RECLAMATION
Managing Water in the West

Hydrology, Hydraulics, and Sediment Studies for the Foster Park Well Design - DRAFT Report

Matilija Dam Ecosystem Restoration Project, Ventura, CA

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

May 2007

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Executive Summary

The Sedimentation and River Hydraulics Group of the Denver Technical Service Center US Bureau of Reclamation was requested by the Los Angeles District of the Army Corp of Engineers to complete a hydrology, hydraulics, and sedimentation study to support the design of two shallow groundwater wells located adjacent to the Ventura River, Ventura, CA. The wells are being constructed as part of the Matilija Dam Ecosystem Restoration Project to replace two existing surface water diversions for the City of Ventura.

The report gives the hydraulic conditions in the vicinity of the well and the water surface elevations for given flood events. Future trends in bed elevations and flood water surface elevations are also discussed. The report also contains information necessary to design rock protection for the wells.

The minimum bed elevations in the reach near the proposed well location have remained relatively stable since 1970 with some slight deposition of approximately 1 to 2 feet. Under future conditions, the minimum river bed elevations should remain relatively stable but with some fairly minor erosion of up to 2 to 3 feet.

The Ventura River in the reach adjacent to the wells is primarily a braided river. Therefore, channel evulsions and bank erosion are frequent. The main channel may shift from one side of the river corridor to the other side in a relatively short period of time. For example, the channel in 1978 was on the West side and now is located on the East side. In the future, it may once again be on the West. The groundwater study by Hopkins (2006) stated that the wells may not be able to produce the design goal of 700 gpm if the main channel is on the West side. Therefore, the City of Ventura should be aware that the future well production at these two locations is somewhat uncertain.

The predicted local scour at the well locations below the current thalweg elevations is approximately 7.5 feet. Assuming a side slope of 2 horizontal to 1 vertical and assuming that the rock is not grouted or cabled together, the median rock size necessary to protect the well heads from direct exposure to the flow is estimated to be 34 inches or about 1 ton.

Table of Contents

Table of Figures

Table of Tables

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1. Introduction

Two water supply wells will be constructed upstream of the Casitas Vista Road Bridge on the East bank of the Ventura River. The wells are being constructed as part of the Matilija Dam Restoration Project. The removal of Matilija Dam will adversely affect the operation of the current surface diversions and the wells will mitigate the project impacts. This report will detail the hydraulic and sediment transport analysis necessary to design the wells.

Two wells will be constructed upstream of the Foster Park Bridge between approximately RM 6.15 and 6.25 on the East Bank of the Ventura River. Figure 1 shows the approximate project location within the Ventura River Watershed and Figure 2 shows a smaller scale aerial photo with the approximate location of each well. The two well locations are shown as green dots and the historical channel migration zone from 1947 to 2001 is shown in red. Note that the river has eroded terrace material from 2001 to 2005 on the East side of the channel just upstream of the wells. It is possible that the bank erosion continues and the wells are in the direct path of the river in the near future. Therefore, it is critical to protect the well with sufficient size rock to prevent damage.

Throughout this document, the “Project” refers to the removal of Matilija Dam. Therefore, “Without-Project” refers to the conditions if Matilija Dam remains in place and “With-Project” refers to the conditions if Matilija Dam is removed.

All elevations in this report are given in NAVD 88 unless otherwise noted.

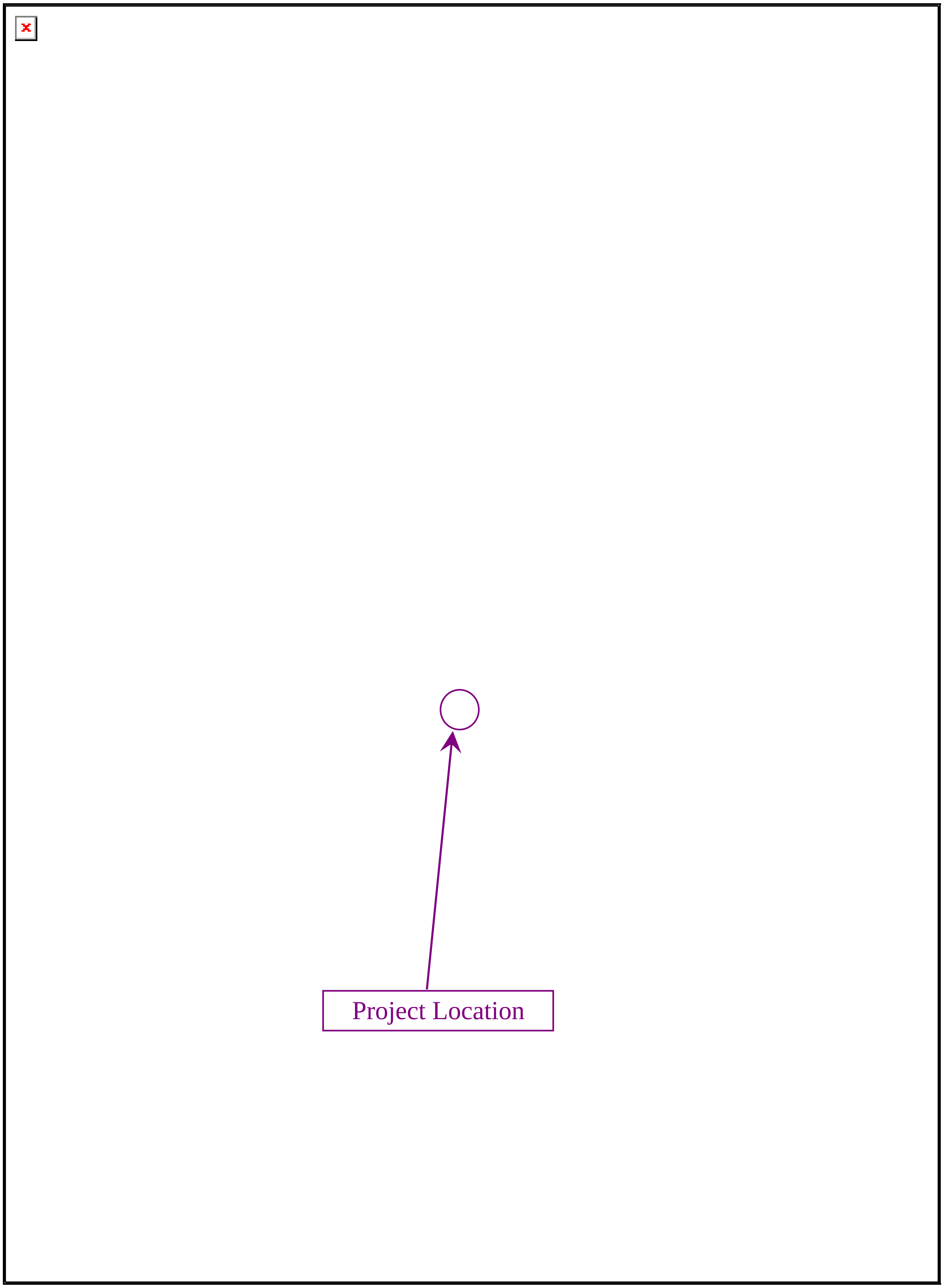

Figure 1. Project Location.

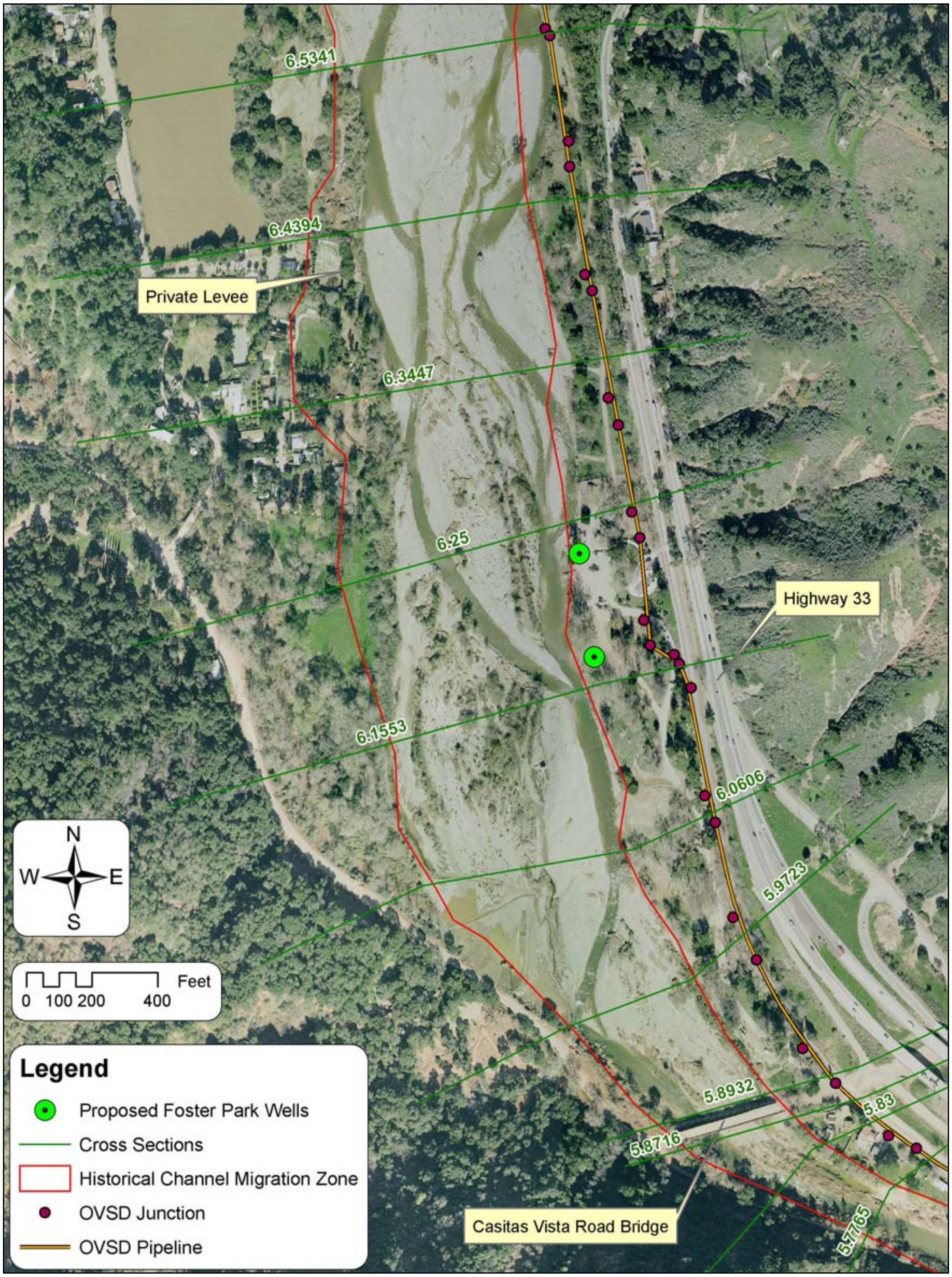

Figure 2. Approximate Location of Proposed Foster Park Wells.

2. Hydrology

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 3). 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.

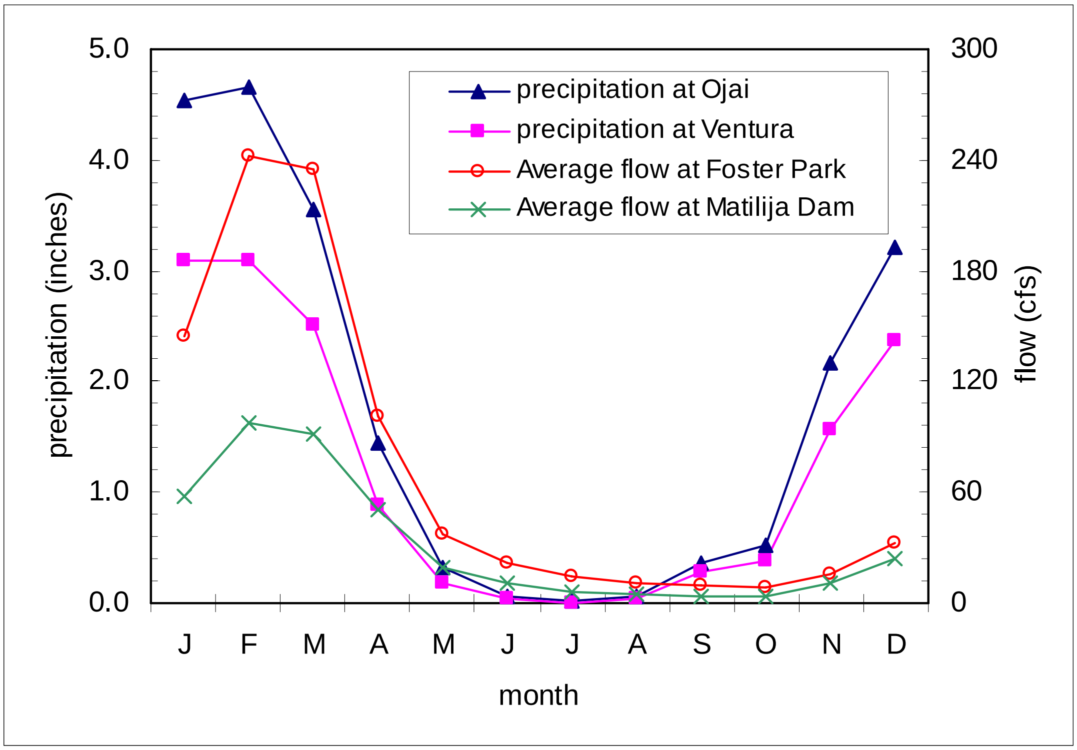

Figure 3. Seasonal variation of average rainfall and flow in Ventura River Watershed.

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 that 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). The results of the flood frequency analysis for the location nearest the well are given in Table 1.

The flow duration data is given in Table 2. The flow is below 3 cfs 50% of the time and there is no recordable flow at the gage more than 20% of the time.

Table 1. Design Flood Flows near Foster Park.

Return Period (yr)	Flood Flow at Casitas Road Bridge (cfs)
2	4,520
5	11,060
10	36,400
20	46,400
50	59,700
100	69,700
500	93,100

Table 2. Flow Duration Data based upon Daily Average Flows for Stream Gage Near the Casitas Vista Road Bridge (USGS gage #11118500).

Begin Year	1930			
End Year	2000			
Number of Years	71			
Drainage Area (mi^2)	188.0			
Gauge Datum (ft)	205.23			
% of time below	**Flow (cfs)**		**% of time below**	**Flow (cfs)**
0	0.00		94	140
10	0.0		95	189
20	0.0		96	275
30	0.3		97	410
40	1.2		98	609
50	3.0		99	1180
60	6.2		99.5	2100
70	11		99.7	3300
80	22		99.9	7130
90	63		99.95	10400
91	73		99.99	20000
92	88		100	22000
93	109			

Several structures affect the flow in the Ventura Watershed. Matilija Dam, impuonding Matilija Creek, was built in 1947 with an initial reservoir capacity of 7,018 ac-ft. 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% (Reclamation 2004). 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, located 2 miles downstream of Matilija Dam. When diversions are being performed at Robles Diversion Dam, the reservoir level is cycled to produce larger flows in the Ventura River, optimizing 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 combined maximum of 250 cfs.

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. 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.

3. Groundwater

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 4.

The Upper Ventura River (upstream of San Antonio Creek) is underlain by alluvial deposits with a maximum thickness of 200 feet and 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 5). Therefore, the groundwater beneath the Ventura River is divided into an upper cell and a 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 below Foster Park 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 value 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/yr from 1963 to 1973.

Entrix (2001) has prepared a report analyzing the surface-groundwater interactions. In that 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.

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 based upon the field samples collected at almost 20 sites along the Ventura River (See Reclamation 2006). 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.

