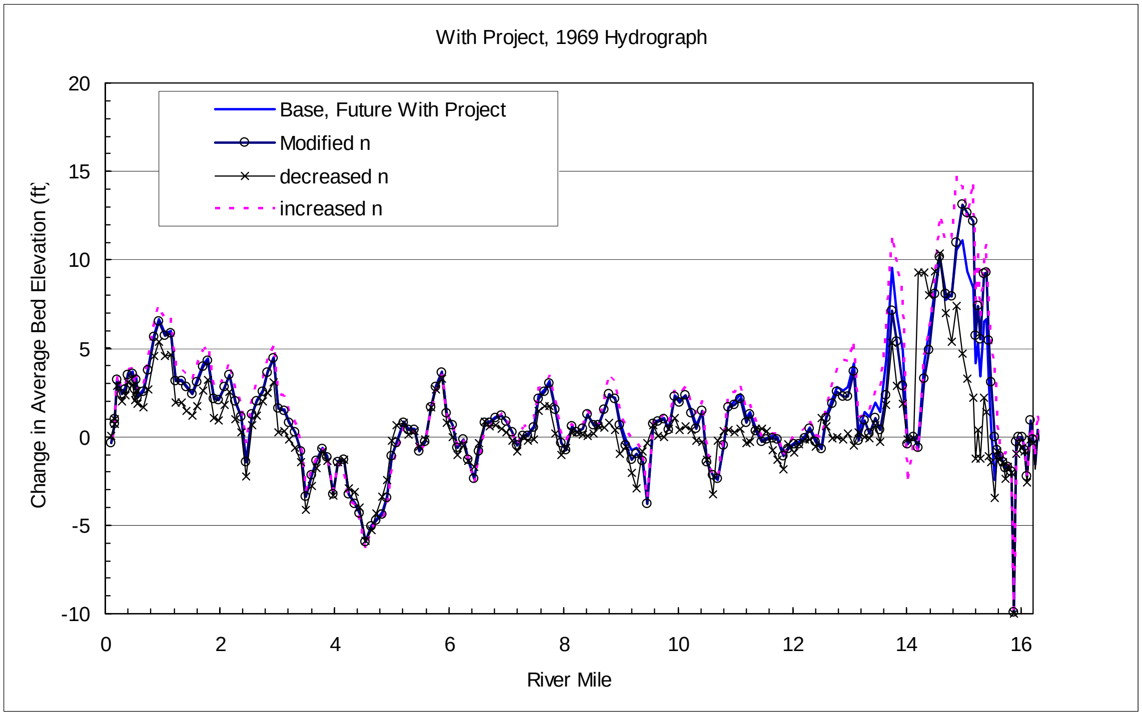


Figure 12.9. Affect of Manning's Roughness Coefficient on Average Bed Elevations for 1969 Hydrograph.

12.3.5. Sensitivity to Critical Shear stress

The non-dimensional critical shear stress was varied between 0.02 and 0.04. Increasing the critical shear stress to 0.04 had little effect on the simulation. Decreasing the critical shear stress to 0.02 actually increased the deposition at some locations. This is counter intuitive, but it is primarily because decreasing the critical shear stress in the reaches above the dam increased the sediment input in the Ventura River by increasing the amount of sediment eroded from the area upstream of Matilija Dam.

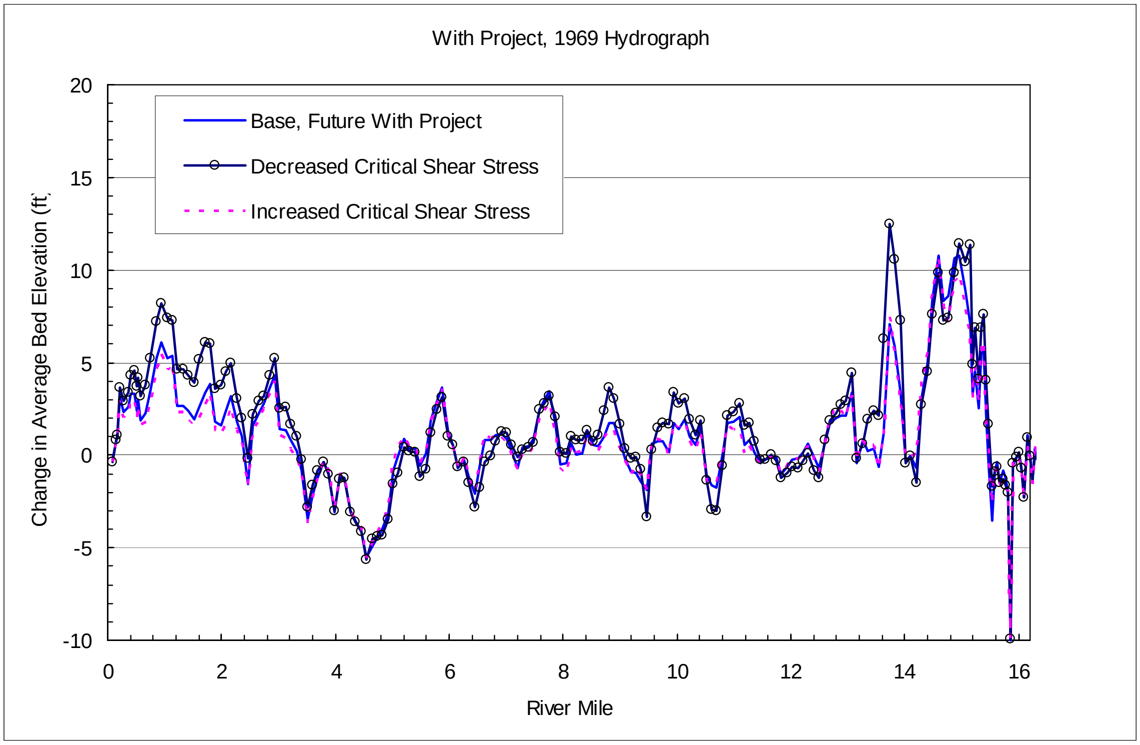


Figure 12.10. Affect of Non-Dimensional Critical Shear Stress on Average Bed Elevations for 1969 Hydrograph.

12.3.6. Sensitivity to Transport Formula

The Parker transport formula (Parker, 1990) was used instead of the Wilcock-Crowe (2003) bedload formula to compute bedload. The result to the calculation is shown in Figure 12.11. The simulation using the Parker Formula generally predicted greater extremes in the erosion and deposition. This is similar to what was found in the historical calibration (Section 6.5). The Parker Formula also tended to predict more severe erosion in the historical calibration. Based upon the historic calibration, the Wilcock-Crowe Formula gives more accurately estimates bedload on the Ventura River.

Figure 12.11. Affect of Transport Capacity Formula on Average Bed Elevations for 1969 Hydrograph.

12.3.7. SENSITIVITY TO BED LOAD ADAPTATION LENGTH

The bed load adaptation length is the length required to reach approximately two-thirds (e^{-1}) of the bed load capacity. Changing the bed load adaptation length generally had a small effect on the simulation results (Figure 12.12). However, at RM 15, increasing the bed load adaptation length slightly increased the deposition.

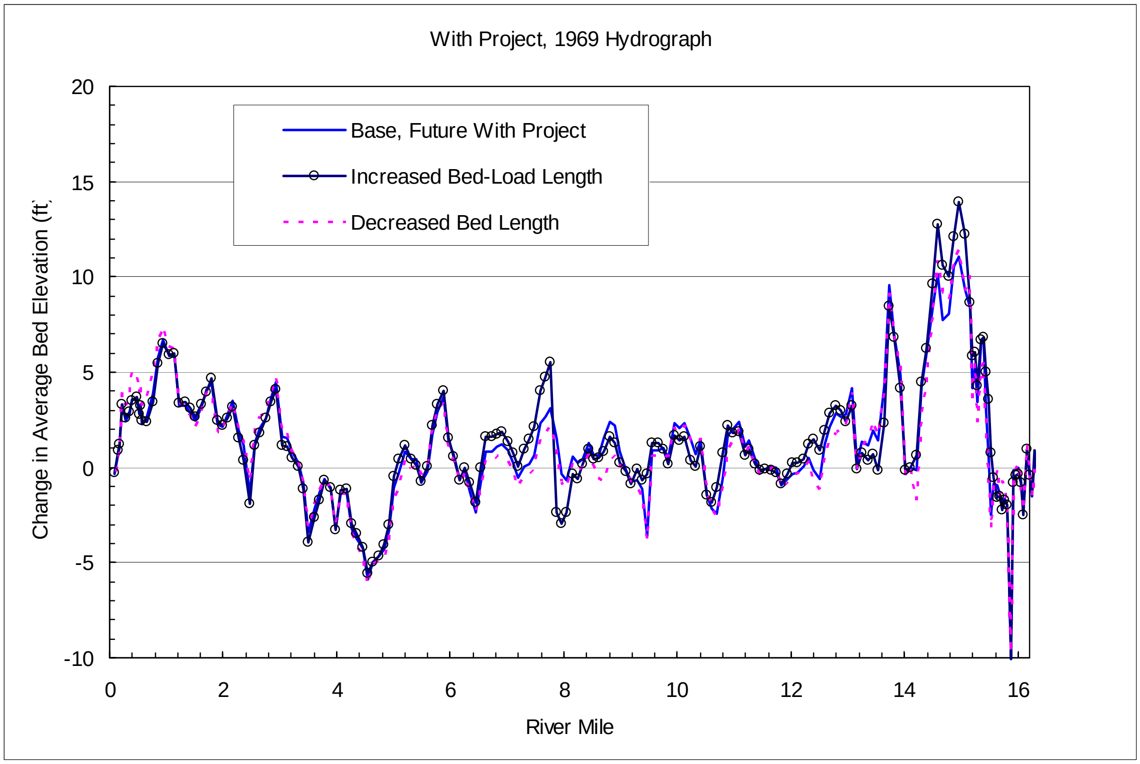


Figure 12.12. Affect of Bed Load Adaptation Length on Average Bed Elevations for 1969 Hydrograph.

12.3.8. SENSITIVITY TO ROBLES OPERATION

Sediment behind Robles is removed after most major storm events. The excavation is assumed to continue in the future. Figure 12.13 demonstrates what will happen if sediment is not excavated from behind Robles Dam. Generally, there will be more sediment that deposits upstream of the dam and less that deposits downstream of the dam.