Hopkins Groundwater Consultants, Inc. (2006) drilled 4 6-inch test holes to collect data necessary to estimate the future production rate of the proposed wells. The wells will be designed to produce at rate of 700 gpm. They stated: “The proposed well sites are believed capable of producing the desired production rate (or possibly at greater rates) when the surface flow of the river is located along the eastern side of the relatively wide active channel. However, we are advising the City that when the surface flow of the river is re-established on the western side of the channel the proposed wells may not be capable of sustaining the desired operational rate.”

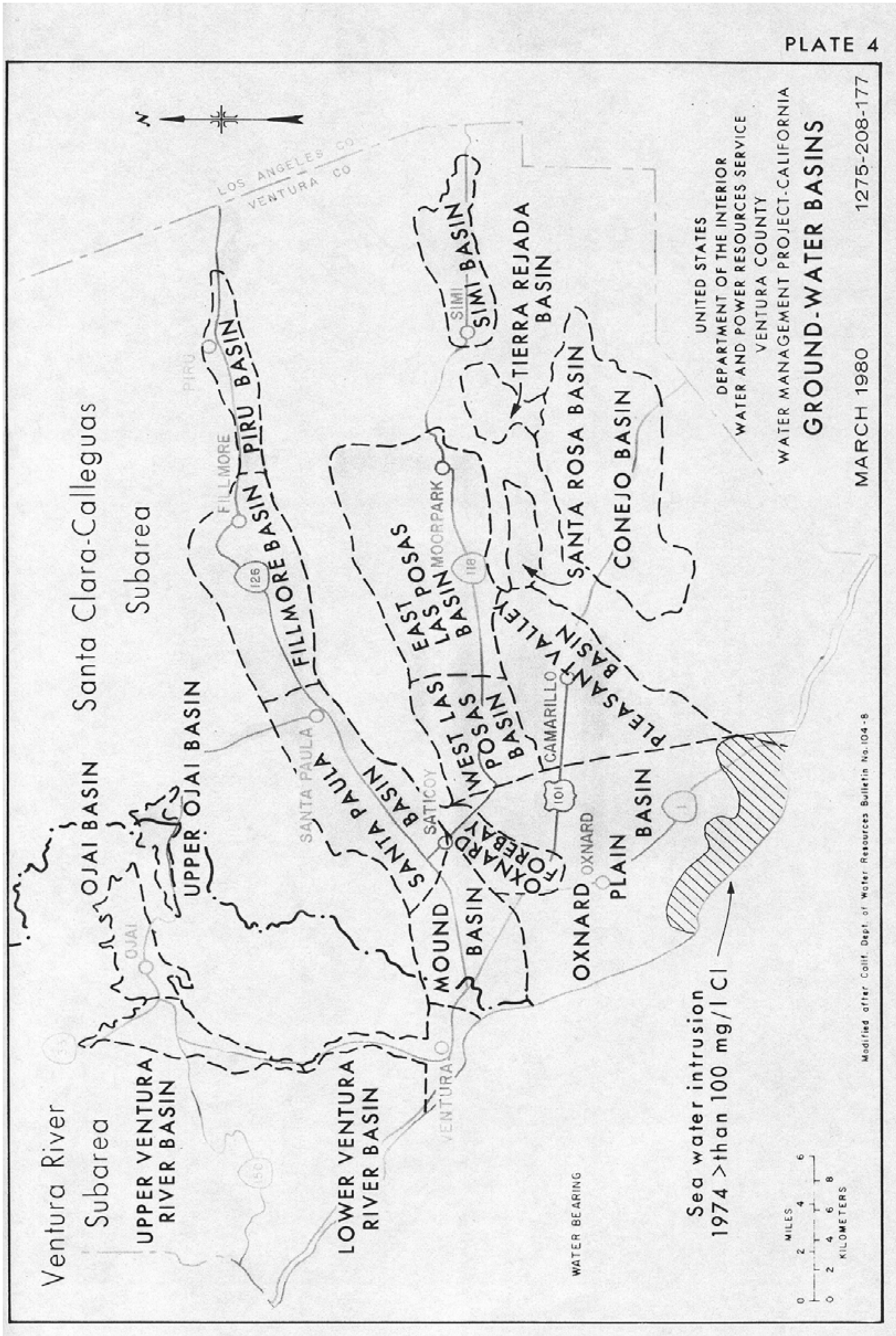

Figure 4. Map of groundwater basins in Ventura County. From Reclamation (1981).

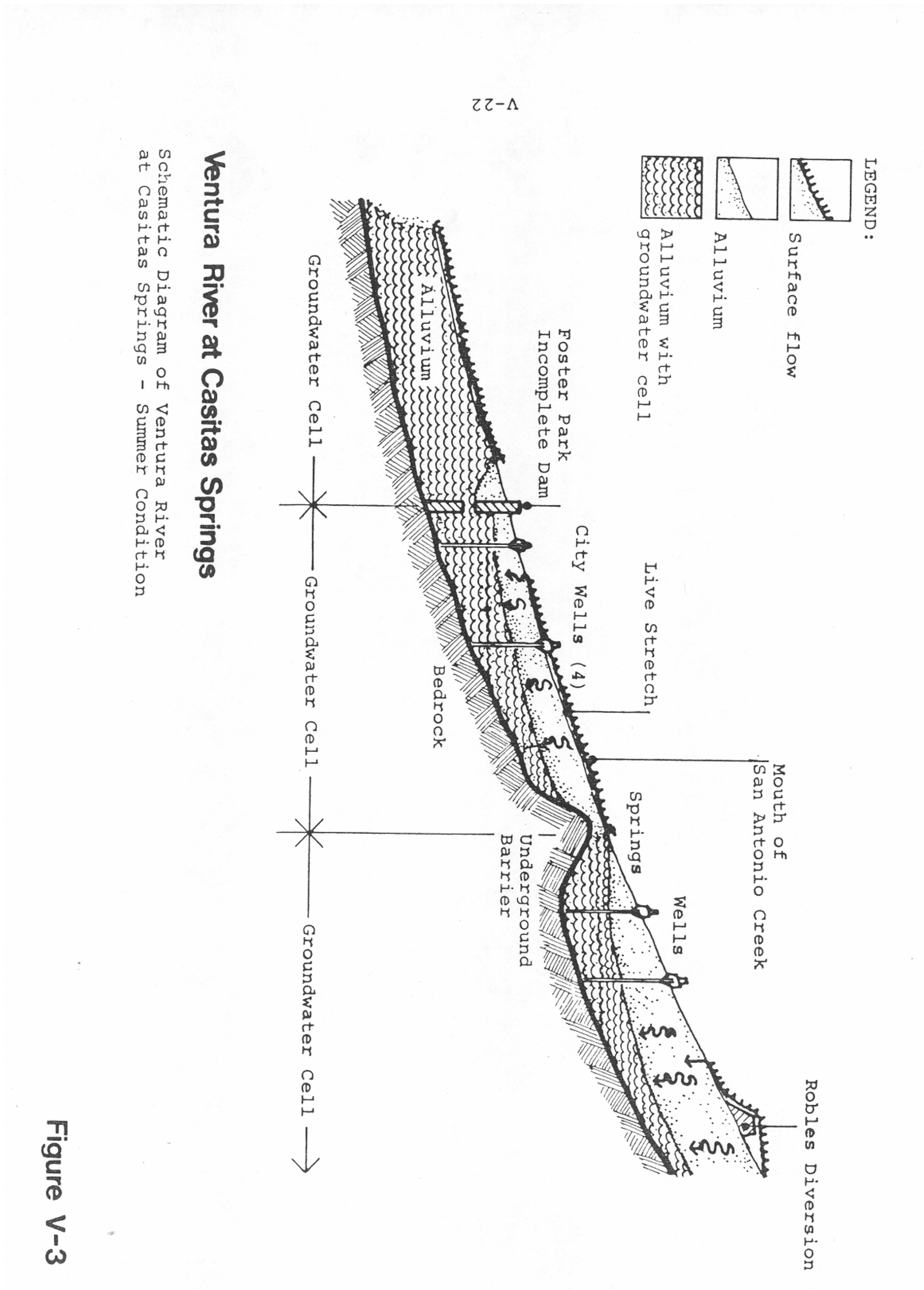

Figure 5. Schematic of groundwater basins below Ventura River (Turner, 1971).

4. Hydraulics

A detailed hydraulic study was performed by Reclamation (2006). The study used a LiDAR aerial survey performed by Airborne1 in March of 2005 as the base survey. A HEC-RAS 3.1.1 hydraulic model was generated using HEC-GeoRAS Ver 4.1. The hydraulic model was calibrated using high water marks from the 2005 flood. A hydraulic roughness of 0.04 was determined to be the best estimate for the hydraulic roughness using this data. The hydraulic information used here is identical to that reported in Reclamation (2006).

Flood inundation maps were also generated in Reclamation (2006). The flood boundaries in the project area are given in Appendix A for the 10-, 50-, 100-, and 500-yr flows. Three conditions are shown:

1) Current Conditions: The flood boundaries using the 2005 Aerial survey

2) Without-Project Future Conditions: The estimated flood boundaries 50-years in the future assuming that Matilija Dam remains in place for the next 50 years.

3) With-Project Future Conditions: The estimated flood boundaries 50-years in the future assuming that Matilija Dam is removed and the associated project features are in place.

For all conditions, the proposed Foster Park wells are located within the 10-yr floodplain. The floodplains for the With-Project Conditions are generally narrower from RM 6.63 to 6.34, and slightly wider from RM 6.34 to 5.77.

Plots of the cross section immediately downstream and upstream of the wells are shown in Figure 6 and Figure 7, respectively. The hydraulic data calculated from HEC-RAS for the 10-yr, 50-yr, 100-yr, and 500-yr flood is given in Table 3 through Table 6 for cross sections near the wells.

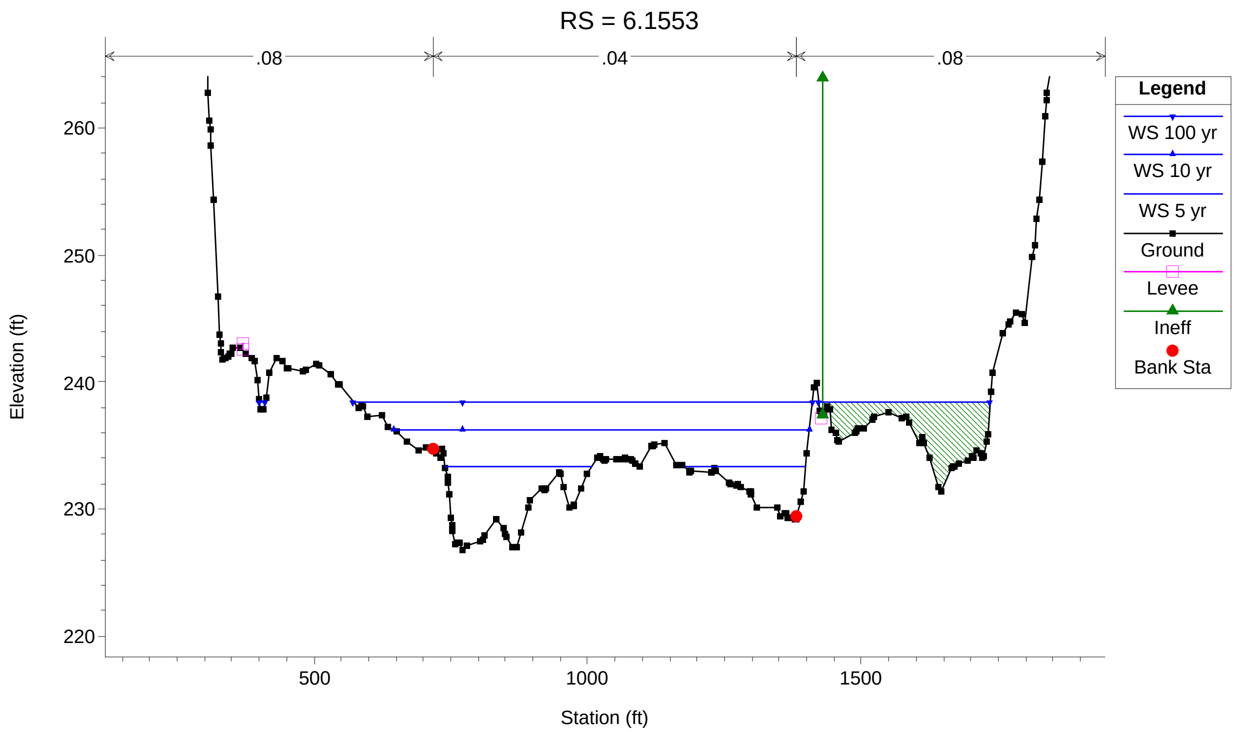

Figure 6. Cross section at RM 6.1553.

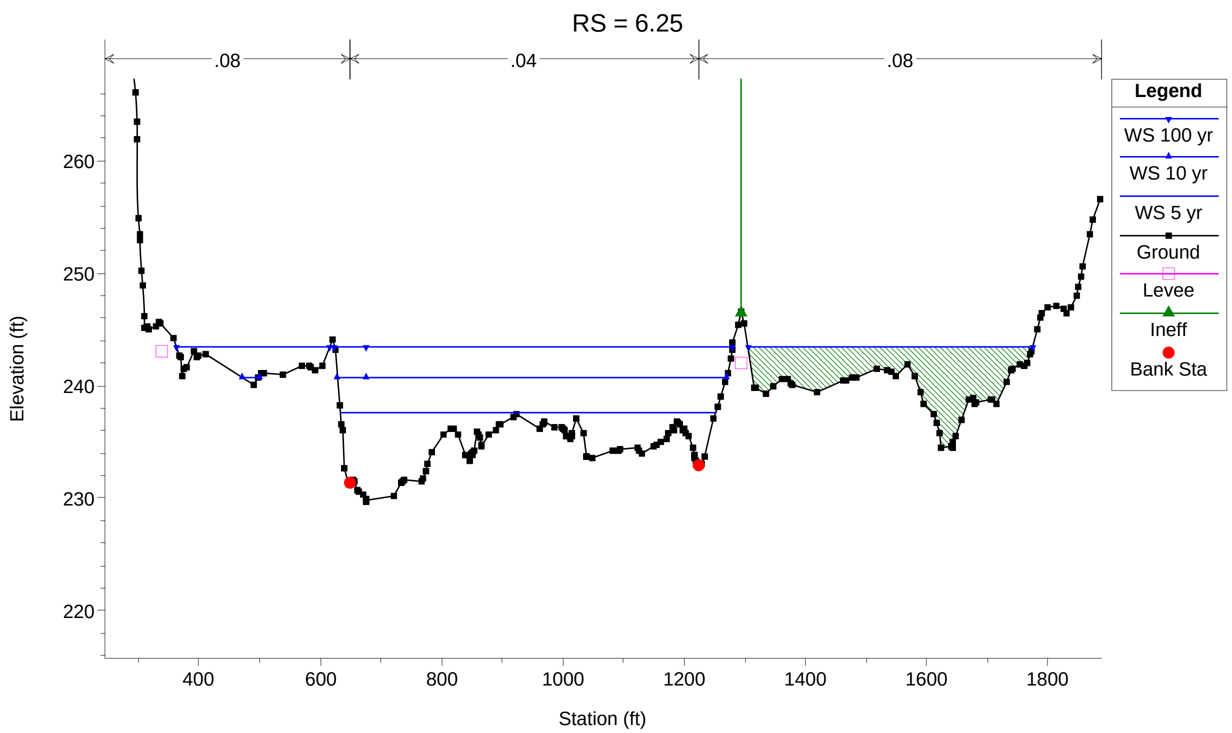

Figure 7. Cross section at RM 6.250.

Table 3. Hydraulic Data for Current Conditions 10-yr Flood.