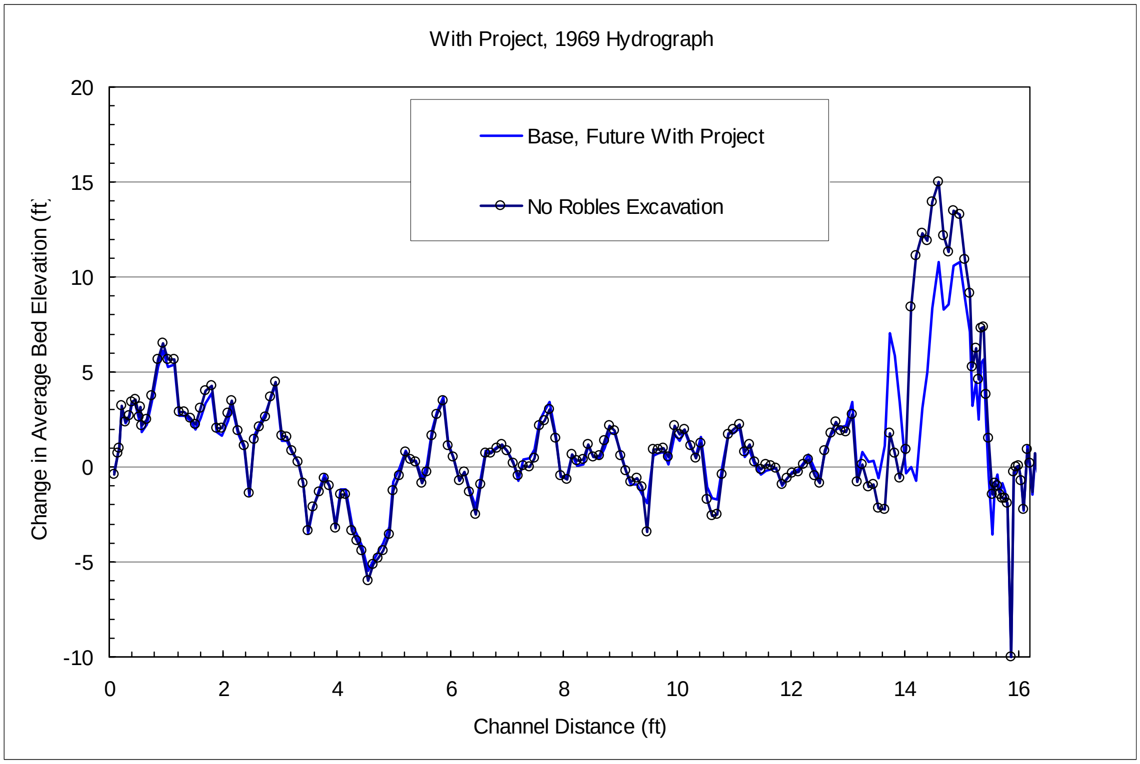


Figure 12.13. Affect of not Excavating Sediment at Robles Diversion.

12.4. Uncertainty Analysis for Flood Protection

A risk and uncertainty analysis was conducted to determine recommended levee heights throughout the project reach. The following procedure was followed to determine the risk and uncertainty of the flood impacts for the Future With-Project Conditions. The results from the analysis are given in Exhibit E. .

There were six locations identified in the feasibility study that need to be protected. For each location, the procedure to estimate the uncertainty 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 or on the web at: http://www.usace.army.mil/publications/eng-manuals/em1110-2-1619/toc.htm.

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 low, mean, and high bed geometry.

    a. For the "low" bed geometry the existing condition, 2005, cross sections were used.

    b. The "mean" bed geometry was determined by simulating the 50-yr period with GSTARS-1D starting with the 2005 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 2005 cross sections, as in the mean bed case. However, no riprap protection in the reservoir was assumed. All the sediment in the reservoir was available for erosion. In addition, the inflowing sediment loads from Matilija Creek and North Fork Matilija Creek were doubled. The Manning's Roughness Coefficient was increased in 0.05 in the main channel as the higher Manning's Roughness generally results in more deposition. 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 not significantly different from or 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 of the water surface due to the sediment model deposition (σ_m) was then computed by taking the absolute value of the difference between the high value (WS elevation) and the low value (WS elevation) and dividing by four (4).

7. The hydrologic standard deviation (σ_h) for each reach was taken from Figure 12.4.

8. The total standard deviation (σ_T) at each cross section was computed assuming that the hydrologic uncertainty is uncorrelated with the model uncertainty. $\left(\sigma_T = \sqrt{\sigma_h^{\ 2} + \sigma_m^{\ 2}}\right)$

9. Finally, the standard deviation at each index location (σ_I) was computed by taking the maximum standard deviation that occurs in that reach as defined in Figure 1.3 and applying it at that location.

A similar procedure was also followed to estimate the Without-Project Future condition uncertainty. The only difference is that the dam was assumed to remain in place.

The results from the uncertainty analyses are given in Exhibit E. Water Surface Elevations and Uncertainties.

13. Monitoring of Impacts

Matilija Dam is one of the largest and most complex dam removal projects ever undertaken in the United States. Many of the proposed approaches are novel and have not been tried previously. This statement applies foremost to the management of sediment. Substantial uncertainties exist regarding the amount of sediment of various sizes that will be supplied to the channel network, the rate the sediment particles will be transported downstream, and where sediment will accumulate following a given flood. This monitoring plan is designed specifically for this project as described in the Matilija Dam Ecosystem Restoration Feasibility Study – Final Report – September 2004. This monitoring plan is not a *wish list*; rather it states the minimum data that needs to be collected to ensure a successful project.

The Final Report of the Feasibility Study anticipates that the proposed dam removal activities and subsequent adjustment of the Ventura River channel will require approximately 20 years. Given a well-formulated monitoring plan, there will be many opportunities to evaluate progress on various aspects of the plan and make appropriate adjustments. In particular, a portion of reservoir deposit will be stabilized initially with soil cement and, then, later that the soil cement will be gradually removed. Success of the project depends, in part, on collecting the information necessary to assess conditions in the Ventura River channel at various stages in the removal process, and the likely adjustment of the channel and flow characteristics to the release of additional sediment.

The broad objectives of the flow and sediment monitoring plan are:

1. To determine the water surface elevation of a given discharge along the 17 mile reach from Matilija Dam to the coast.

2. To determine the quantity and particle size distribution of sediment supplied to the channel network, the rate of downstream movement, location of sediment accumulation in the channel and/or floodplain, and the quantity of sediment contributed to the coastal zone.

3. To determine the erosion of material in the reservoir region.

4. Evaluate the condition of aquatic and riparian habitat in the Ventura River through the entire project reach.

5. Estimate deposition of material in Robles Diversion.

These objectives are closely related. An accumulation of sand in the Ventura River channel would affect and alter the hydraulic and sediment transport characteristics of the channel in complex ways. The accumulation would locally reduce the channel capacity to pass a flood. The loss of channel capacity would be offset somewhat by a decrease in flow resistance. Similarly, the decrease in bed-material size and increased velocity will lead to a local increase in the transport rate, which will tend to reduce the local accumulation of sediment.

It is not possible to continuously observe and measure all of the flow and sediment transport characteristics of interest along the entire 17 mile reach. Furthermore, a river flow and sediment routing model, most likely a refined version of the one developed for the Feasibility Study, will be the analytical framework with which to analyze observations and evaluate the significance of various events. The quantity of flow and sediment entering and leaving the project reach will be measured at gaging stations. Similarly, the channel size, morphology, and particle size composition will be surveyed following significant flows or contributions of sediment. These measurements will provide significant information about the condition of the Ventura River at specific locations and points of time. The river flow and sediment routing model will allow one to investigate the hydraulic and geomorphic characteristics more or less continuously in space and time. The observations and measurements described below serve as necessary input to the model, as well as a means to evaluate model results, in addition to providing essential hydrologic and geomorphic information that are independent of any model results.

The removal of Matilija Dam and the subsequent increase in sediment contributed to the downstream reach will occur within the context of large variations of flow and sediment supplied to the channel network across the entire drainage basin. Over the estimated 20 years span from dam removal to full stabilization of the remaining reservoir deposit, there will be a large variation in flow and sediment supply.

13.1. Water Surface Elevation

The primary objective of river flow and sediment monitoring activities following the removal of Matilija Dam and the release of stored sediment will be to identify any increase in flood risk due to an increase in water surface elevation at a given discharge. Increases in water surface elevation will occur, as described above, where channel capacity is reduced due to the accumulation of sediment and/or the hydraulic resistance of the channel increases due to coarsing of the river bed-material, encroachment of vegetation etc. Based on experience, the magnitude and directions of changes in the water surface elevation at a given discharge will vary significantly through the project reach and over time.

To monitor the project impacts on the flood water surface elevations, we propose using small, relatively inexpensive stage recorders. These stage recorders provide a continuous measurement of water surface elevation over the range of stream flows that occur after the project. An example of such a stage recorder can be found at: http://www.solinst.com/index2.html.

As described above in Chapter 4, Section 4.4, specific reaches of the Ventura River have been identified as having a greater flood risk, e.g. the reach from Camino Cielo to the Robles Diversion and near Meiners Oaks. Within these reaches of particular interest, stage recorders should be deployed every few to several hundred feet adjacent to the channel margin where the possibility of overbank flow is greatest. Approximately 10 to 30 stage recorders may be required to cover the reaches of interest. The stage recorders can be retrieved and data downloaded after any significant flood or annually as desired.

Detailed information about the variations of water surface elevation with stream flow derived from the stage recorders as well as surveys of high marks will be applied to improve the calibration of the hydraulic models.

13.2. Sediment Supply, Transport, and Deposition

After dam removal, approximately 2.6 million tons of sediment will be eroded from the reservoir deposit and transported downstream in the Ventura River. The majority of the material transported downstream will be sand, with a significant amount of silt material. There are lesser amounts of gravel and cobble. As described in Chapter 5, silt and clay particles tend to remain suspended within the water column at any appreciable discharge in the Ventura River, and thus, are unlikely to accumulate in the channel. Only the sand, gravel, and cobble sized sediment in the reservoir deposit have a significant potential to accumulate in the channel of the Ventura River. The temporary accumulation within a short reach of the Ventura River of even a relatively small portion of the sand, gravel, and cobbles stored behind Matilija Dam could substantially reduce the channel capacity locally. Sediment, sand sized and larger, will accumulate in the Ventura River channel when the total quantity of material, sand sized and larger, supplied to the channel by all sources, including tributaries as well as by erosion of the reservoir deposit, exceeds the transport capacity of the river flows.

As described in Chapter 5, sand in the Ventura River is overwhelmingly transported as suspended load within the water column and the flux at a given discharge is most efficiently determined by physical sampling the flow-weighed concentration. Gravel and cobble-sized particles are transported as bedload, that is, frequently in contact with the riverbed, and the flux at a given discharge is most efficiently determined by applying an appropriate computational method.

Suspended Sediment

The concentration of sediment suspended in river flows has been determined over a range of discharges at 4 gaging stations in the Ventura River Basin, Matilija Creek near reservoir near Matilija Hot Springs, the North Fork Matilija Creek near Matilija Hot Springs, San Antonio Creek near Casitas Springs, and the Ventura River near Ventura, see Figure 2.1. Sampling of suspended sediment concentrations at these gaging stations plus the proposed activation of an additional gaging station downstream of Matilija Dam would provide the information needed to determine the daily quantity of suspended sediment eroded from the reservoir deposit as well as the daily quantity of suspended contributed (1) to the upstream end of the reservoir, (2) by the two largest tributaries to the Ventura River, the North Fork Matilija Creek and San Antonio, and (3) to the lower reach of the Ventura River.

Sampling of suspended sediment concentrations at 3 of the existing gaging stations, Matilija Creek near reservoir near Matilija Hot Springs, the North Fork Matilija Creek near Matilija Hot Springs, and San Antonio near Casitas Springs is sparse and the variation of suspended sediment concentrations with discharge at each gaging station is large. To the extent possible, additional suspended sediment samples should be collected over the range of discharges at the 3 existing headwater gaging stations to better define the baseline, pre-project conditions.

Most of the suspended sediment transport by the Ventura and its tributaries in a given year occurs, on average, during the 3 to 5 days of highest river flow. Periodic sampling of suspended sediment, e.g. once a week, will not be a cost-effective approach. The average condition noted above is, itself, somewhat misleading. Over 90 percent of all sediment transported passed the Ventura River near Ventura gage since 1969 has been transported when an El Niño condition prevailed, which typically occur every 3 to 5 years, Andrews et al (2004). When El Niño conditions prevail, significant quantities of sediment will be transported during 10 to 20 days between December and April. During years with relatively little runoff, i.e. non-El Niño conditions, there may be few, if any, days with river flows transporting any appreciable quantity of suspended sediment.

An efficient, cost-effective sampling scheme should focus as the significant sediment transporting flows. The periods of high river flow are very busy with many competing demands on the individuals who maintain and operate the gaging stations. It seems unlikely that the collection of suspended sediment samples during the periods of highest river flows can be added to the hydrologist's responsibilities. We propose that an individual be trained and dedicated to the collection of suspended sediment samples during those winters when El Niño conditions exist. When evaluating this proposal, it should be recognized that there will only be 1 or possibly 2 El Niño episodes between now, Fall 2006, and the removal of the Matilija Dam.

13.3. River Bed Material

Determination of the size distribution of river bed-material at a given location is a relatively quick and simple process and provides information needed to evaluate (1) areas of sediment accumulation and depletion with the channel, (2) aquatic habitat, (3) the rate of stream flow infiltration into the river bed, and (4) the transport rate of bed-material over a range of river flows. Accordingly, monitoring the size distribution of bed-material in the Ventura River from Matilija Dam to the coast is a cost-effective method to assess the downstream impact following the removal of Matilija Dam. During the Feasibility Study

(2001), the distribution of particle sizes in the channel bed-material were determined at sites spaced every 1 to 2 years beginning in Matilija Creek above the reservoir downstream to the Ventura River estuary. Subsequently the distribution of bed-material particle size was determined near the North Fork Matilija and San Antonio Creek gaging stations in 2005. Assuming that the deconstruction of Matilija Dam will not begin for a few more years, the particle size distribution of bed-material through the project reach should be resurveyed prior to the dam removal to establish unambiguously the baseline condition.

As described above, a majority of the sediment stored behind Matilija Dam projected to be released downstream is sand. Consequently, if a significant quantity of sediment were to accumulate somewhere in the project reach, it would appear as an increase in the percent of sand exposed on the surface bed-material, which is typically coarse gravel and cobbles. The percent of sand exposed on the surface bed material can be determined at a given location in just a few minutes. Thus, the percent of sand exposed on the riverbed should be monitored at those sites where a lot of channel capacity would be of greatest concern and/or where model calculations suggest an accumulation is most likely. Given the simplicity of these measurements, the percent of sand on the riverbed surface at the priority sites should be resurveyed after every significant flood.

Monitoring of infiltration rates (direct and).

13.4. River Channel Topography

A dedicated topographic description of the Ventura River channel is essential for the accurate prediction of water surface elevations over a range of flood discharges. Relatively small errors in the assumed size, shape and/or alignment of the channel will degrade the computed results of the river flow and sediment routing model. The current topography of the Ventura River channel was determined in ------ from aerial photography and a LIDAR survey. The cost of making the basic survey and then constructing the digital topography is substantial. Consequently, repeated and complete topographic surveys of the project reach are not an efficient approach to identify possible changes in the Ventura River channel. As described above, we propose that continuous recordings of high flow water surface elevations in reaches of particular interest, and relatively frequent re-examination of bed-material size, including the percent of sand on the riverbed surface be relied upon to identify reaches with appreciable channel changes. When such changes are identified, a topographic survey should be conducted of the affected reach. In any case, a LIDAR and aerial photography survey should be conducted of the entire project reach 5 and 10 years after dam removal or following a relatively large flood, recurrence interval of 25 years or greater.

Reservoir erosion

Many of the impacts downstream are related to the erosion of sediment in the reservoir region. In addition, the goal of the project is to remove all revetments within the reservoir area. If the revetments are removed too quickly, it is possible that the river is overloaded with fine sediment. If the revetments are removed too slowly, many decades may pass and the goal of removing the revetments is forgotten. There may be loss of habitat value if the revetments are not removed and the project goals will not be fully realized.

One of the critical factors in determining when to remove the reservoir revetment is the progression of reservoir erosion. The LIDAR information will provide the information necessary to track reservoir erosion.

13.5. High Flow Bypass

The high flow bypass will be built at Robles Diversion to sluice sediment from behind Robles Diversion Dam. CMWD currently tracks the amount of sediment they remove from the basin behind the dam (Table 1.5, p.35). It is assumed that they will continue to record this information and this information should be transmitted to the County and other agencies involved in monitoring the removal. If the amount of sediment being removed is excessive, alternative operational strategies may be implemented.

13.6. Summary of Monitoring and Adaptive Management

Table 13.1. Summary of Monitoring Plan.

Observations	Frequency	Location
Water Surface Elevation	Continuous recording of river stage	Twenty to 30 locations where the risk of overbank flow is of particular concern and along major levees.
Suspended Sediment	Every 1 to 3 days when discharge exceeds the 10 percentile duration.	1. Matilija Creek near reservoir near Matilija Hot Springs 2. Matilija Creek at Matilija Hot Springs 3. North Fork Matilija Creek near Matilija Hot Springs 4. San Antonio Creek near Casitas Springs 5. Ventura River near Ventura
Riverbed Material		Every river mile from 16 to 8, and every 2 river miles from 8 to coast
Aerial Photography and LIDAR Surveys	Repeated during year 5 and 10 after dam removal, after any flood with RI >25 years, and as indicated by changes in water surface elevation or river bed material size.	Beginning 5 miles upstream of dam to the coast (approximately 21 miles).
Robles Excavation Volumes	After every major storm	Upstream of Robles Diversion

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15. Exhibit A. Hydrology Reports

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16. Exhibit B. Flow Duration Curves by Month

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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	Flow Values (cfs)											
Time Flow	January	February	March	April	May	June	July	August	September	October	November	December
is Below												
This Value												
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	Flow Values (cfs)											
Time Flow	January	February	March	April	May	June	July	August	September	October	November	December
is Below												
This Value												
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	Flow Values (cfs)											
Time Flow	January	February	March	April	May	June	July	August	September	October	November	December
is Below												
This Value												
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	Flow Values (cfs)											
Time Flow	January	February	March	April	May	June	July	August	September	October	November	December
is Below												
This Value												
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	Flow Values (cfs)											
Time Flow	January	February	March	April	May	June	July	August	September	October	November	December
is Below												
This Value												
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