RM	Return Period (yr)	Channel Discharge (ft3/s)	Thalweg elev (ft)	Thalweg Depth (ft)	Channel Velocity (ft/s)	Hydraulic Depth (ft)	Hydraulic Radius (ft)	Friction Slope (-)	Top Width (ft)
6.629	10	35171	246.8	9.28	10.83	5.48	5.39	0.006715	592.6
6.534	10	32296	244.5	8.78	8.82	5.94	4.84	0.006806	617
6.439	10	35052	241.4	7.61	11.25	5.69	4.59	0.009624	547
6.345	10	35963	235.1	8.89	11.55	5.42	5.21	0.007423	574
6.250	10	35160	229.7	11.04	9.56	6.40	5.92	0.007746	575
6.155	10	35763	226.7	9.55	11.22	4.79	4.40	0.009618	665
6.061	10	35296	222.6	9.13	10.25	5.30	4.57	0.006211	649
5.972	10	35461	215.7	13.02	10.56	8.34	7.62	0.002776	403
5.893	10	36199	210.9	17.05	8.60	13.04	11.87	0.002078	323
5.872	10	35171	246.8	9.28	10.83	5.48	5.39	0.006715	592.6
5.830	10	32296	244.5	8.78	8.82	5.94	4.84	0.006806	617

Table 4. Hydraulic Data for Current Conditions 50-yr Flood.

RM	Return Period (yr)	Channel Discharge (ft3/s)	Thalweg elev (ft)	Thalweg Depth (ft)	Channel Velocity (ft/s)	Hydraulic Depth (ft)	Hydraulic Radius (ft)	Friction Slope (-)	Top Width (ft)
6.629	50	56512	246.8	10.75	13.71	6.91	6.76	0.0065	596
6.534	50	49654	244.5	10.96	9.92	8.11	5.27	0.0061	617
6.439	50	56448	241.4	9.57	13.47	7.66	5.27	0.0094	547
6.345	50	58841	235.1	10.8	13.98	7.33	6.85	0.0075	574
6.250	50	57017	229.7	13.12	11.71	8.48	6.28	0.0081	575
6.155	50	58259	226.7	11.04	13.93	6.29	5.5	0.0055	665
6.061	50	56046	222.6	13.18	9.23	9.35	7.72	0.0026	649
5.972	50	55868	215.7	18.68	9.88	13.83	10.27	0.0017	409
5.893	50	58896	210.9	22.71	9.76	18.7	16.8	0.0017	323
5.872	50	58127	210.3	23.02	9.98	20.29	17.96	0.0020	287
5.830	50	55745	208.1	22.95	14.78	18.51	16.27	0.0041	204

Table 5. Hydraulic Data for Current Conditions 100-yr Flood.

RM	Return Period (yr)	Channel Discharge (ft3/s)	Thalweg elev (ft)	Thalweg Depth (ft)	Channel Velocity (ft/s)	Hydraulic Depth (ft)	Hydraulic Radius (ft)	Friction Slope (-)	Top Width (ft)
6.629	100	66473	246.8	11.42	14.71	7.57	7.36	0.006553	596.5
6.534	100	58348	244.5	11.75	10.62	8.91	6.02	0.005983	616.9
6.439	100	65198	241.4	10.37	14.08	8.46	5.68	0.009173	547.4
6.345	100	68613	235.1	11.49	14.89	8.03	7.26	0.007644	573.9
6.250	100	66089	229.7	13.81	12.54	9.17	6.84	0.008140	574.7
6.155	100	67737	226.7	11.72	14.62	6.96	5.95	0.004234	665.1
6.061	100	64248	222.6	15.18	8.71	11.35	9.43	0.001923	649.4
5.972	100	63651	215.7	20.91	9.69	16.04	12.35	0.001526	409.6
5.893	100	68557	210.9	24.80	10.22	20.79	18.56	0.002149	322.6
5.872	100	67612	210.3	24.94	10.60	22.20	19.61	0.002031	287.3
5.830	100	64815	208.1	24.59	15.79	20.15	17.63	0.004228	203.7

Table 6. Hydraulic Data for Current Conditions 500-yr Flood.

RM	Return Period (yr)	Channel Discharge (ft3/s)	Thalweg elev (ft)	Thalweg Depth (ft)	Channel Velocity (ft/s)	Hydraulic Depth (ft)	Hydraulic Radius (ft)	Friction Slope (-)	Top Width (ft)
6.629	500	88770	246.8	12.73	16.74	8.89	7.97	0.00667	597
6.534	500	74860	244.5	13.19	11.73	10.35	7.39	0.00552	617
6.439	500	82879	241.4	12.22	14.69	10.31	6.34	0.00824	547
6.345	500	91274	235.1	13.06	16.57	9.6	8.01	0.00855	574
6.250	500	87588	229.7	14.79	15.02	10.15	7.65	0.00529	575
6.155	500	87711	226.7	16.04	11.69	11.28	8.64	0.00187	665
6.061	500	82661	222.6	20.05	7.85	16.22	14.13	0.00110	649
5.972	500	81529	215.7	26.04	9.4	21.17	14.86	0.00092	410
5.893	500	77633	210.9	30.56	9.07	26.55	14.56	0.00182	323
5.872	500	73895	210.3	29.95	9.45	27.22	14.21	0.00149	287
5.830	500	85960	208.1	27.97	17.94	23.53	19.82	0.00467	204

5. Channel Morphology

The Ventura River morphology is described in more detail in Reclamation (2006). Using the reach designation from that report, the wells are located in Reach #3. This reach is a transition from the braided reaches upstream to the geologically confined reaches below Casitas Vista Road Bridge. Figure 2 shows the multiple channels upstream of the bridge converging to a single channel that is narrower and deeper.

The river channels in the reach upstream of RM 6 are active and frequently change location. Section 12 “Appendix B: Historical Aerial Photographs” contains the aerial photographs of the reach in 1947, 1970, 1978, 2001, and 2005. In 1947, the active channel was narrower with more woody vegetation. By 1970, the active channel was much wider and there was evidence of channel excavation and straightening. The 1969 flood was one of the largest floods on record and caused large amounts of bank erosion. The flood stripped most of the vegetation from the channel. The photo in 1978 shows the channel flowing almost full and there is a side channel that formed on the west side of the river near cross section 6.3447. In 2001, the channel is still mostly clear of vegetation. The channel has begun to erode the east bank near the proposed location of the wells. In 2005, the erosion near the wells has accelerated and the east bank just upstream of the upstream well is eroding. This erosion will likely continue in the future and would expose the well to river flows if left unprotected.

It is also possible that within one flood event, such as a 1969 or 1978 event, the river will destroy the private levee that protects the community on the West side of the river and the main channel will re-establish on the West side. Based upon Hopkins (2006) comments as referenced in Section 3, if main river channel is on the West Side of the river, the wells may not be able to produce 700 gpm. Therefore, the City of Ventura should be aware that the future well production at these two locations is somewhat uncertain. One alternative is to have wells on both sides of the river so that future well production is not as sensitive to channel location.

Figure 8. Picture Looking Across River from Downstream Well.

Figure 9. Picture looking Upstream from Upstream Well.

Figure 10. Picture Downstream of Proposed Well Locations Looking Upstream. Groins are Seen Along the Left Bank. The Concrete Structure is an Existing Well Operated by City of Ventura.

6. Sedimentation

Reclamation (2006) performed a detailed sedimentation analysis. Most information given here is a summary of the information contained in that report.

6.1. Current Conditions

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. The pebble count nearest the well location is given in Table 7.

Table 7. Pebble Count Gradation near RM 6.1.

% finer	Dia (mm)	Dia (ft)			Dia (mm)	Dia (ft)
4.8	2	0.007		**D_{16}**	25	0.082
5.1	4	0.013		**D_{50}**	79	0.258
5.2	8	0.026		**D_{84}**	132	0.434
7.1	16	0.052		**d_g**	2.3	2.3
18.2	32	0.105				
42.9	64	0.210				
74.7	128	0.420				
97.4	256	0.840				
100	360	1.181				

The concentration of suspended sediment during periods of relatively high flow has been sampled, more or less, continuously since 1968 at the USGS stream gage 11118500 at Casitas Vista Road Bridge. The data is reported in Reclamation (2006). Regression curves were fit to the clay and silt concentration and the sand concentration of the form:

$$C = aQ^b$$

where: C = Sediment concentration in mg/l
a, b = constants
Q = Flow rate (ft^3/s)

The results from the regression are given in Table 8. The total sediment concentration during flood flows is often above 10 g/l and sometimes as high as 20 g/l (1 to 2 % by mass), which is considered relatively high for natural rivers.

Table 8. Regression coefficients Fit to Suspended Sediment data

	Silt and Clay		Sand	
River	***a***	***b***	***a***	***b***
Ventura River	25	.608	0.009	1.37

Several rock groin structures were installed in 2005 in the Ventura on the east side of river downstream of the proposed well location 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.

The elevations in the reach have remained relatively stable since 1970. Figure 11 shows the change to the thalweg elevations from 1970 to 2001. From RM 7 to 6 there has been less than 2.5 feet of change. A difference of less than 2.5 feet is not considered significant because the accuracy of the 1970 survey is estimated to be +/– 2 feet and it was not possible to exactly locate the 1970 cross sections.

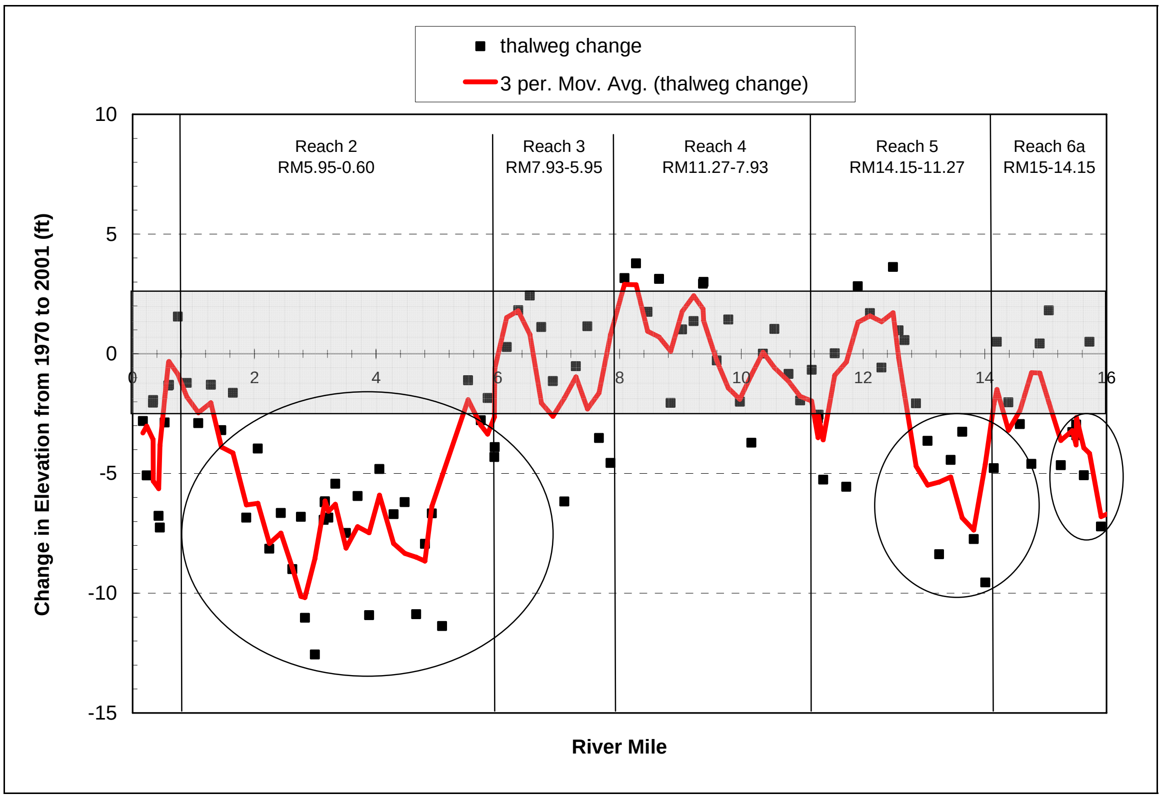

Figure 11. Comparison of change in thalweg elevation between 2001 and 1970. Negative changes indicate areas of degradation in the channel bed. Positive changes indicate areas that have aggraded. Areas within 2.5 feet of change are considered to be within the error range of the 1970 data.

6.2. Future Conditions

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.

Reclamation (2006) reports the results using several hydrological inputs. In this report, the results are derived from two representative hydrological scenarios: The 50-yr 1969 historical hydrograph and the 100-yr flood hydrograph. The 50-yr 1969 hydrograph was derived by using the historical record from 1969 to 2001 then appending the record from 1950 to 1968, for a total of 50 years of hydrologic record. The hydrologic record consisted of daily average flows that had to be modified during the peak flow events. A storm pattern was assumed and imposed on the daily average flow record while enforcing volume conservation.

The hydraulic input was taken from the HEC-RAS model described in Section 4. The hydraulic input includes the geometry data obtained from a 2005 LiDAR study. The same hydraulic roughness values were used in the GSTAR-1D model as in the HEC-RAS model. The sediment input consisted of bed material values throughout the entire river, and sediment loads from all major tributaries. All this data is described in Reclamation (2006).

The results from the modeling will only be described for the reach near the proposed location of the wells. Based upon the simulations, the thalweg in the reach adjacent to the Coyote Creek Levee from RM 6.4 to 6.1 will decrease in the range of 2 to 3 feet (Figure 12). In the immediate vicinity of Casitas Vista Road Bridge (RM 5.9 to 5.7), the thalweg may increase approximately 2 to 3 feet.

Figure 13 shows the predicted 100-yr water surface elevations under With- and Without-Project Conditions. The changes to the 100-yr water surface elevations generally follow the same trend as the thalweg changes, however, the magnitude of the change is generally less. From RM 6.4 to 6.2, the 100-yr water surface elevation is expected to decrease 1 to 2 feet. From RM 6.1 to 5.8, the 100-yr water surface elevation is expected to increase approximately 1 foot. These are both relatively small changes given the uncertainties in the sediment model.

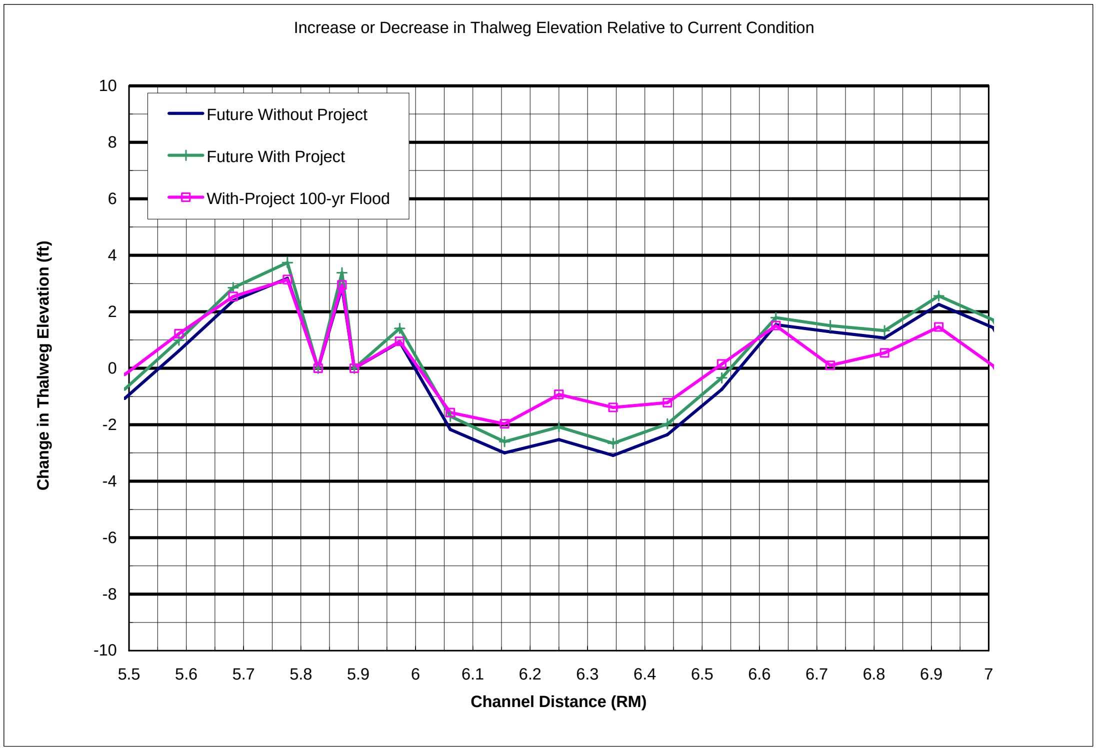

Figure 12. Change in Thalweg Elevation Relative to Current Condition.