17. Exhibit C. Cross Sections used in Study

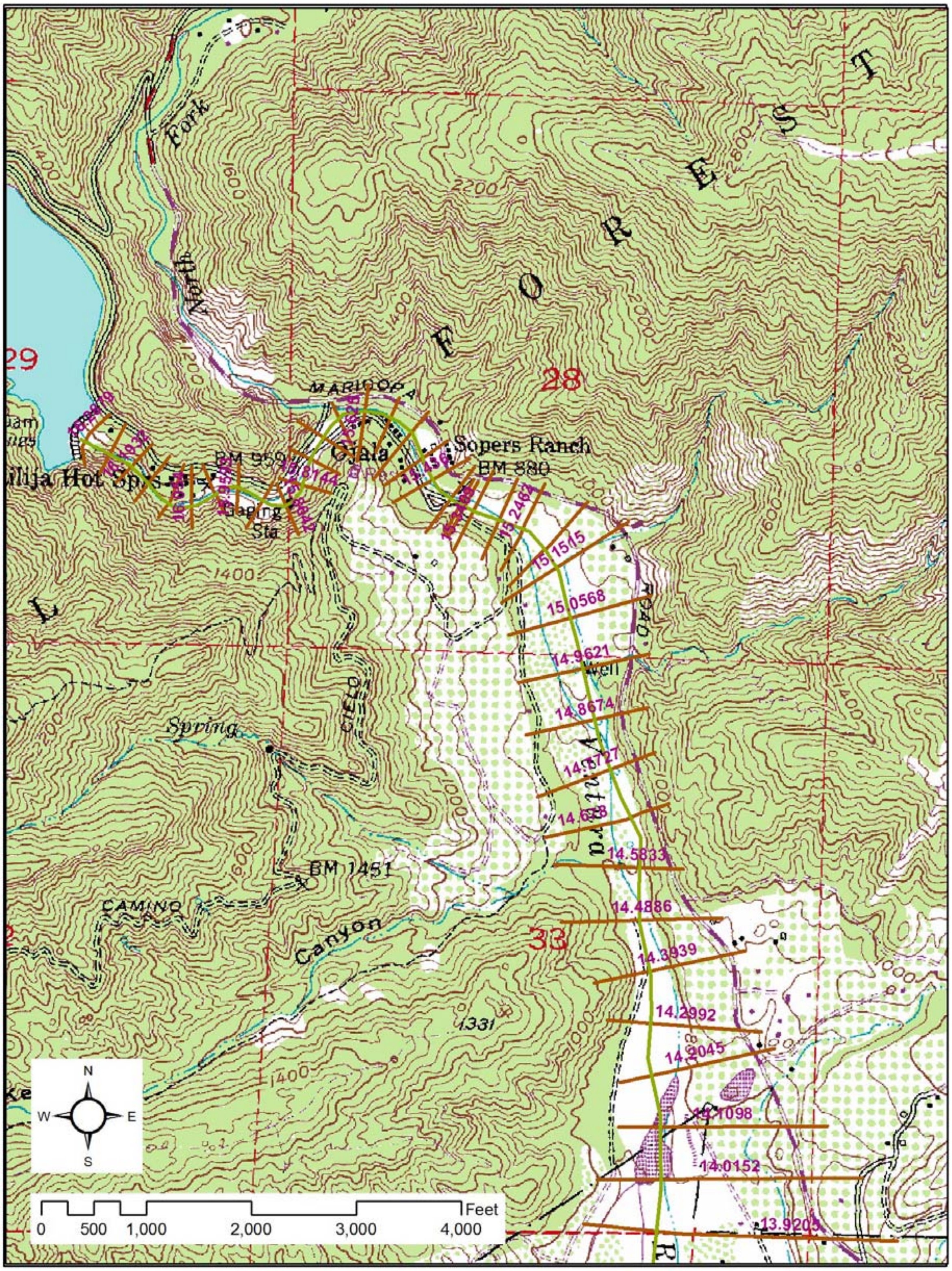
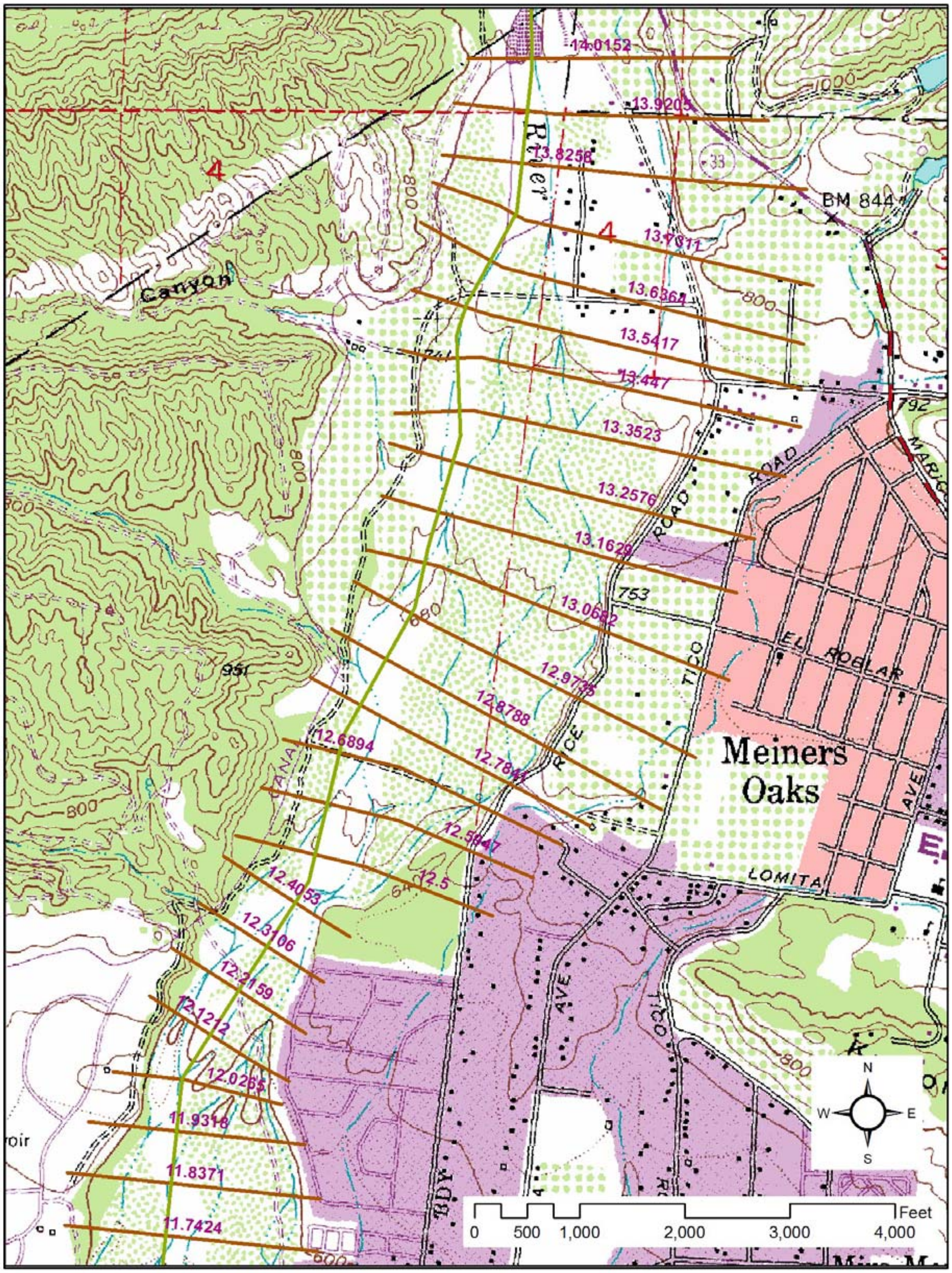
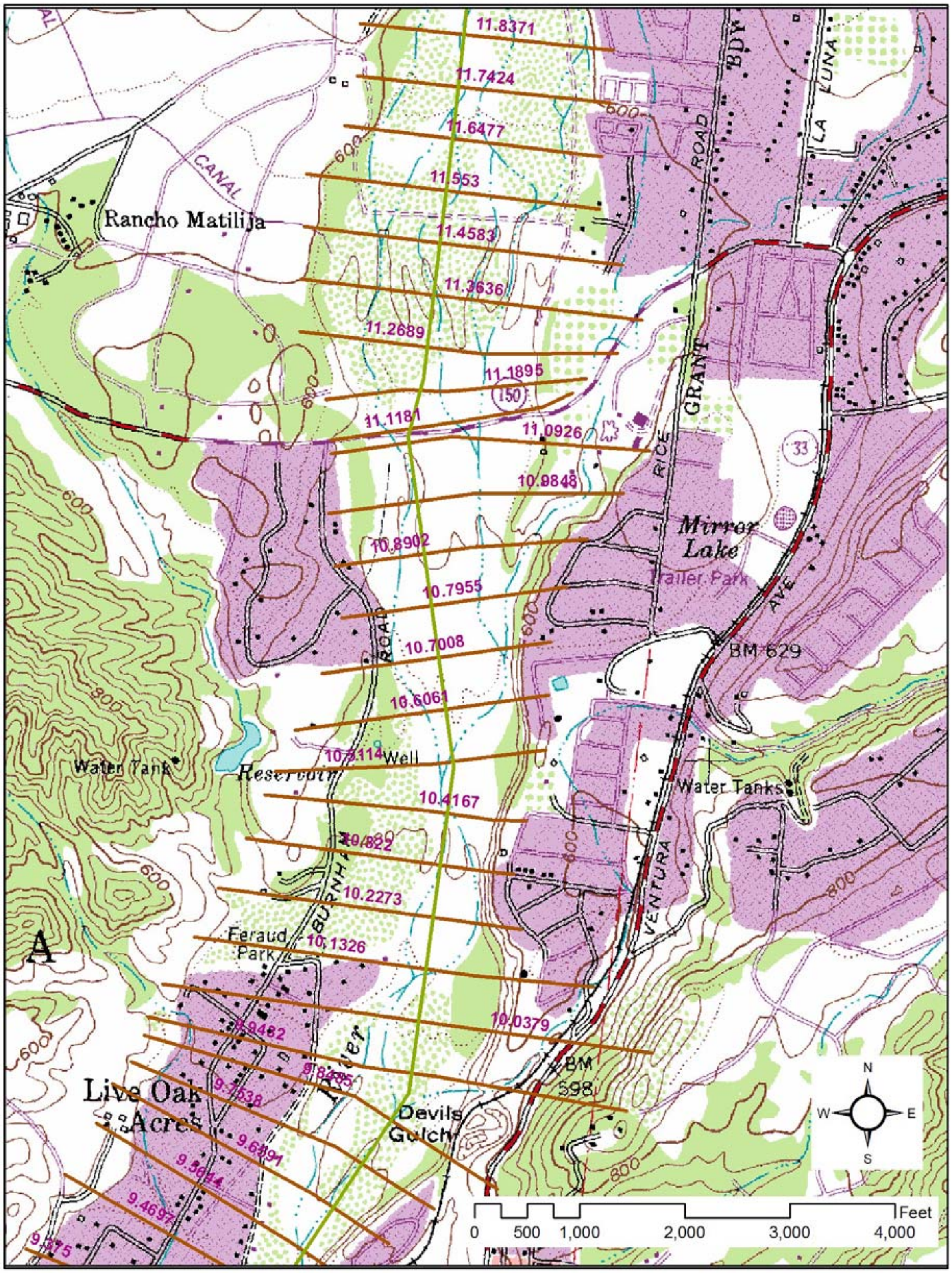


Exhibit C. Cross Sections used in Study





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11.6477
11.553
11.4583
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11.2689
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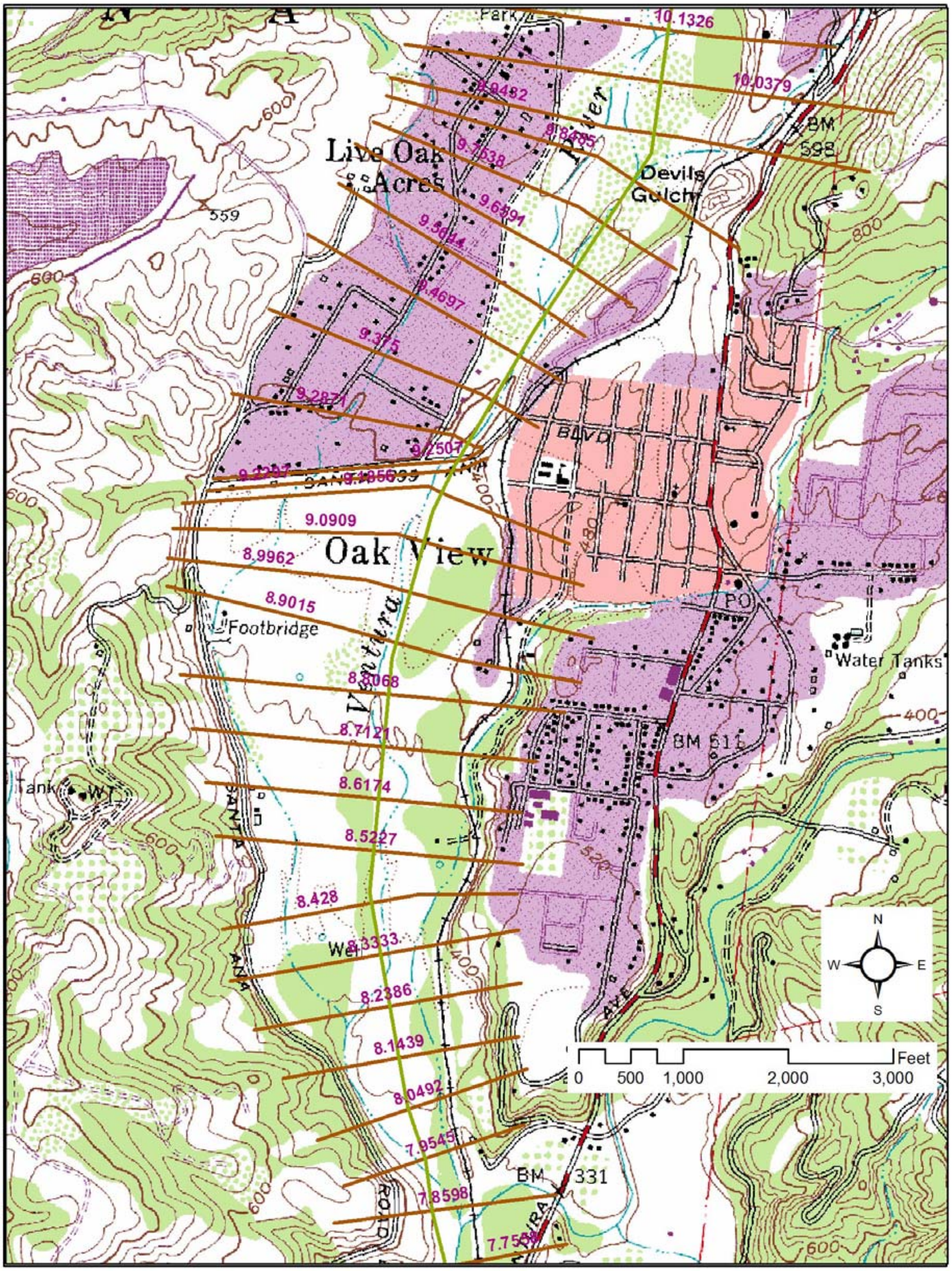
10.8902
10.7955
10.7008
10.6061
10.5114
10.4167
10.322
10.2273
10.1326
10.0379
9.9432
9.8485
9.7538
9.6591
9.5644
9.4697
Rancho Matilija
CANAL
ROAD
RICE
GRANT
BDY
LA LUNA
Mirror Lake
Trailer Park
AVE
BM 629
Water Tank
Reservoir
Well
Water Tanks
BURNHAM
VENTURA
Feraud Park
A
Live Oak Acres
Devils Gulch
BM 598
N
S
E
W
0 500 1,000 2,000 3,000 4,000
Feet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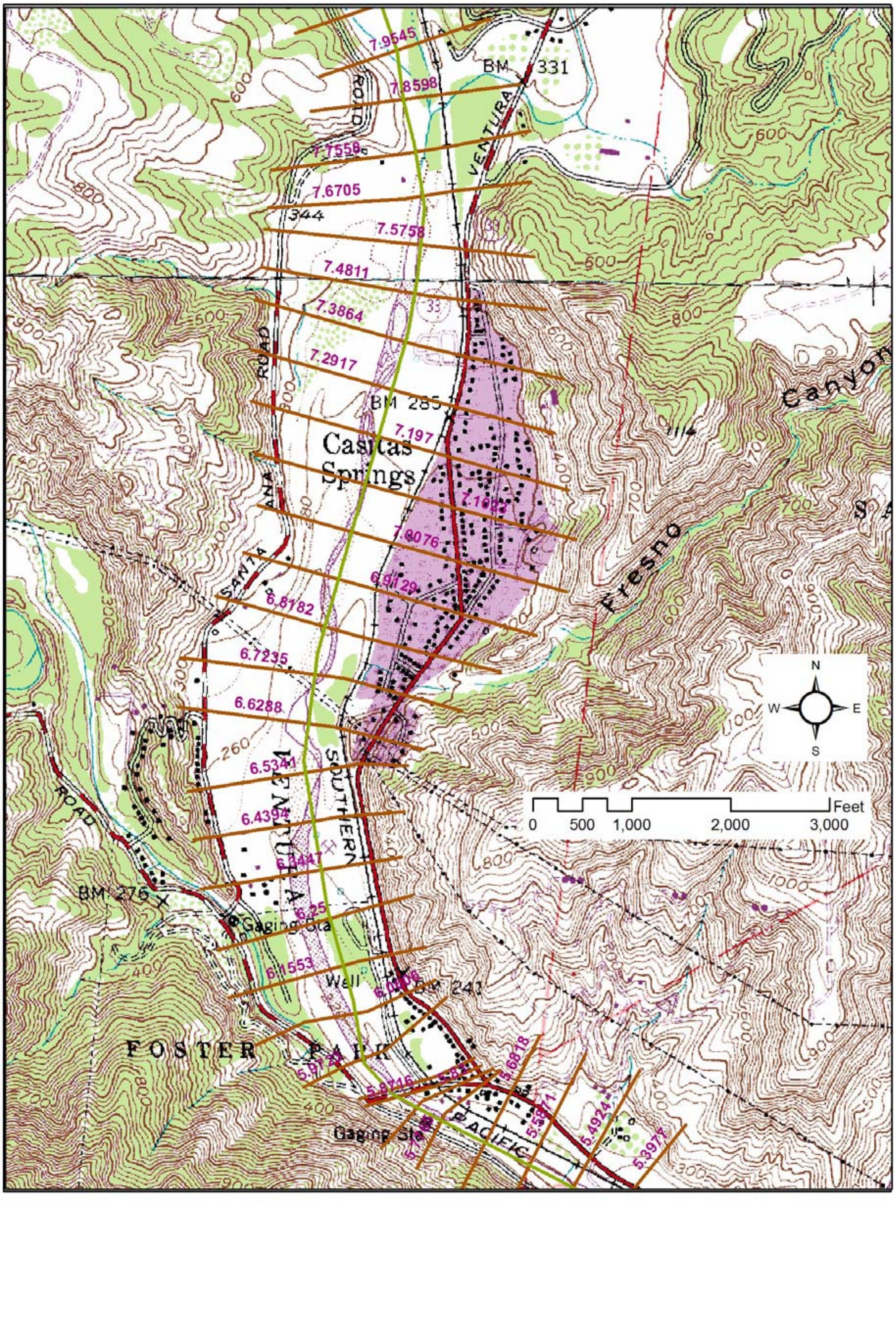
9.1856
9.0909
8.9962
8.9015
8.8068
8.7121
8.6174
8.5227
8.428
8.3333
8.2386
8.1439
8.0492
7.9545
7.8598
Live Oak Acres
Oak View
Devils Gulch
Footbridge
Water Tanks
Ventura
River
BLVD
PO
BM 598
BM 511
BM 331
Tank
WT
Well
SANTA ANA
559
600
800
400
520
480
N
W
E
S
Feet
0
500
1,000
2,000
3,000