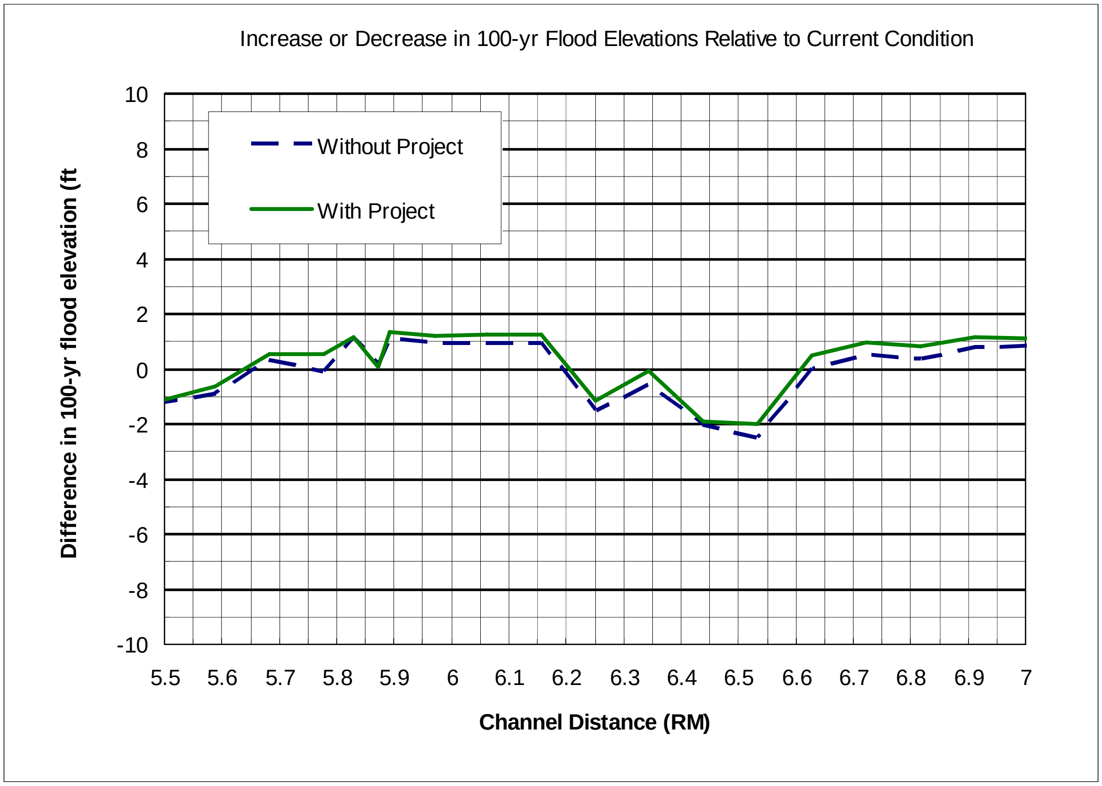

Figure 13. Change in 100-yr Flood Elevations Relative to Current Conditions.

7. Turbidity Impacts from Dam Removal

Currently, the diversion at Foster Park is a combination of surface diversion and subsurface wells. ENTRIX (1997) states that on average 2,500 ac-ft of surface water and 3,900 ac-ft of groundwater is diverted at Foster Park annually. The surface diversion is actually a combination of an above ground surface diversion and an intake that is approximately 4 feet below the riverbed. The subsurface wells are approximately 50 feet deep. The surface diversion decreased after 1993 and more water is now taken from the groundwater wells. The surface bed material at Foster Park is generally large cobbles with a small amount of sands. Therefore, the hydraulic conductivity of the bed material is very large and the subsurface diversions are not limited by the infiltration rates into the bed.

Because Foster Park is located approximately 10 miles from the dam, the present effect of Matilija Dam on the sediment loads there is small. Both North Fork Matilija Creek and San Antonio Creek enter the Ventura River between Matilija Dam and Foster Park. In addition, there is a large sediment supply from the banks of the riverbed between Robles Diversion and San Antonio Creek. Therefore, there are presently very high sediment concentrations that occur at Foster Park. The City of Ventura and the non-profit group Surfriders have collected turbidity samples at Foster Park (Figure 14).

The City of Ventura presently discontinues surface diversion when the turbidity rises above 10 NTU in the Ventura River. The data from flow duration curve was used along with Figure 15 to compute the total fraction of time the City of Ventura Water Treatment Plant cannot divert from its surface diversion. The computation is shown in Table 9. Under current conditions and for the average year, the surface diversion is shut down approximately 4.6% of the time, or about 17 days per year on average.

The City of Ventura provided the daily average flows for the period from 1984 until 2002 for the shallow intake and the period from 1991 until 2000 for the above ground surface diversion. The maximum recorded daily diversion at the shallow intake was 8.60 MGD (13.3 cfs), and 8.64 MGD (13.4 cfs) at the surface diversion. The average diversion for the shallow intake was 1.2 MGD (1.8 cfs) and was 1.8 MGD (2.9 cfs) for the surface diversion. To estimate the amount of water not diverted due to high turbidity, the 90th percentile of diversion flow was calculated. The 90th percentile was used as a representative diversion during the high flows that are linked to high turbidity. The 90th percentile diversion is 2.5 cfs for the shallow intake and 4.6 cfs for the surface diversion.

Table 9. Computation of Fraction of Time Turbidity would exceed 10 NTU under current conditions.

C1 Flow (cfs)	C2 % non-exceed	C3 fraction in flow bin	C4 Fraction of Readings exceeding 10 NTU	C5 Fraction of Time > 10 NTU (C3 * C4)
0 to 1	0	0.378	0.000	0.0000
1 to 5	37.8	0.185	0.005	0.0010
5 to 10	56.3	0.117	0.020	0.0023
10 to 30	67.9	0.140	0.007	0.0010
30 to 100	82.0	0.106	0.115	0.0123

100 to 300	92.6	0.036	0.212	0.0077
300 to 1000	96.2	0.025	0.441	0.0110
1000 to 3000	98.7	0.010	0.750	0.0072
3000 to 30000	99.7	0.003	1.000	0.0035
Total				**4.6%**

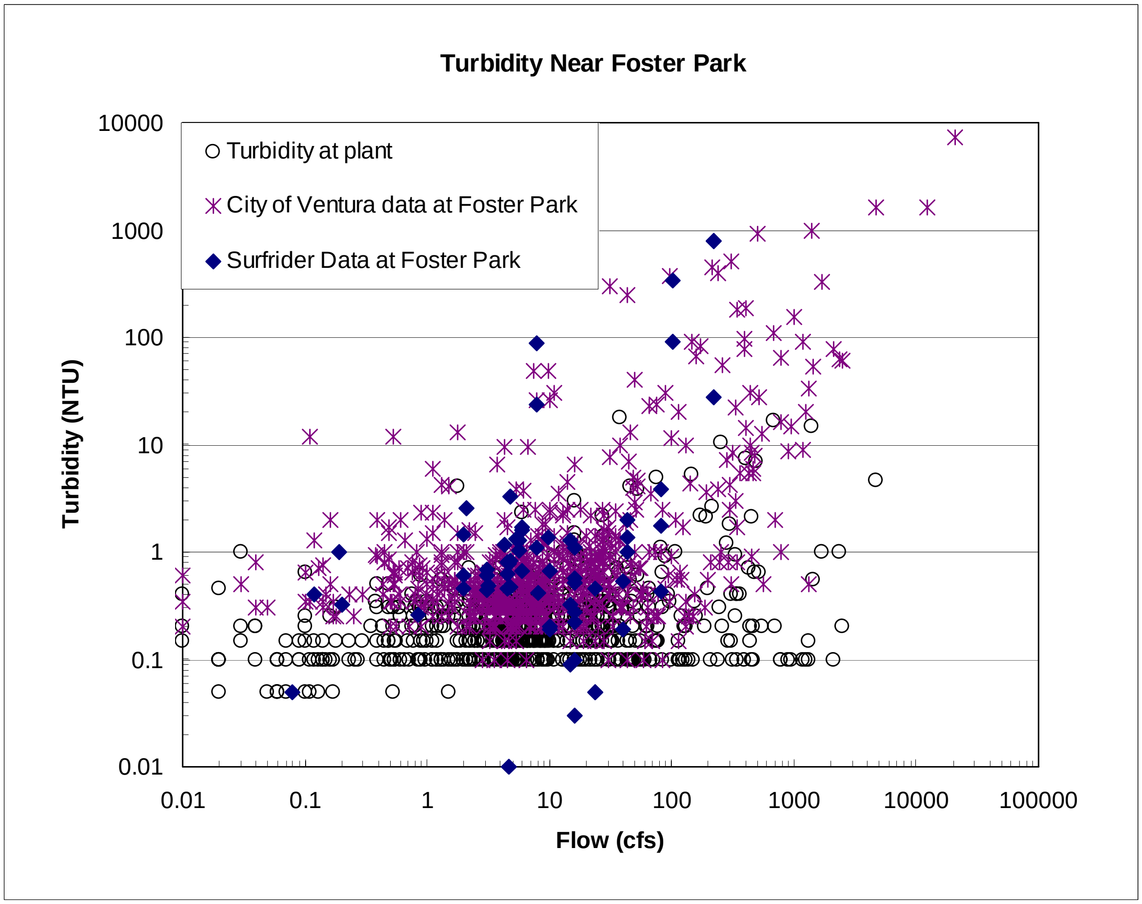

Figure 14. Current Turbidity and Sediment Concentration in the Ventura River at Foster Park.

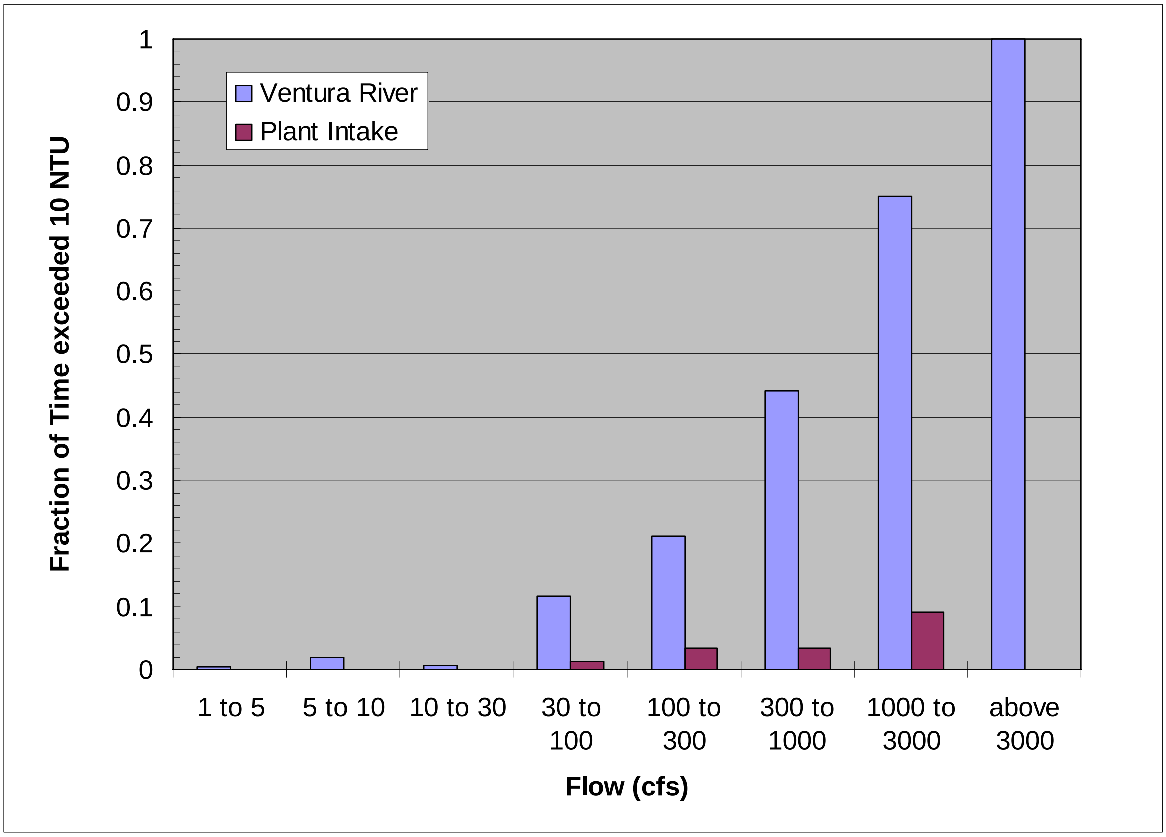

Figure 15. Fraction of Time 10 NTU Criteria is Exceeded in Ventura River and at City of Ventura Water Treatment Plant Intake under Without Project Conditions.

The increase in sediment loads due to the release of sediment would affect the diversion at Foster Park through two possible mechanisms.

1. Increase in Fine Sediment Concentrations – An increase in fine sediment concentration would increase the turbidity and increase the time at which they are unable to divert. The sediment concentration is related to turbidity, but the relation may not be linear. Therefore, doubling the fine sediment concentration may more than double the turbidity.

2. Decrease in Infiltration Rates – If large sediment concentrations exist at low flows (less than 50 cfs), it is possible that as the water is pumped from the subsurface wells, the riverbed may become clogged with sediment. This could only occur until the next high flows mobilize the sediment, but during this period, the yield from the subsurface wells may be reduced. For this to occur, however, the infiltration throughout the entire Ventura River would have to be appreciably reduced. This is not deemed possible, for as soon as this clogging occurs, surface flow would occur that would then erode the fine material from the bed. Therefore, infiltration into the bed would always occur. The aquifer is connected to the River throughout its entire length and there is a groundwater dam just downstream of the diversion that forces water to the surface.

The impact to the diversion for each alternative is discussed below. For each alternative, a range of potential impacts were estimated to capture the uncertainty in the estimates. Only one estimate was calculated for the No Action Alternative as it was assumed that only the differences from the No Action Alternative were critical to this study. The lower bound of the impact is given in Table 10 and an upper bound in given in Table 11.

To calculate the impact, the concentration was assumed to increase over the current condition by some multiple. The multiple of concentration increase was based upon the model results presented in Reclamation (2004). The concentrations simulated before and after flood events were compared against the results for the No Action Alternative for a variety of floods. The simulations were performed for a series of floods occurring back to back, so that the effect of the decrease in concentrations following successive floods could be estimated. Once the multiple of concentration increase was determined, it was assumed that the relative concentration increase would be proportional to the increase in turbidity.

It is estimated that floods with a peak flow of over 3,000 cfs would be sufficient to move significant amounts of sediment from the reservoir. Such floods occur every 2.7 years on average and therefore it would be assumed that floods occur every 3 years for the following tables (Table 10 and Table 11). As previously discussed, a representative diversion rate is 2.5 cfs for the shallow intake and 4.6 cfs for the surface diversion. Therefore, for every day of missed surface diversion, approximately 14 ac-ft of water is not diverted.