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7.7559
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7.4811
7.3864
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7.197
7.1023
7.0076
6.9129
6.8182
6.7235
6.6288
6.5341
6.4394
6.3447
6.25
6.1553
Casitas Springs
FOSTER PARK
Gaging Sta
Well
BM 331
BM 285
BM 276
BM 241
VENTURA
SANTA ANA ROAD
SOUTHERN PACIFIC
Fresno
Canyon
N
S
E
W
Feet
0
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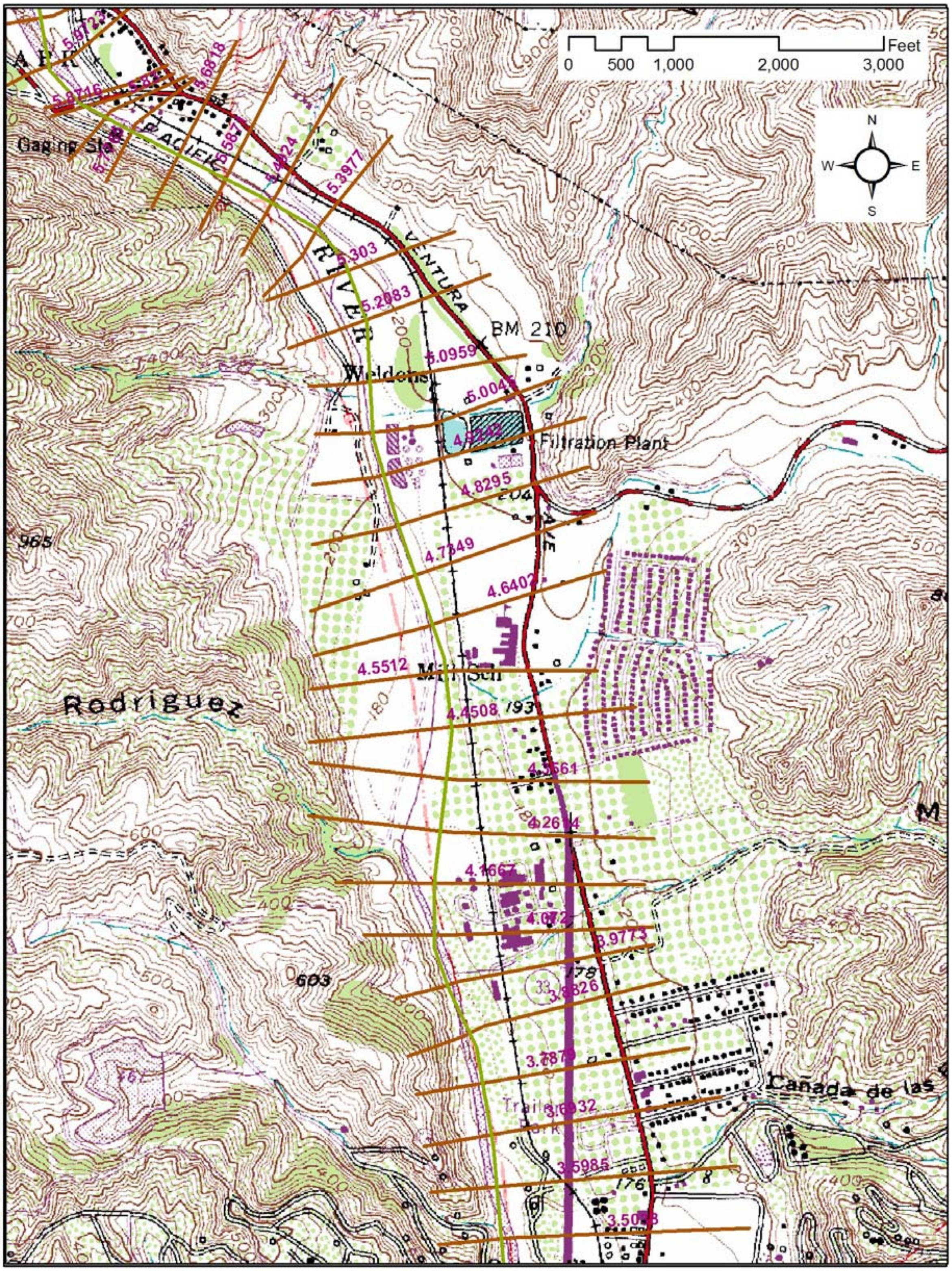
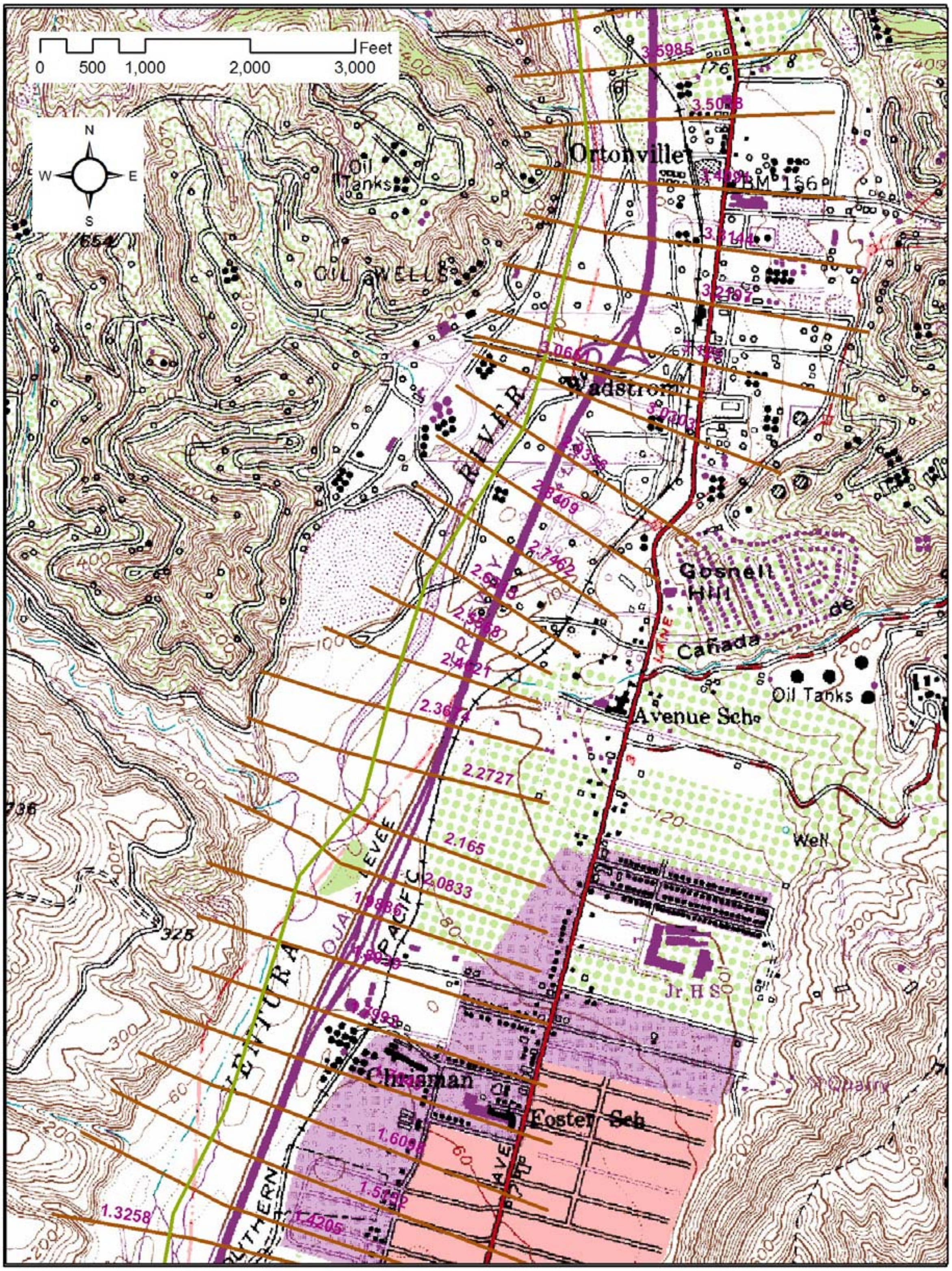
1,000
2,000
3,000



0 500 1,000 2,000 3,000 Feet
N
W
E
S
Gaging Sta
PACIFIC
RIVER
VENTURA
BM 210
Weldons
Filtration Plant
5.303
5.2083
4.8295
4.7349
4.6402
4.5512
4.4508
3.9773
3.5985
965
Rodriguez
603
Cañada de las



Feet
0 500 1,000 2,000 3,000
N
W
E
S
Ortonville
Oil Tanks
OIL WELLS
Wadstrom
RIVER
Gosnell Hill
Cañada
Oil Tanks
Avenue Sch
Well
LEVEE
PACIFIC
VENTURA
SOUTHERN
Jr. H S
Chrisman
Foster Sch
3.5985
3.5088
3.2144
3.0604
3.0303
2.8409
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2.2727
2.165
2.0833
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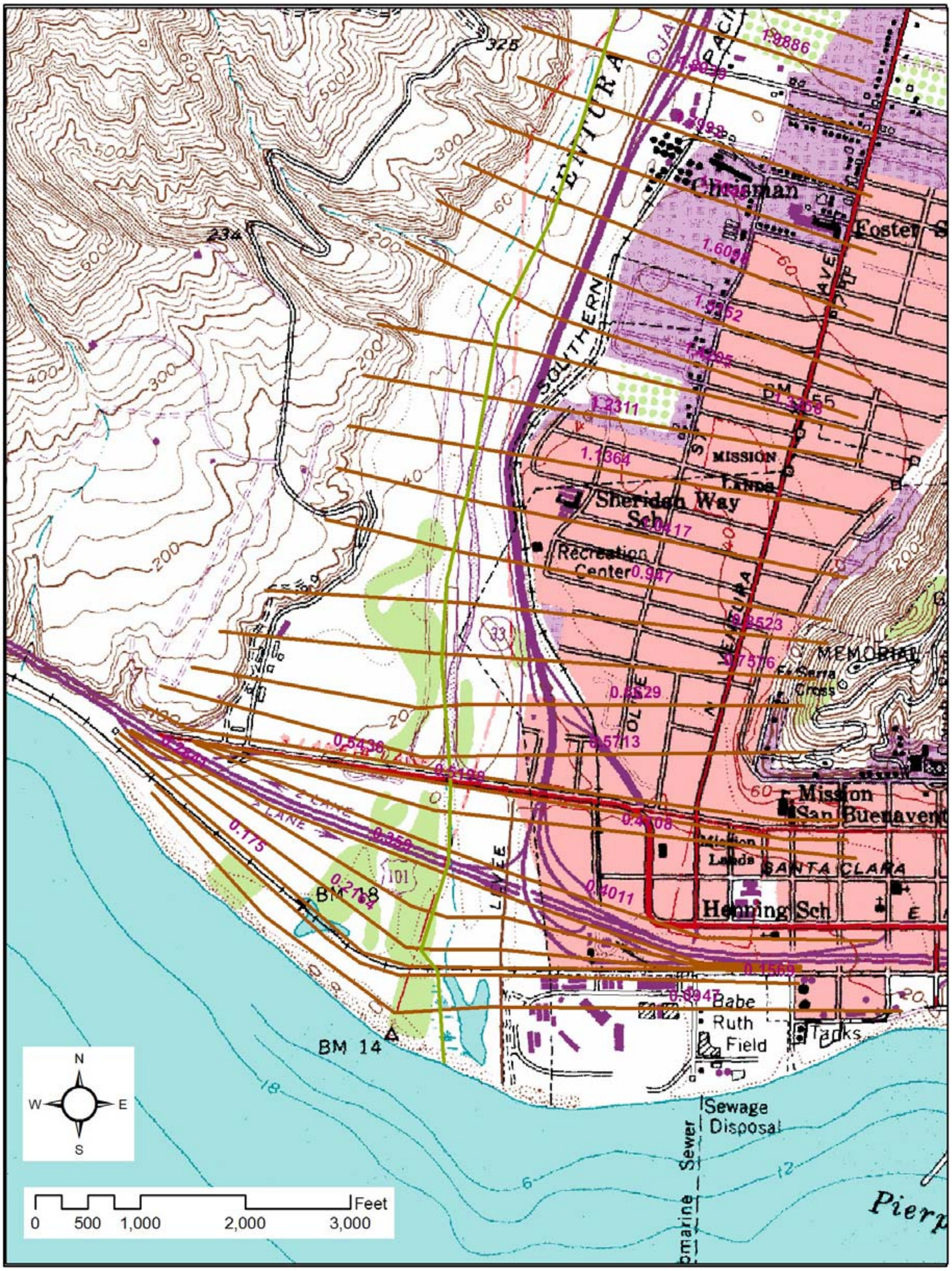
Chrisman
Foster
MISSION
Sheridan Way Sch
Recreation Center
MEMORIAL
Mission San Buenaventura
SANTA CLARA
Henning Sch
LEVEE
SOUTHERN PACIFIC
VENTURA
BM 18
BM 14
Babe Ruth Field
Tanks
Sewage Disposal
Submarine Sewer
Pierp
N
S
E
W
0
500
1,000
2,000
3,000
Feet

18. Exhibit D. River Changes from 2001 to 2005

19. Exhibit E. Water Surface Elevations and Uncertainties

Table 19.1. Station Information at Index Locations.