<u>Without-Project:</u>

In the future, the fine sediment concentrations would not be significantly different from the present condition. For a period of 15 years, the estimated amount of missed surface diversion would be 3,600 ac-ft.

<u>With-Project</u>:

The delta region consists of approximately 30% silt and clay. This material would be allowed to travel downstream whenever the temporary stabilization structures would be overtopped or removed. It is estimated that there would be four separate removals of revetment. In addition, it is assumed that one flood would pass through the reservoir before any revetment would be removed. The residual sediment that would be left in the constructed channel and in the areas that would be unprotected may increase the turbidity before the first flood.

As a lower bound on the impact, the turbidity levels are assumed approximately twice-current levels until a flood passes through the area after the final removal of revetment. This would mean that approximately 15 years would pass before the turbidity levels decrease to current levels.

As an upper bound on the impact, the turbidity levels are assumed to increase by a factor of 10 for a period of 9 years following dam removal. After year 9, it is assumed that the turbidity levels decrease to approximately four times the current levels until year 15.

After year 15, the turbidity levels decrease to current conditions. The large difference between the lower and upper bounds is justified based upon the uncertainty associated with the bank erosion mechanics in the reservoir area as well as the uncertainty of the hydrology. For example, a large short flood may erode a large portion of the bank but not carry this sediment all the way past Foster Park. Smaller flows may then erode this sediment and prolong the turbidity impact. The increase in sediment concentration would be controlled by the rate at which the revetment would be removed. The upper and lower bounds on the volume of missed surface diversions are 8,820 and 4,950 ac-ft, respectively.

Summary of Foster Park Diversion Impacts

It is estimated that floods with a peak flow of over 3,000 cfs would be sufficient to move significant amounts of sediment from the reservoir. Such floods occur every 2.7 years on average and therefore it would be assumed that floods occur every 3 years for the following tables. As previously discussed, a representative diversion rate is 2.5 cfs for the shallow intake and 4.6 cfs for the surface diversion. Therefore, for every day of missed surface diversion, approximately 14 ac-ft of water would not be diverted.

Table 10. Lower Bound Estimated Annual Surface Water Loss at Foster Park Diversion for a period of 15 years.

	Annual Water Not Diverted due to High Turbidity (ac-ft/yr)					
	Years 1 to 3	**Year 4 to 6**	**Years 7 to 9**	**Years 10 to 12**	**Years 13 to 15**	**TOTAL (ac-ft)**
Without-Project	240	240	240	240	240	3600
With-Project	330	330	330	330	330	4950

Table 11. Upper Bound of Estimated Annual Surface Water Loss at Foster Park Diversion.

	Annual Water Not Diverted due to High Turbidity (ac-ft/yr)					
	Years 1 to 3	**Year 4 to 6**	**Years 7 to 9**	**Years 10 to 12**	**Years 13 to 15**	**TOTAL (ac-ft)**
Without-Project	240	240	240	240	240	3600
With-Project	700	700	700	420	420	8820

8. Rip Rap Design

The Riprap will surround the well and protect it against scour. The top of riprap will be 12 inches below the ground surface. The methods recommended in EM-1110-2-1601 “Hydraulic Design of Flood Control Channels” (USCOE, 1994) were used to design the size of the riprap.

$$D_{30} = S_f C_s C_V C_T d^{-0.25} \left(\frac{V_{ss}}{\sqrt{K_1 g (s-1)}} \right)^{2.5} \qquad \text{Eq 1}$$

where

S_f	= safety factor
	= 1.1
C_s	= stability coefficient for incipient failure
	= 0.3 for angular rock
C_v	= vertical velocity distribution coefficient
	$= 1.283 - 0.2\log_{10}\left(\min\left(26, \max\left(2, R/W\right)\right)\right)$
C_T	= thickness coefficient
	= 0.5 to 1.0, depending upon d_{15}/d_{85} and relative layer thickness
d	= local depth of flow, at same location as V, from HEC-RAS output
s	= specific gravity of the riprap
	= 2.65 (165 lb/ft^3)
V_{ss}	= local side slope corrected velocity
	$= V_{ave}\left[1.74 - 0.52\log_{10}\left(\min\left(26, \max\left(2, R/W\right)\right)\right)\right]$
V_{ave}	= cross section average velocity
K_1	= side slope correction
	= ERF(.4Z$^{1.5}$), where Z = run/rise of side slope
g	= acceleration of gravity
	= 32.2 ft/s^2

For braided streams, EM 1601 suggests that the most severe attack in braided streams may occur when the water surface is at or slightly above the top of the mid channel bars. On the Ventura River, the 10-yr flood is approximately the flood that begins to inundate mid-channel bars. The riprap required under the 100-yr flow was also computed, but was found to be smaller than that required for the 10-yr flood. The radius of curvature for the 100-yr flood is much larger than that under the 10-yr flood and therefore the local side slope velocity, V_{ss}, is smaller for the 100-yr flood than for the 10-yr flood.

The radius of curvature at the 10-yr flood was computed as 1,000 feet. There is considerable uncertainty in computing this value, but below a radius of curvature to width ratio of 2 the riprap sizes do not change. The hydraulic conditions at RM 6.1553 were determined to be the most critical in terms of riprap design. The average channel velocity was 11.2 ft/s and thalweg depth was 9.5 ft (see Table 3). It was assumed that the side slope (Z) was 3, which gives a K_1 factor of 1.0. This gives, for C_T = 1.0, a stable d_{30} of 2.2 ft.

CHANLPRO V2.0 was used to compute the stable ETL gradations using the same input. The output from the program is given in the Appendix C: CHANLPRO V2.0 Output. The minimum stable gradation is summarized in Table 12. Because the layer thickness was larger than the d_{100}, the resulting d_{30} is smaller than 2.2 ft for $Z = 3$.

When the riprap gradation is specified in the design of the protection, the weight of rock should take priority over the size of the rock. However, the specific gravity should be equal to or greater than 2.6 times that of water.

Table 12. The Minimum Stable ETL gradations from CHANLPRO V2.0 for side slope, Z = 3.

Name	12					
Layer Thickness (in)	71					
d_{30} (min)	1.95					
d_{90} (min)	2.82					
	d_{100} (max)	**d_{100} (min)**	**d_{50} (max)**	**d_{50} (min)**	**d_{15} (max)**	**d_{15} (min)**
Weight (lb)	5529	2212	1637	1106	818	346
Diameter (in)	48	35.4	32	28.1	25.4	19

Table 13. The Minimum Stable ETL gradations from CHANLPRO V2.0 for side slope, Z = 2.

Name	13					
Layer Thickness (in)	90					
d_{30} (min)	2.19					
d_{90} (min)	3.17					
	d_{100} (max)	**d_{100} (min)**	**d_{50} (max)**	**d_{50} (min)**	**d_{15} (max)**	**d_{15} (min)**
Weight (lb)	7873	3149	2330	1575	1165	492
Diameter (in)	54	39.8	36	31.6	28.6	21.4

As a check on the recommended gradation, the stable diameter was computed based upon the Shields shear stress criteria:

$$\theta_{cr} = \frac{\tau_b}{(\gamma_s - \gamma) d_{cr}} \qquad \text{Eq 2}$$

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. Assuming a θ_c of 0.02, gives a critical sediment diameter of 30 in, which is similar to the mean d_{50} recommended by CHANLPRO. A non-dimensional critical shields stress of 0.02 was used because it is a typical value used for no motion of sediment. A commonly used value of 0.04 is not for incipient motion, but for some reference transport rate, usually considered the lowest measurable rate.

As another check on the riprap results, the recommended riprap diameters were compared against the existing bed material in the Ventura River. The results from the pebble count at RM 6.0 showed that the 100% of the material was finer than 1.2 feet. Therefore, the river has been unable to move rocks larger than 1.2 feet appreciable distances in this reach.

9. Scour Estimates

The riprap needs to be buried below the elevation of maximum scour. Both the 10-yr scour estimates assuming a 1,000 ft radius of curvature and the 100-yr estimates assuming a large radius of curvature were used to estimate the scour. The 100-yr flood estimates had larger scour estimates.

9.1. Scour Estimation Methods

The scour elevations were estimated using several methods:

9.1.1. Neill

The depth of scour below thalweg elevation, d_s, is predicted by Neill (1973) as reported in Reclamation (1984):

$$d_s = Zd_i\left(\frac{q_f}{q_i}\right)^m$$

where:

m = exponent varying from 0.67 for sand to 0.85 coarse gravel
d_i = bankfull depth
q_i = Bankfull discharge
q_f = design discharge per unit width
Z = 0.6 for moderate bend

9.1.2. Lacey

The scour equation of Lacey (1930) as reported in Reclamation (1984) is:

$$d_s = Z0.47\left(\frac{Q}{f}\right)^{1/3}$$

where:

Q = Flow rate in channel at design discharge (ft^3/s or m^3/s)
f = $1.76\sqrt{d_{50}}$
Z = 0.5 for moderate bend
d_{50} = mean grain size in mm

9.1.3. Blench

The scour equation of Blench (1969) as reported in Reclamation (1984) is:

$$d_s = Z\frac{q_f^{2/3}}{F_{bo}^{1/3}}$$

where:

q_f = design discharge per unit width
F_{bo} = $1.75 d_{50}^{0.25}$
d_{50} = mean grain size in mm
Z = 0.6 for moderate bend

9.1.4. Limiting Velocity

The limiting velocity method as reported in Reclamation (1984) is:

$$d_s = d_m \left(\frac{V_m}{V_c} - 1 \right)$$

where:

d_m = mean depth
V_m = mean channel velocity
V_c = minimum competent velocity

The competent velocity can be estimated using a shear stress based incipient motion criteria:

$$u_\tau = \theta_c \sqrt{g(s-1)D_c}$$

where:

u_τ = friction velocity = $nV_c \sqrt{g} \Big/ \left(C_m R^{\frac{1}{6}} \right)$
V_c = minimum competent average channel velocity
n = Manning's roughness coefficient
g = acceleration of gravity
R = hydraulic radius
C_m = Manning's constant (1.0 for SI, 1.486 for English units)
θ_c = critical non-dimensional shear stress (often between 0.03 to 0.05)
s = specific weight of bed material
D_c = d_{50} of surface bed material

Alternatively, one could use the competent bottom velocity method as recommended in Reclamation (1984) Eq (3). That equation can be rewritten to be dimensionally consistent as:

$$V_c = 0.57\sqrt{g(s-1)D_c}$$

and this equation in used in the analysis in this report.

9.1.5. EM1601

The COE manual EM1601 (COE, 1994) recommends using the following equation:

$$d_s = S_f Z d_m - d_f$$

where:

d_m = average depth in the crossing upstream of the bend.
d_f = depth of thalweg at bend
S_f = Safety Factor = 1.14
Z = factor based upon radius of curvature to width ratio
= $3.37 - 0.66\ln(R/W)$ for sand bed
= $3.37 - 0.7\ln(R/W)$ for gravel bed

The correlation between *Z* and *R*/*W* for gravel bed rivers is very weak based upon Plate B-42 in Appendix B of EM1601. We recommend using the upper value of 2.5 for this design.

9.1.6. Thorne and Abt (1993)

For gravel beds, Thorne and Abt (1993) use the following equation:

$$d_s = S_f Z d_m - d_f$$

where:

d_m = average depth in the crossing upstream of the bend.
d_f = depth of thalweg at bend
S_f = Safety Factor
Z = factor based upon radius of curvature to width ratio
= $2.15 - 0.27\ln(R/W - 2), \quad 2.1 \le R/W < 22$

where the safety factor has been added for design purposes. Thorne suggests that R/W only needs to be greater than 2, but practically R/W should be greater than 2.1. The relationship is only slightly different from the one proposed in EM1601. Because the value of R/W is uncertain in braided rivers, and this relation gives approximately the same values as EM1601, this method is considered identical to EM1601 for this case.

9.1.7. HEC 11

The scour method proposed by HEC-11 (Federal Highway Administration, 1989) is only a function of bed particle size:

$$d_s = \min\left(12, 6.5 d_{50}^{-.11}\right)$$

9.2. Results

The results for each method are given in Table 14. The final design scour elevation at the well sites was computed from the average scour estimates from Neill, Blench, Limiting velocity, EM1601 and HEC11. This gives an average scour estimate of 7.5 feet below thalweg elevation. The sediment simulations predict a reach averaged decrease in thalweg elevations adjacent to the wells. The predicted decrease is between 2 to 3 feet over a period of 50 years. Because the scour estimates are conservative and are the considered to be the maximum scour potentials, it is not recommended that this 2 to 3 feet be added to the local scour values. The sediment supply in the Ventura River is extremely large and deep (meaning over 5 feet) scour holes are not commonly observed in most of the river, therefore the scour estimates in Table 14 are considered to be already conservative.

A profile plot of the scour estimates giving the scour elevation for each well location is in Figure 16. The design scour elevation for the upstream well is 221 and the downstream well design scour is 219, rounding down to the nearest foot. This is currently 16 and 18 feet below current ground elevation for the upstream and downstream well respectively. The large depths are necessary because the river may erode into the bank where the wells are located. The cross sections at the exact well locations are given in Figure 17 and Figure 18, for the downstream and upstream well, respectively.

Table 14. Scour Estimates from Each Method.

	Design Scour Esimates (ft)						
RM	**Neill (1973)**	**Lacey (1930)**	**Blench (1969)**	**Limiting Velocity**	**EM1601**	**HEC11**	**Averaged**
6.3447	8.3	3.8	8.4	6.5	8.5	7.5	7.85
6.25	9.2	3.8	8.2	6.7	6.2	7.5	7.56
6.1553	7.1	3.8	7.5	6.5	8.3	7.5	7.40

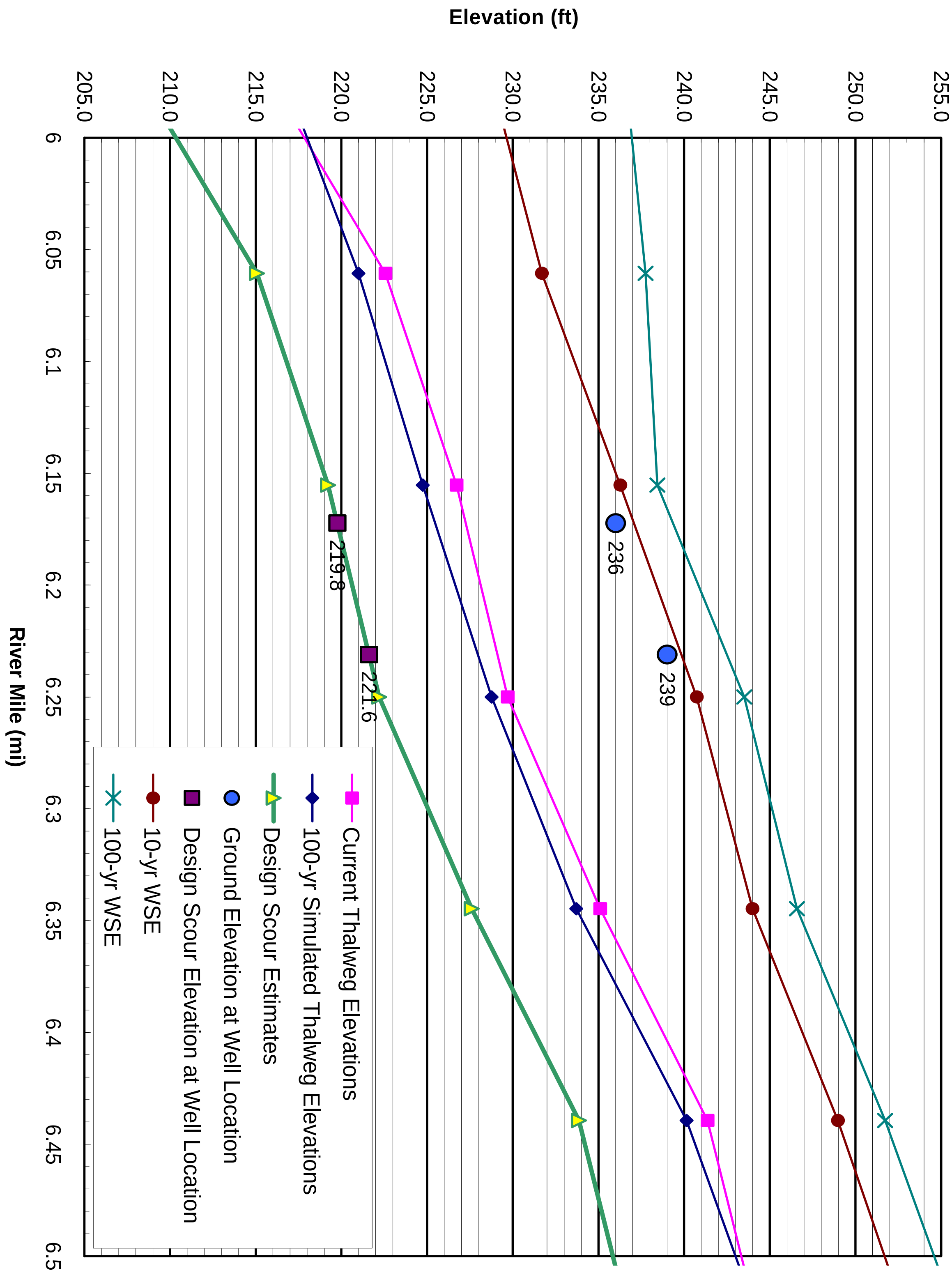

Figure 16. Scour Estimates for Riprap Design.

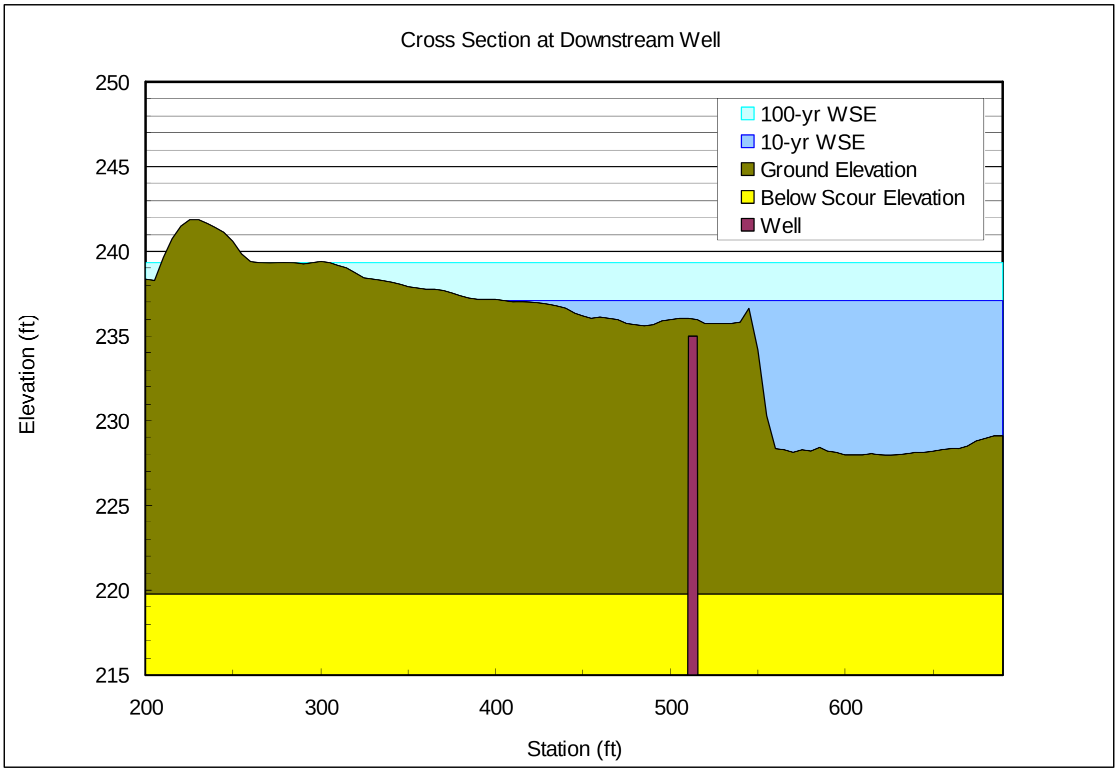

Figure 17. Close up of Cross section at Downstream Well.

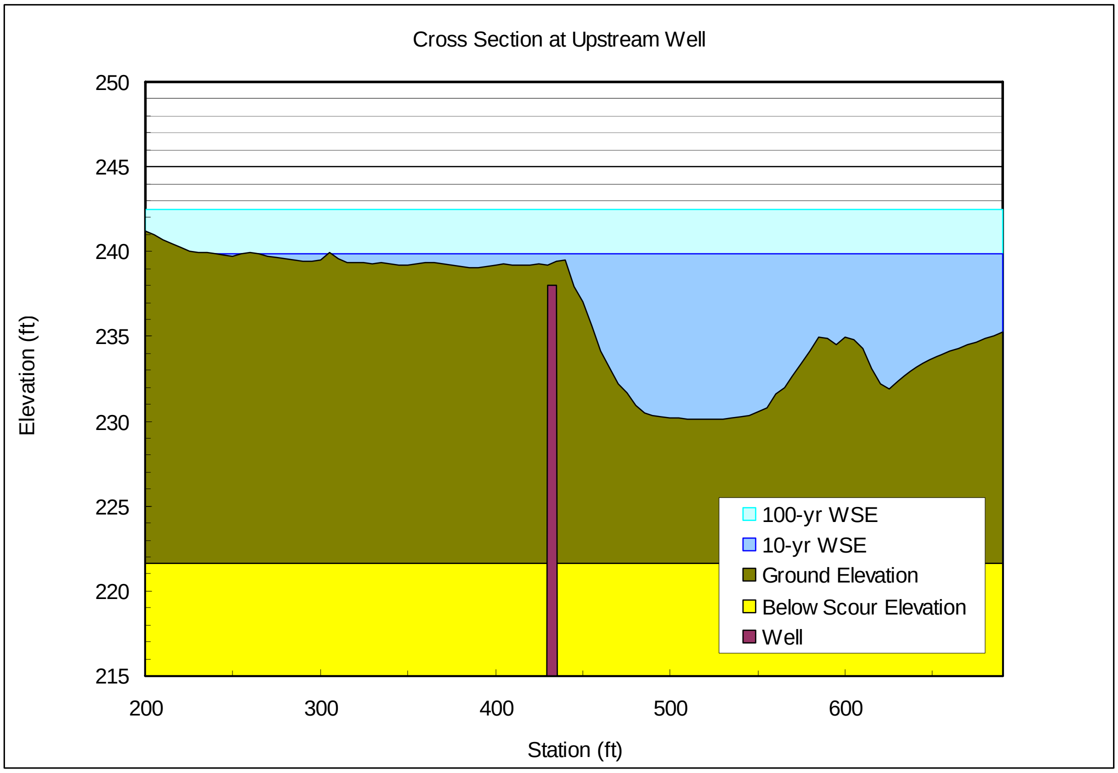

Figure 18. Close up of Cross Section at Upstream Well.

10. References

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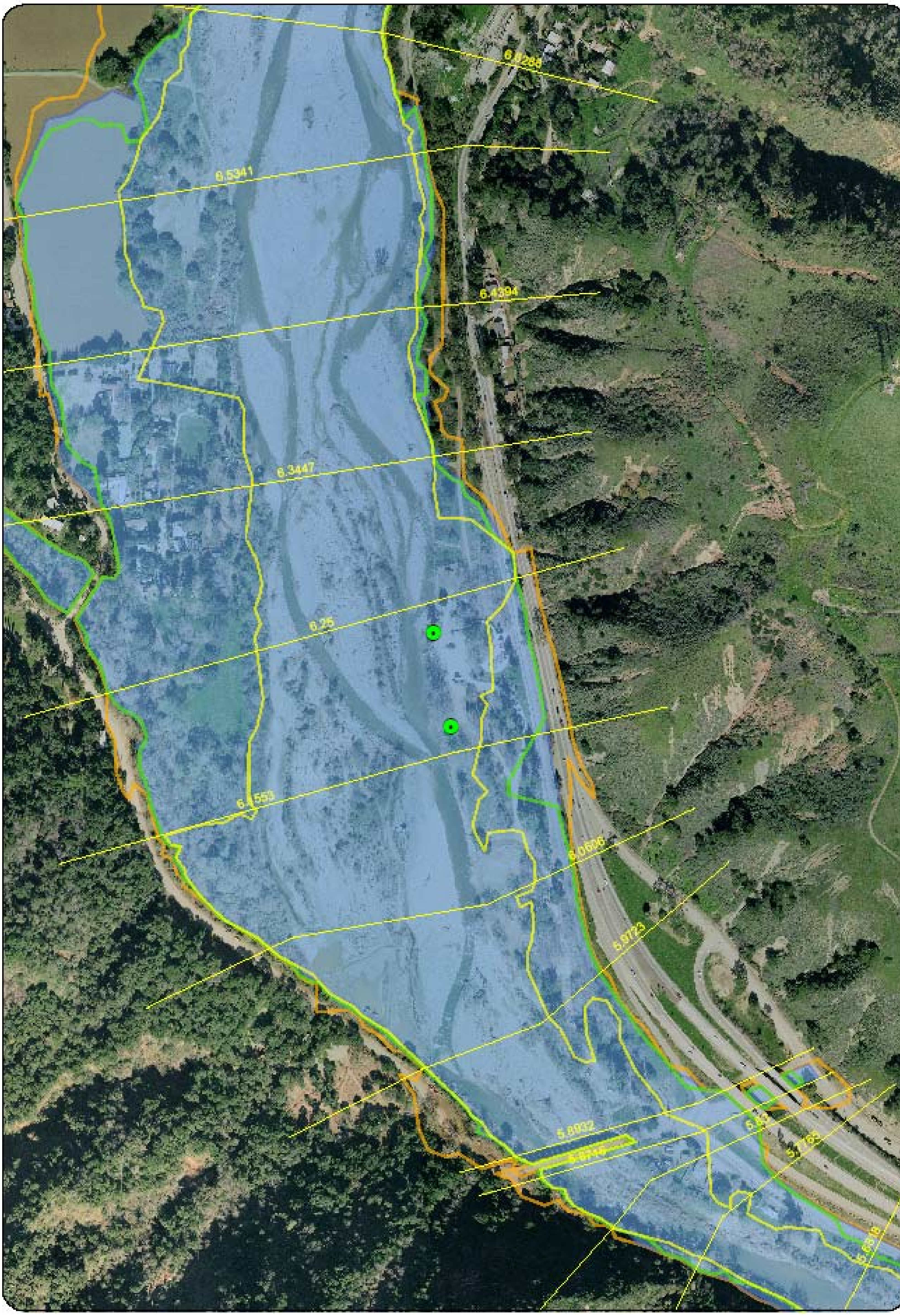
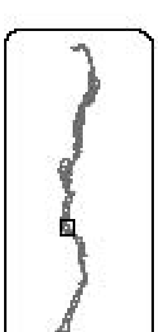
11. Appendix A: Floodmaps





N
S

Foster Park Wells
Current 10-yr
Current 50-yr
Current 100-yr
Current 500-yr

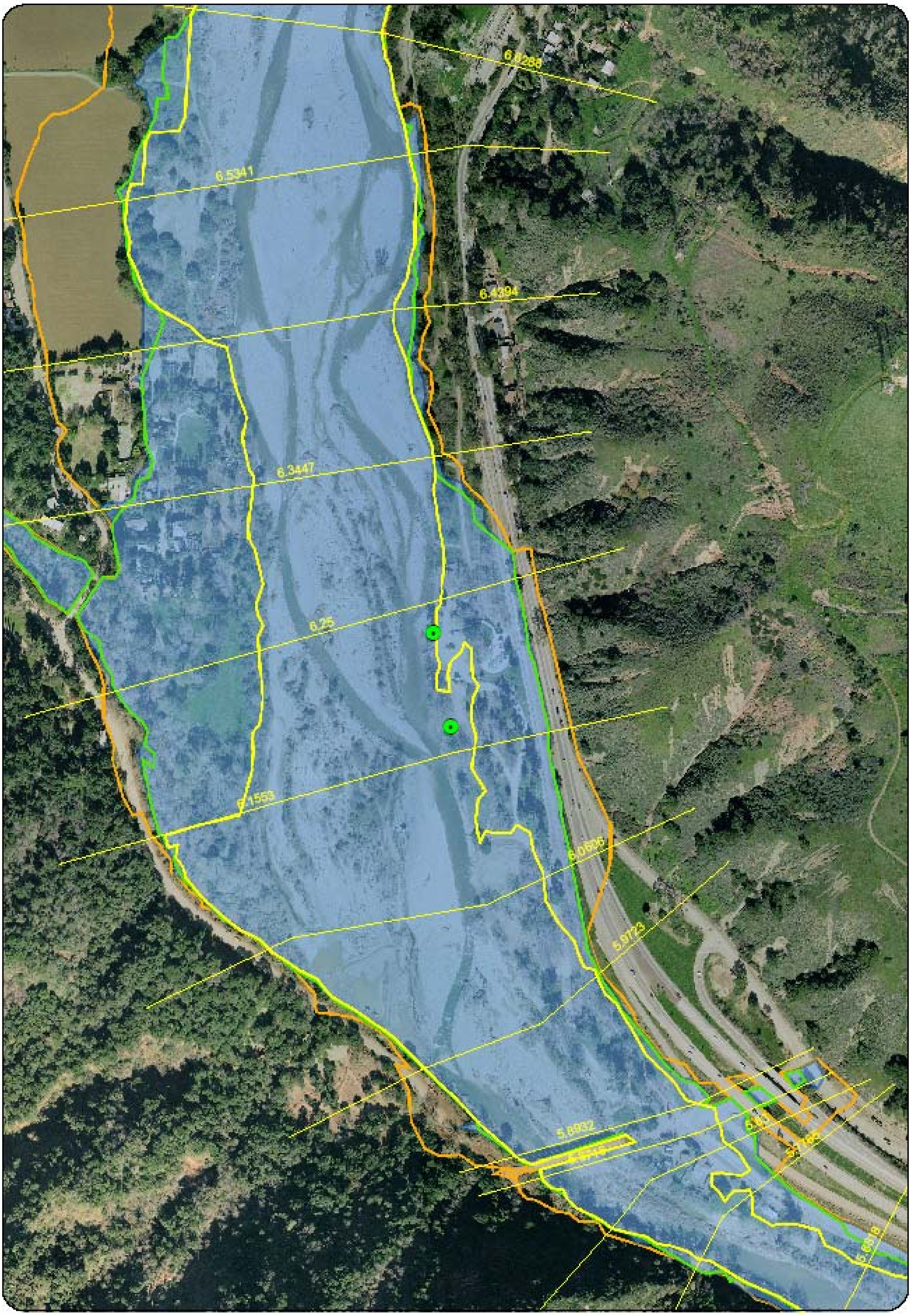
Current Conditions Flood Boundaries

Matilija Dam Ecosystem
Restoration Project
Ventura County, CA

0 100 200 400 Feet

Figure A1

Principal Investigators:
Blair Greimann, David Mooney
US Bureau of Reclamation
Technical Service Center
April 9, 2007



- Foster Park Wells
- Without-Project Future 10yr
- Without-Project Future 50yr
- Without-Project Future 100yr
- Without-Project Future 500yr