				Levee Height		Bank Elevation (ft)			Flow at Given Return Period							
Description	HEC-RAS Station	Reach	Equiv Record	Left	Right	Left	Right	New Levee	2	5	10	20	50	100	200	500
Hot Springs Area	16.0984	6l	73	NA	NA	967	1040	F	3060	7090	12500	15200	18800	21600	23000	27900
Hot Springs Area	16.054	6k	73	NA	NA	962	1040	F	3060	7090	12500	15200	18800	21600	23000	27900
Hot Springs Area	16.0038	6j	73	NA	NA	962	1040	F	3060	7090	12500	15200	18800	21600	23000	27900
Camino Cielo Area	15.625	6i	68	NA	NA	909	906	F	3250	7580	15000	18800	24000	27100	28900	35200
Camino Cielo Area	15.5303	6h	68	NA	NA	915	901	F	3250	7580	15000	18800	24000	27100	28900	35200
Camino Cielo Area	15.436	6g	68	NA	NA	890	889	F	3250	7580	15000	18800	24000	27100	28900	35200
Camino Cielo Area	15.3409	6f	68	NA	NA	882	882	F	3250	7580	15000	18800	24000	27100	28900	35200
Camino Cielo Area	15.2462	6e	68	NA	NA	888	876	F	3250	7580	15000	18800	24000	27100	28900	35200
D/S of Camino Cielo	15.1901	6d	68	NA	NA	865	863	F	3250	7580	15000	18800	24000	27100	28900	35200
D/S of Camino Cielo	15.1515	6c	68	NA	NA	862	859	F	3250	7580	15000	18800	24000	27100	28900	35200

D/S of Camino Cielo	15.0568	6b	68	NA	NA	849	850	F	3250	7580	15000	18800	24000	27100	28900	35200
Reach Index Locations	14.678	6a	68	NA	NA	900	816	F	3250	7580	15000	18800	24000	27100	28900	35200
Mieners Oak	13.8258	5d	68	NA	NA	758	758	T	3250	7580	15000	18800	24000	27100	28900	35200
Mieners Oak	13.6364	5c	68	NA	NA	738	741	T	3250	7580	15000	18800	24000	27100	28900	35200
Mieners Oak	13.447	5b	68	NA	NA	727	723	T	3250	7580	15000	18800	24000	27100	28900	35200
Reach Index Locations	12.5	5a	68	NA	NA	650	642	F	3250	7580	15000	18800	24000	27100	28900	35200
Reach Index Locations	10.8902	4h	68	NA	NA	545	524	F	3380	7910	16000	19800	24800	28300	30200	36700
Live Oak	10.1326	4g	68	NA	465.8	590	465.9	T	3380	7910	16000	19800	24800	28300	30200	36700
Live Oak	9.9432	4f	68	NA	450.6	590	450.6	T	3380	7910	16000	19800	24800	28300	30200	36700
Live Oak	9.7538	4e	68	NA	440.5	590	444.6	T	3380	7910	16000	19800	24800	28300	30200	36700
Live Oak	9.5644	4d	68	NA	427	590	427	T	3380	7910	16000	19800	24800	28300	30200	36700
Live Oak	9.375	4c	68	NA	415.8	590	421.6	T	3380	7910	16000	19800	24800	28300	30200	36700
Live Oak	9.2507	4b	68	NA	413	590	413	T	3380	7910	16000	19800	24800	28300	30200	36700
Reach Index Locations	8.7121	4a	68	NA	NA	520	388	F	3380	7910	16000	19800	24800	28300	30200	36700
Casitas Springs Levee	7.5758	3f	68	307.7	NA	307.7	302	T	4130	9820	35200	44400	56600	66600	71600	89000
Casitas Springs Levee	7.3864	3e	68	297.2	NA	297.2	292	T	4130	9820	35200	44400	56600	66600	71600	89000
Casitas Springs Levee	7.197	3d	68	288.3	NA	288.3	283	T	4130	9820	35200	44400	56600	66600	71600	89000
Casitas	7.0076	3c	68	278.0	NA	280.0	320	T	4130	9820	35200	44400	56600	66600	71600	89000

Springs Levee																	
Casitas Springs Levee	6.8182	3b	68	272.8	NA	272.8	320	T	4130	9820	35200	44400	56600	66600	71600	89000
Reach Index Locations	6.25	3a	68	NA	NA	244	246	F	4520	11060	36400	46400	59700	69700	74900	93100
Reach Index Locations	5.3977	2e	68	NA	NA	213	207	F	4520	11060	36400	46400	59700	69700	74900	93100
Ojai Valley Sanitation Disctrict	5.0045	2d	68	205.8	NA	206	230	F	4520	11060	36400	46400	59700	69700	74900	93100
Ojai Valley Sanitation Disctrict	4.9242	2c	68	208.5	NA	209	230	F	4520	11060	36400	46400	59700	69700	74900	93100
Reach Index Locations	3.5985	2b	68	NA	NA	142	340	F	5080	12250	41300	52700	67900	78900	84800	105500
Reach Index Locations	1.3258	2a	68	63.2	NA	46	46	F	5080	12250	41300	52700	67900	78900	84800	105500
Reach Index Locations	0.2841	1	68	24.7	NA	15	18	F	5080	12250	41300	52700	67900	78900	84800	105500

Table 19.2. Current Conditions Water Surface Elevations (ft)

	Water Surface Elevation (ft) at given Return Period (yr)							
HEC-RAS Stationing	2	5	10	20	50	100	200	500
16.0984	962.4	965.0	968.9	970.4	971.6	972.4	972.9	974.1
16.054	955.1	959.3	962.0	963.6	965.7	966.5	966.6	968.2
16.0038	945.5	948.8	951.7	953.1	954.9	956.2	956.8	958.8
15.625	902.3	904.2	906.9	908.1	909.5	910.3	910.7	912.1

15.5303	889.6	892.5	894.7	895.8	896.8	897.4	897.7	898.9
15.436	880.1	882.7	885.4	886.5	887.7	888.3	888.6	890.4
15.3409	872.8	874.8	877.2	878.2	879.4	880.1	880.5	881.7
15.2462	863.2	865.4	867.6	868.5	869.4	870.2	870.5	871.5
15.1901	857.8	859.9	862.1	862.8	863.9	864.3	864.5	865.2
15.1515	853.4	856.0	858.1	859.0	859.6	860.1	860.4	861.1
15.0568	843.2	845.3	847.9	848.8	850.5	851.0	851.2	852.0
14.678	809.5	811.9	814.6	815.6	817.4	818.3	818.9	819.8
13.8258	741.5	743.4	745.6	746.5	747.6	748.3	748.6	749.8
13.6364	730.0	732.0	733.5	734.5	735.7	736.4	736.8	738.1
13.447	715.7	718.1	719.4	720.0	720.8	721.2	721.5	722.3
12.5	638.3	639.3	640.2	640.7	641.2	641.5	641.6	642.1
10.8902	511.2	512.5	513.8	514.3	515.0	515.4	515.5	516.1
10.1326	458.2	459.5	460.7	461.1	461.7	462.0	462.2	462.7
9.9432	444.3	445.7	447.2	447.7	448.3	448.6	448.8	449.3
9.7538	431.2	432.6	434.3	434.7	435.2	435.6	435.7	436.2
9.5644	418.0	419.9	422.2	423.0	424.1	424.8	425.2	426.6
9.375	409.0	410.4	411.9	412.6	413.4	414.0	414.3	415.9
9.2507	400.1	402.3	405.2	406.3	408.2	409.7	410.4	413.5
8.7121	364.0	365.0	366.5	367.1	367.7	368.1	368.2	368.7
7.5758	294.3	296.2	300.0	300.7	301.6	302.1	303.1	304.3
7.3864	287.2	288.6	292.1	293.1	294.2	295.0	295.4	296.5
7.197	279.8	280.9	283.4	283.9	284.6	285.1	285.4	286.3
7.0076	268.5	269.7	273.0	273.8	274.7	275.4	275.7	276.8
6.8182	259.5	261.7	264.9	265.6	266.5	267.2	267.5	268.6
6.25	235.5	237.5	240.7	241.8	242.8	243.5	243.5	244.5
5.3977	201.0	202.9	206.0	206.8	208.5	209.6	210.1	213.9
5.0045	182.1	186.0	195.2	198.0	201.7	204.1	205.4	209.1
4.9242	179.6	183.0	190.2	192.2	194.1	195.9	196.6	200.0
3.5985	128.3	131.1	137.4	139.2	141.4	142.8	143.4	146.2
1.3258	38.8	41.5	46.7	48.2	50.2	51.6	52.3	54.7
0.2841	8.9	11.4	16.0	16.8	17.8	18.4	19.3	19.8

Table 19.3. Future With-Project Conditions Water Surface Elevations (Ft) and Standard Deviations of Water Surface Elevations.