N
S

0 100 200 400 Feet

Without-Project Future Flood Boundaries

Matilija Dam Ecosystem Restoration Project
Ventura County, CA

Figure A2

Principal Investigators:
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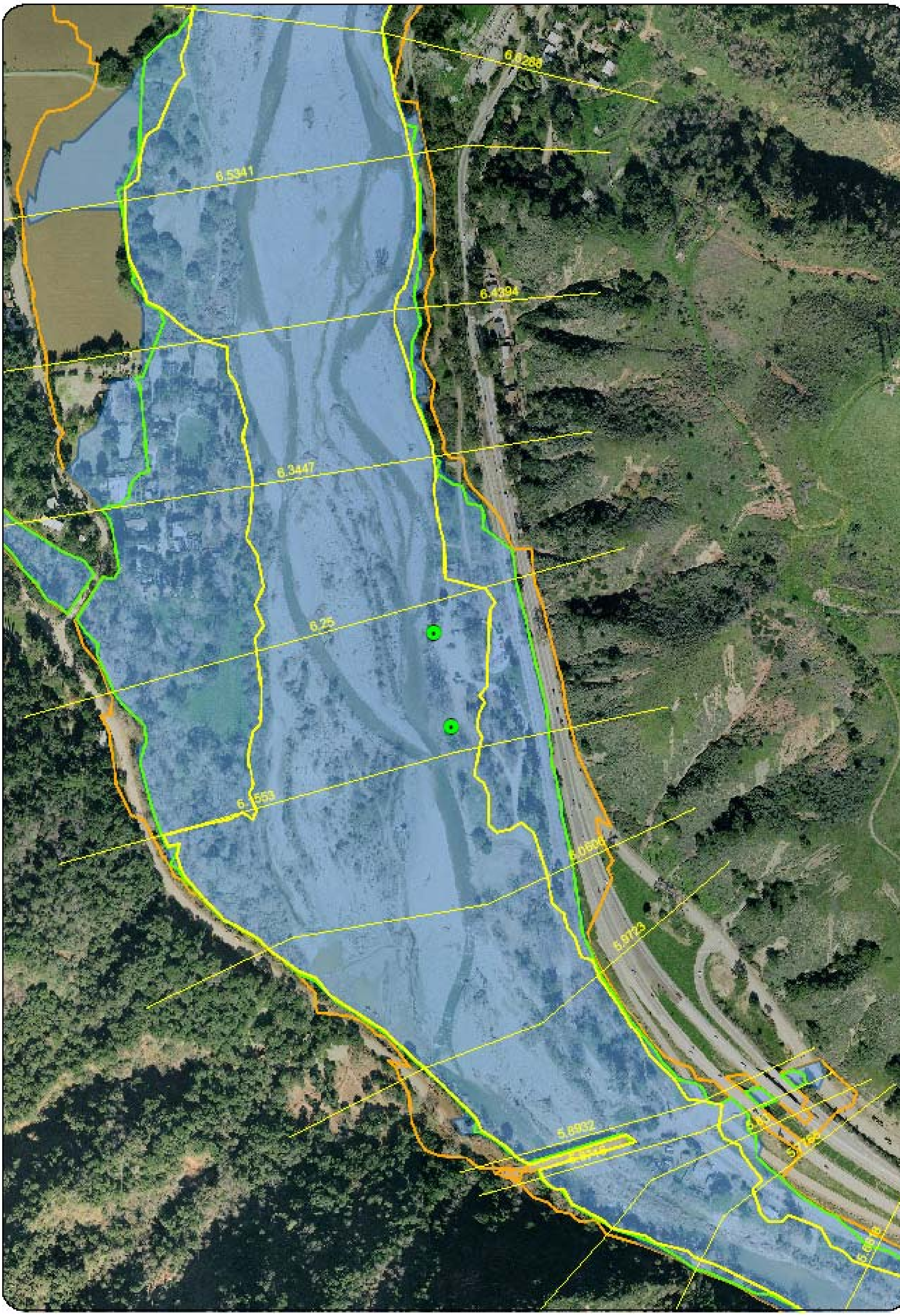
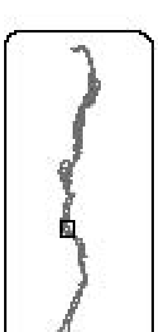
Technical Service Center
April 9, 2007





N
S

- Foster Park Wells
- With-Project Future 10yr
- With-Project Future 50yr
- With-Project Future 100yr
- With-Project Future 500yr

With-Project Future Flood Boundaries

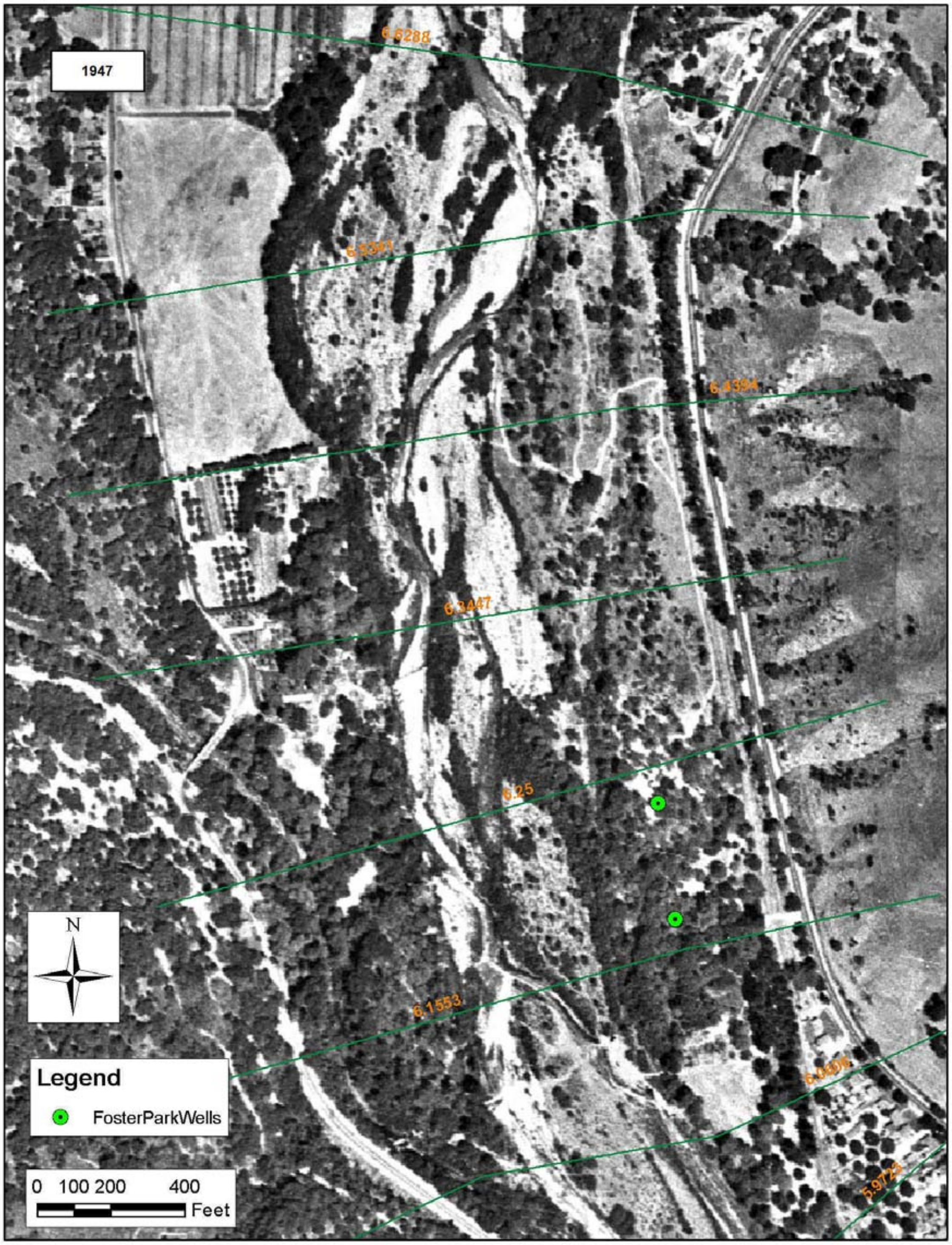
Matilija Dam Ecosystem
Restoration Project
Ventura County, CA

0 100 200 400 Feet

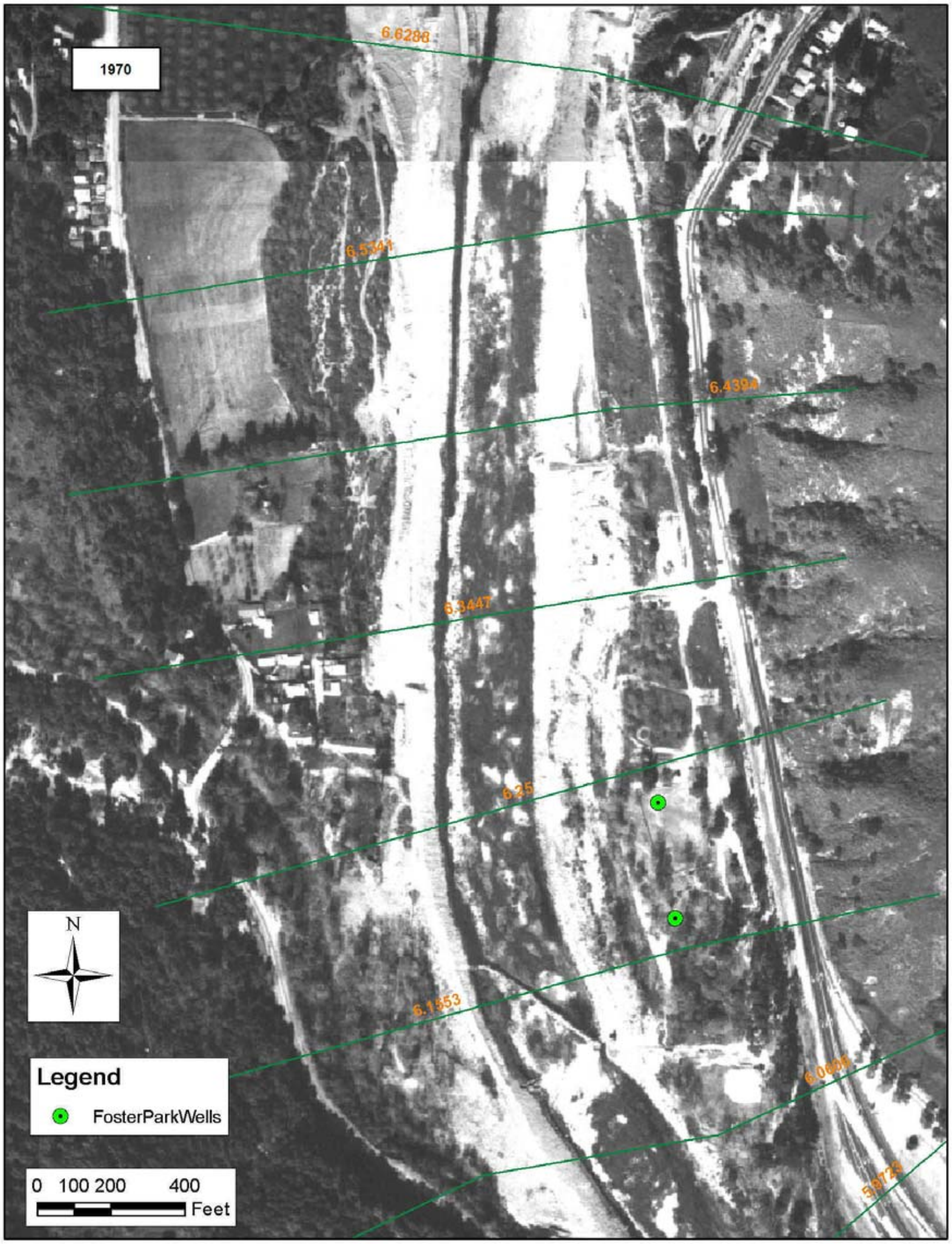
Figure A3

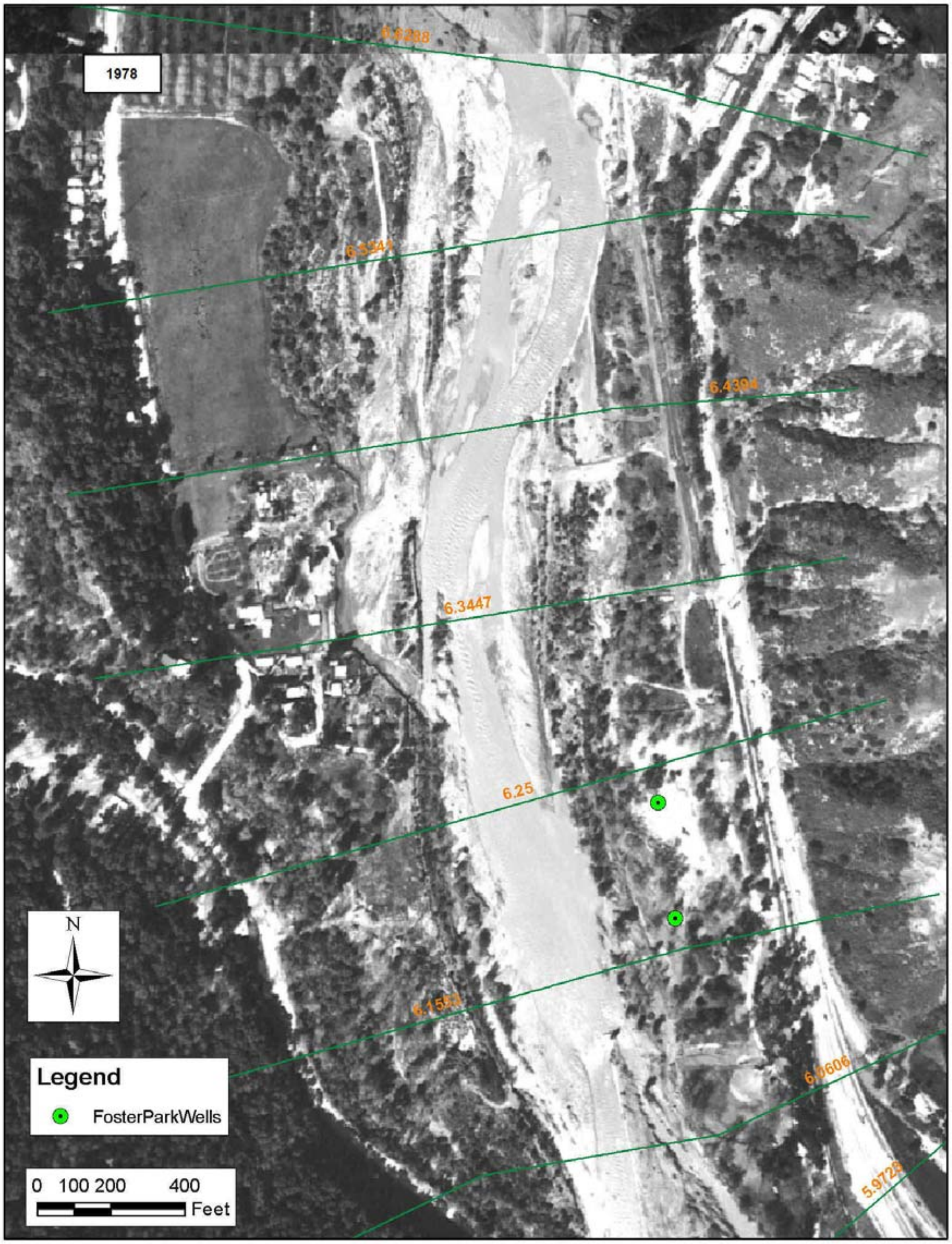
Principal Investigators:
Blair Greimann, David Mooney
US Bureau of Reclamation
Technical Service Center
April 9, 2007