	Water Surface Elevation (ft) at given Return Period (yr)								Standard Deviaiton of Estimate (ft) at given Return Period (yr)							
Station	2	5	10	20	50	100	200	500	2	5	10	20	50	100	200	500
16.0984	962.9	966.4	970.6	972.0	975.3	976.7	977.9	978.7	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
16.054	954.6	958.2	962.1	963.8	965.9	967.4	968.3	974.8	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
16.0038	945.2	948.8	952.3	953.8	955.3	956.5	957.2	959.4	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
15.625	901.6	904.0	906.4	907.9	909.4	910.2	910.7	912.1	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
15.5303	889.8	892.0	894.9	896.1	897.4	898.1	898.5	899.8	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
15.436	885.6	888.2	890.8	891.9	893.1	893.8	894.2	895.4	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
15.3409	878.0	879.6	881.7	882.0	883.6	884.2	884.6	885.8	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
15.2462	868.2	871.6	873.2	873.8	874.3	874.8	875.0	875.8	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
15.1901	863.7	865.8	867.8	868.4	869.2	869.4	869.6	870.2	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
15.1515	860.5	862.5	864.1	864.4	865.0	865.2	865.4	866.1	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
15.0568	852.1	853.8	855.5	856.2	857.0	857.4	857.6	858.2	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
14.678	817.9	820.4	822.7	823.3	824.1	824.5	824.7	825.4	3.27	3.04	2.98	3.08	2.96	2.87	2.80	2.67
13.8258	751.7	753.2	755.0	755.7	756.6	757.1	757.3	758.0	3.49	3.29	3.20	3.16	3.14	3.14	3.11	3.04
13.6364	734.5	735.9	737.7	738.6	739.4	740.1	740.5	741.9	3.49	3.29	3.20	3.16	3.14	3.14	3.11	3.04
13.447	717.1	720.0	721.4	722.0	722.8	723.3	723.5	724.4	3.49	3.29	3.20	3.16	3.14	3.14	3.11	3.04
12.5	638.1	638.8	639.6	640.0	640.5	640.8	641.0	641.5	3.49	3.29	3.20	3.16	3.14	3.14	3.11	3.04
10.8902	513.7	514.4	515.3	515.7	516.2	516.5	516.7	517.3	1.89	1.73	1.42	1.45	1.40	1.39	1.39	1.41
10.1326	458.6	460.7	462.9	463.4	463.9	464.2	464.4	465.0	1.89	1.73	1.42	1.45	1.40	1.39	1.39	1.41
9.9432	447.9	448.4	449.3	449.7	450.1	450.5	450.6	451.2	1.89	1.73	1.42	1.45	1.40	1.39	1.39	1.41
9.7538	432.4	434.2	435.5	435.8	436.3	436.6	436.7	437.2	1.89	1.73	1.42	1.45	1.40	1.39	1.39	1.41
9.5644	420.7	421.8	423.3	423.9	424.7	425.2	425.5	426.4	1.89	1.73	1.42	1.45	1.40	1.39	1.39	1.41
9.375	407.5	409.2	411.5	412.4	413.4	414.1	414.5	415.7	1.89	1.73	1.42	1.45	1.40	1.39	1.39	1.41
9.2507	398.6	400.9	403.7	404.7	405.9	407.2	407.7	409.0	1.89	1.73	1.42	1.45	1.40	1.39	1.39	1.41
8.7121	366.6	368.6	369.2	369.4	369.6	369.8	369.9	370.3	1.89	1.73	1.42	1.45	1.40	1.39	1.39	1.41
7.5758	296.4	298.2	301.9	303.0	304.0	304.7	304.9	306.2	1.53	1.40	1.34	1.35	1.36	1.41	1.41	1.50
7.3864	288.4	289.2	292.4	293.4	294.4	295.2	295.5	296.5	1.53	1.40	1.34	1.35	1.36	1.41	1.41	1.50
7.197	277.6	279.7	283.1	283.6	284.3	284.8	285.1	285.9	1.53	1.40	1.34	1.35	1.36	1.41	1.41	1.50
7.0076	269.2	270.4	274.2	274.9	275.8	276.5	276.8	278.0	1.53	1.40	1.34	1.35	1.36	1.41	1.41	1.50
6.8182	260.3	262.0	265.7	266.4	267.3	268.0	268.4	269.4	1.53	1.40	1.34	1.35	1.36	1.41	1.41	1.50
6.25	232.8	235.7	240.1	240.9	241.8	242.4	242.7	244.9	1.53	1.40	1.34	1.35	1.36	1.41	1.41	1.50

5.3977	199.4	202.0	206.3	207.2	208.7	209.7	210.2	212.0	2.38	2.37	2.35	2.36	2.36	2.36	2.36	2.35
5.0045	181.0	183.7	191.6	194.0	197.1	199.4	200.6	204.5	2.38	2.37	2.35	2.36	2.36	2.36	2.36	2.35
4.9242	177.4	180.0	185.9	188.0	190.6	192.3	193.2	196.2	2.38	2.37	2.35	2.36	2.36	2.36	2.36	2.35
3.5985	125.6	128.1	133.9	135.8	137.7	139.0	139.7	141.7	2.38	2.37	2.35	2.36	2.36	2.36	2.36	2.35
1.3258	41.6	44.3	49.7	51.4	53.2	54.4	55.0	56.9	2.38	2.37	2.35	2.36	2.36	2.36	2.36	2.35
0.2841	12.2	14.0	16.9	17.5	18.2	18.8	19.2	21.0	1.40	3.56	3.52	3.51	3.50	3.50	3.50	3.50

Table 19.4. Future Without-Project Conditions Water Surface Elevations (Ft) and Standard Deviations of Water Surface Elevations.

	Water Surface Elevation (ft) at given Return Period (yr)								Standard Deviaiton of Estimate (ft) at given Return Period (yr)							
Station	2	5	10	20	50	100	200	500	2	5	10	20	50	100	200	500
16.0984	962.5	965.0	968.2	969.8	971.3	972.1	972.6	973.9	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
16.054	954.7	958.9	962.1	964.4	965.8	966.6	967.0	968.4	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
16.0038	945.3	948.5	951.2	952.7	954.5	955.8	956.4	958.5	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
15.625	901.3	903.6	905.9	907.5	909.0	909.8	910.3	911.7	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
15.5303	887.9	890.2	892.8	894.0	895.5	896.2	896.6	897.9	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
15.436	879.7	883.2	887.6	888.7	889.8	890.5	891.0	892.3	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
15.3409	872.2	873.9	875.8	876.7	877.6	878.2	878.6	879.9	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
15.2462	861.0	863.7	866.6	867.8	868.3	869.1	869.3	869.7	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
15.1901	855.5	857.8	860.5	861.2	862.9	863.1	863.6	865.0	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
15.1515	852.0	853.8	855.7	856.6	858.0	858.8	859.5	860.4	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
15.0568	841.9	844.1	846.9	848.0	848.8	849.1	849.7	850.8	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
14.678	812.8	815.1	817.3	818.3	819.4	820.0	820.3	821.2	3.20	3.04	2.74	2.49	2.22	2.24	2.31	2.11
13.8258	740.3	743.0	745.2	746.1	747.2	747.8	748.2	749.4	2.29	2.15	1.69	1.46	1.30	1.11	1.07	0.85
13.6364	725.4	727.5	730.2	731.3	732.5	733.1	733.5	734.8	2.29	2.15	1.69	1.46	1.30	1.11	1.07	0.85
13.447	706.9	709.5	712.7	714.2	715.6	718.7	719.0	719.9	2.29	2.15	1.69	1.46	1.30	1.11	1.07	0.85
12.5	635.3	636.1	637.3	637.8	638.5	639.0	639.1	639.7	2.29	2.15	1.69	1.46	1.30	1.11	1.07	0.85
10.8902	512.0	513.3	514.2	514.6	515.1	515.4	515.6	516.2	1.88	1.71	1.40	1.42	1.40	1.39	1.39	1.29
10.1326	457.6	459.6	461.9	462.4	462.9	463.2	463.4	463.9	1.88	1.71	1.40	1.42	1.40	1.39	1.39	1.29
9.9432	445.8	446.9	447.8	448.2	448.7	449.0	449.1	449.7	1.88	1.71	1.40	1.42	1.40	1.39	1.39	1.29
9.7538	428.8	431.1	433.3	434.2	434.9	435.2	435.4	436.0	1.88	1.71	1.40	1.42	1.40	1.39	1.39	1.29
9.5644	416.5	417.9	419.4	420.1	420.9	421.4	421.7	422.7	1.88	1.71	1.40	1.42	1.40	1.39	1.39	1.29
9.375	401.6	404.1	407.7	409.2	411.1	412.4	413.1	415.6	1.88	1.71	1.40	1.42	1.40	1.39	1.39	1.29

9.2507	399.8	401.9	404.9	406.1	408.3	409.7	410.5	413.4	1.88	1.71	1.40	1.42	1.40	1.39	1.39	1.29
8.7121	365.8	367.2	368.0	368.2	368.5	368.7	368.9	369.2	1.88	1.71	1.40	1.42	1.40	1.39	1.39	1.29
7.5758	296.1	297.9	301.5	302.4	303.7	304.4	304.7	305.9	0.95	1.02	1.04	1.00	1.00	1.07	1.07	1.26
7.3864	288.3	289.2	292.3	293.3	294.3	295.1	295.4	296.6	0.95	1.02	1.04	1.00	1.00	1.07	1.07	1.26
7.197	277.3	279.3	282.8	283.4	284.0	284.6	284.8	285.6	0.95	1.02	1.04	1.00	1.00	1.07	1.07	1.26
7.0076	268.7	270.1	273.9	274.6	275.5	276.2	276.6	277.7	0.95	1.02	1.04	1.00	1.00	1.07	1.07	1.26
6.8182	260.0	262.0	265.3	266.0	266.9	267.6	267.9	269.0	0.95	1.02	1.04	1.00	1.00	1.07	1.07	1.26
6.25	232.4	235.3	239.7	240.6	241.4	242.0	242.4	244.5	0.95	1.02	1.04	1.00	1.00	1.07	1.07	1.26
5.3977	199.1	201.8	206.3	207.2	208.6	209.7	210.2	212.0	0.81	0.80	0.71	0.70	0.69	0.68	0.68	0.64
5.0045	180.4	183.3	191.3	194.0	197.3	199.7	200.9	205.0	0.81	0.80	0.71	0.70	0.69	0.68	0.68	0.64
4.9242	176.9	179.6	185.8	187.7	190.3	192.2	193.1	196.2	0.81	0.80	0.71	0.70	0.69	0.68	0.68	0.64
3.5985	125.4	128.0	134.1	135.9	137.9	139.3	140.0	142.2	0.81	0.80	0.71	0.70	0.69	0.68	0.68	0.64
1.3258	40.9	43.5	49.0	50.7	52.6	53.8	54.4	56.4	0.81	0.80	0.71	0.70	0.69	0.68	0.68	0.64
0.2841	11.6	13.0	16.0	16.8	17.5	17.9	18.1	19.1	0.52	1.02	1.02	1.02	1.02	0.74	0.76	0.82

20. Exhibit F. Flood Mapping

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21. Exhibit G. River Bed Material

Table 21.1. Ventura River and Matilija Creek 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 21.2. 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 21.2 (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

22. Exhibit H. 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 22.1. 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

23. Exhibit I. Riprap Design for Live Oak Acres