12. Appendix B: Historical Aerial Photographs

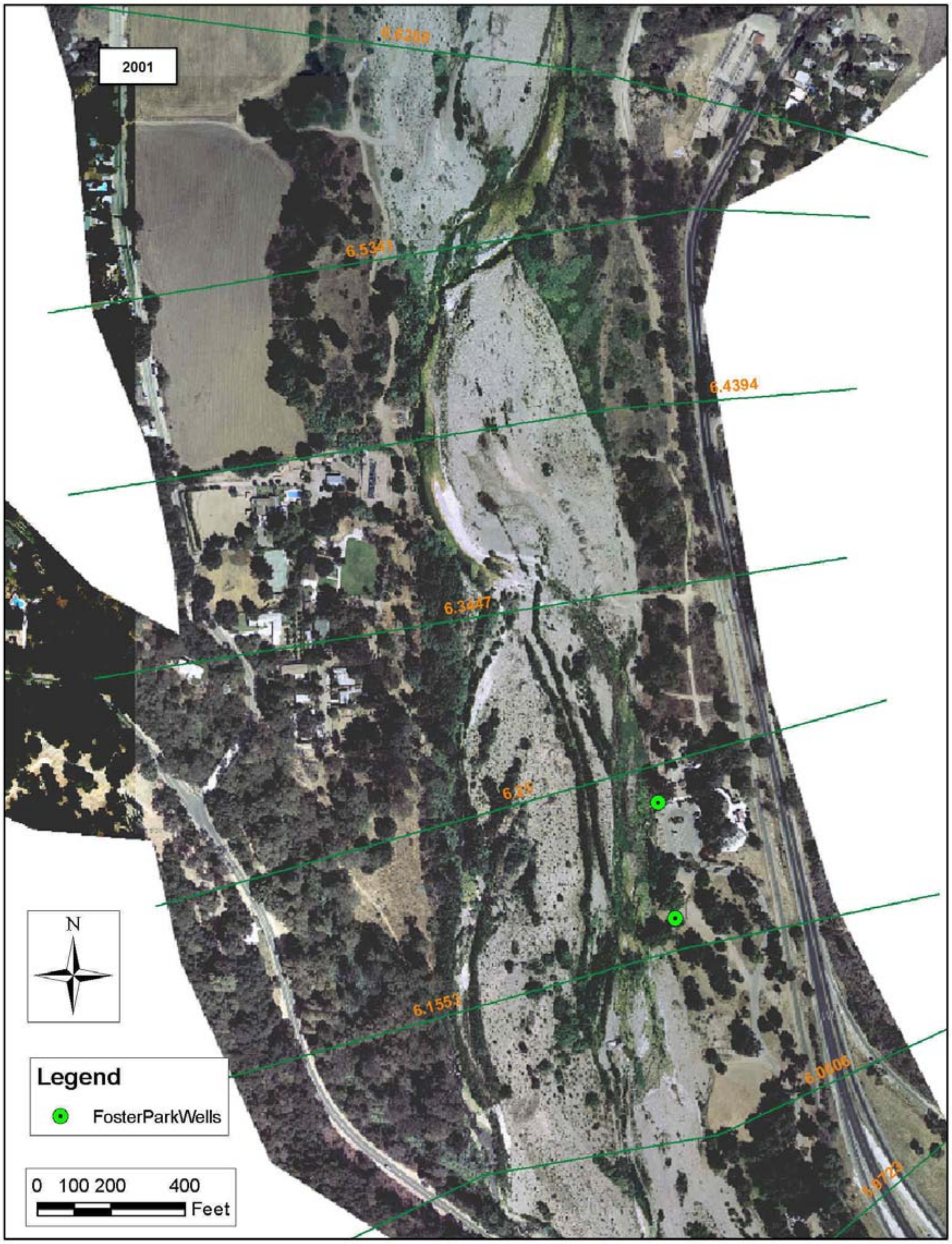


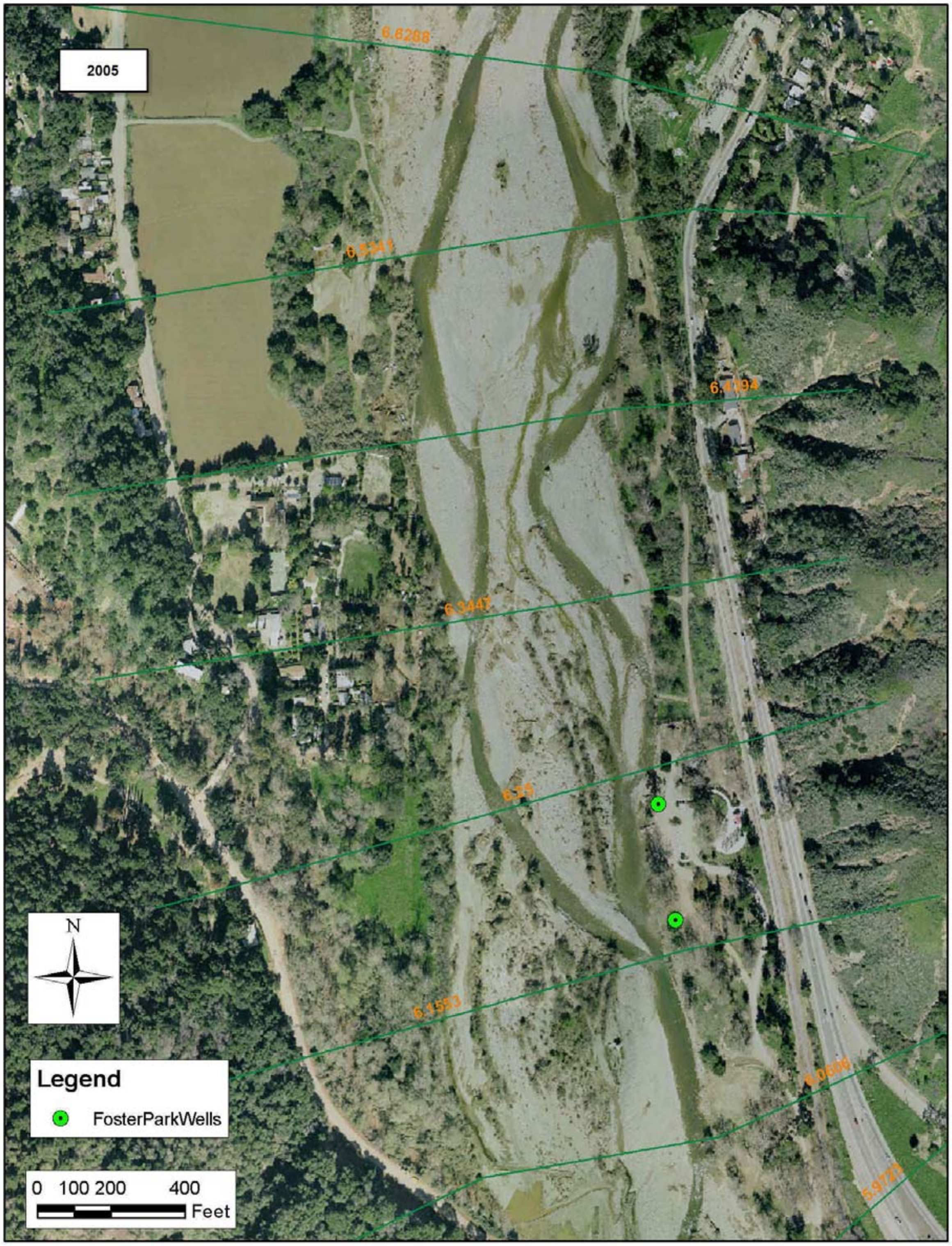
1947
6.6288
6.5341
6.4394
6.3447
6.25
6.1553
5.9723
N
Legend
FosterParkWells
0 100 200 400
Feet


1970
6.6288
6.5341
6.4394
6.3447
6.25
6.1553
6.0606
5.9723
N
Legend
FosterParkWells
0 100 200 400
Feet



1978
6.6288
6.5341
6.4394
6.3447
6.25
6.1553
6.0606
5.9728
N
Legend
FosterParkWells
0 100 200 400
Feet


2001
6.5341
6.4394
6.3447
6.25
6.1553
6.0606
5.9723
N
Legend
FosterParkWells
0 100 200 400
Feet


2005
6.6288
6.5341
6.4294
6.3447
6.25
6.1553
6.0606
5.9723
N
Legend
FosterParkWells
0 100 200 400
Feet

13. Appendix C: CHANLPRO V2.0 Output

Side Slope = 1.0 Vertical: 3.0 Horizontal

```
    PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, BENDWAY
                   INPUT PARAMETERS
    SPECIFIC WEIGHT OF STONE,PCF             165.0
    MINIMUM CENTER LINE BEND RADIUS,FT      1000.0
    WATER SURFACE WIDTH,FT                   600.0
    LOCAL FLOW DEPTH,FT                        9.5
    CHANNEL SIDE SLOPE,1 VER: 3.00 HORZ
    AVERAGE CHANNEL VELOCITY,FPS             11.20
    COMPUTED LOCAL DEPTH AVG VEL,FPS         17.73
    (LOCAL VELOCITY)/(AVG CHANNEL VEL)        1.58
    SIDE SLOPE CORRECTION FACTOR K1            .99
    CORRECTION FOR VELOCITY PROFILE IN BEND   1.22
    RIPRAP DESIGN SAFETY FACTOR               1.10


                           SELECTED STABLE GRADATIONS
                                  ETL GRADATION

    NAME    COMPUTED D30(MIN)  D100(MAX)  D85/D15  N=THICKNESS/   CT  THICKNESS
             D30 FT     FT        IN                  D100(MAX)           IN
     11               1.70      42.00       1.70    NOT STABLE
     12       1.95    1.95      48.00       1.70       1.48      .90    70.8
     13       2.17    2.19      54.00       1.70       1.00     1.00    54.0

  D100(MAX)        LIMITS OF STONE WEIGHT,LB          D30(MIN)  D90(MIN)
     IN          FOR PERCENT LIGHTER BY WEIGHT           FT        FT
                   100           50           15
   48.00     5529   2212   1637   1106    818    346    1.95      2.82
   54.00     7873   3149   2330   1575   1165    492    2.19      3.17

           EQUIVALENT SPHERICAL DIAMETERS IN INCHES
  D100(MAX)  D100(MIN)  D50(MAX)  D50(MIN)  D15(MAX)  D15(MIN)
   48.0       35.4       32.0      28.1      25.4      19.0
   54.0       39.8       36.0      31.6      28.6      21.4
```

Side Slope = 1.0 Vertical: 2.0 Horizontal

```
PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, BENDWAY
               INPUT PARAMETERS
SPECIFIC WEIGHT OF STONE,PCF                  165.0
MINIMUM CENTER LINE BEND RADIUS,FT           1000.0
WATER SURFACE WIDTH,FT                        600.0
LOCAL FLOW DEPTH,FT                             9.5
CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ
AVERAGE CHANNEL VELOCITY,FPS                  11.20
COMPUTED LOCAL DEPTH AVG VEL,FPS              17.73
(LOCAL VELOCITY)/(AVG CHANNEL VEL)             1.58
SIDE SLOPE CORRECTION FACTOR K1                 .88
CORRECTION FOR VELOCITY PROFILE IN BEND        1.22
RIPRAP DESIGN SAFETY FACTOR                    1.10


                         SELECTED STABLE GRADATIONS
                                ETL GRADATION

  NAME    COMPUTED D30(MIN)  D100(MAX)  D85/D15  N=THICKNESS/   CT  THICKNESS
           D30 FT     FT         IN                D100(MAX)           IN
   12                1.95      48.00       1.70    NOT STABLE
   13       2.19     2.19      54.00       1.70       1.66       .87    89.9

D100(MAX)        LIMITS OF STONE WEIGHT,LB          D30(MIN)  D90(MIN)
   IN          FOR PERCENT LIGHTER BY WEIGHT           FT        FT
                100           50            15
 54.00     7873   3149   2330   1575   1165    492    2.19      3.17

         EQUIVALENT SPHERICAL DIAMETERS IN INCHES
D100(MAX)  D100(MIN)  D50(MAX)  D50(MIN)  D15(MAX)  D15(MIN)
 54.0       39.8       36.0      31.6      28.6      21.4
```

14. Appendix D: Plate B-42 from EM1601

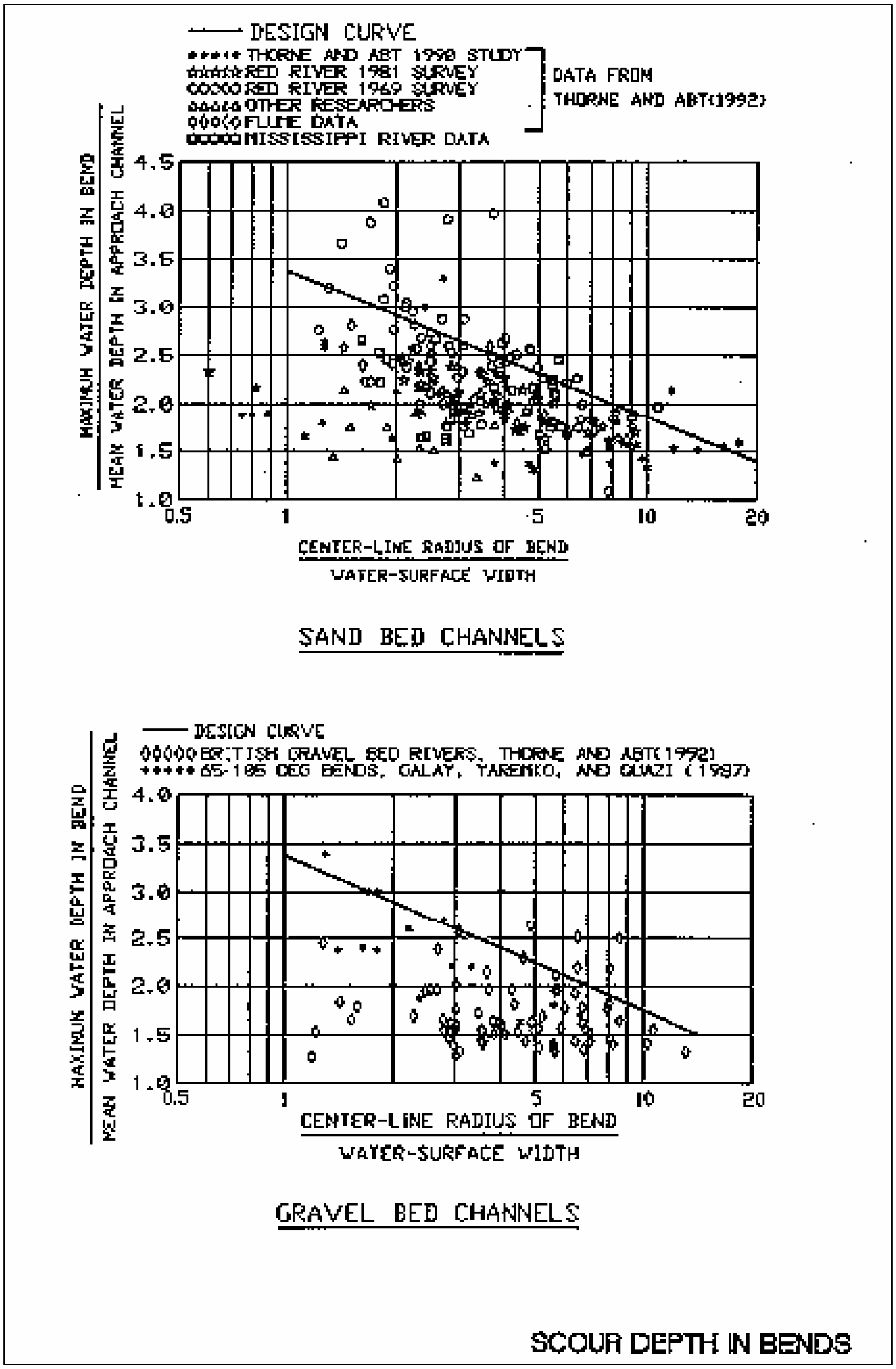


Figure 19. Plate B-42 from EM1601.