

# RECLAMATION

*Managing Water in the West*

## Hydraulic and Sediment Considerations for Proposed Modifications to O'Neill Diversion Weir on Santa Margarita River, California



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

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# **Hydraulic and Sediment Considerations for Proposed Modifications to O'Neill Diversion Weir on Santa Margarita River, California**

Report Prepared by:

Jennifer A. Bountry, P.E., Hydraulic Engineer  
Sedimentation & River Hydraulics Group, Technical Service Center

Report Peer Reviewed by:

Tom Gill, Hydraulic Engineer  
Water Resources Research Laboratory, Technical Service Center

Hydraulic Model Results Peer Reviewed by:

Kent Collins, P.E., Hydraulic Engineer  
Sedimentation & River Hydraulics Group, Technical Service Center

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

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## Background

The O'Neill Diversion Weir is located on the Santa Margarita River in southern California; about 10.5 miles upstream of the mouth which enters into the Pacific Ocean (Figure 1). The diversion is currently operated by United States Marines Corps Base Camp Pendleton (Camp Pendleton). This diversion facility is proposed to be modified to reduce sedimentation issues that currently occur in the diversion canal and in the river channel. The Bureau of Reclamation (Reclamation) has been requested to assist Camp Pendleton in developing possible design criteria for the new diversion facility.

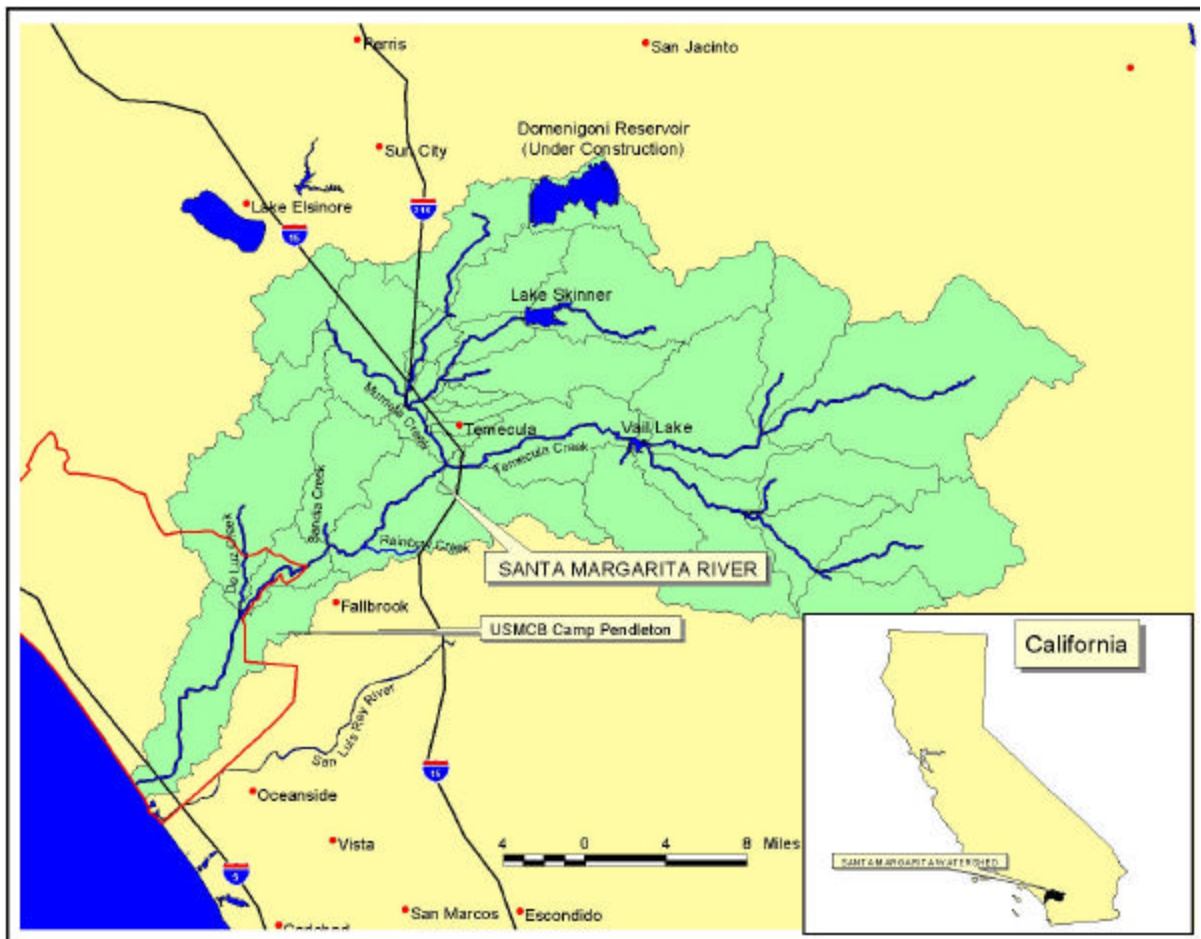


Figure 1. Overview of Santa Margarita watershed (reprinted from Figure 2-1 of WEST (2000) report).

## Study Objective

The objective of this report is to provide a cursory level assessment of hydraulic and sedimentation considerations for proposed modifications to assist feasibility level design efforts. This report uses readily available data and information provided in previous studies and did not include any new data collection. More detailed hydraulic and sedimentation data and analysis may be required for later stages of design and development of new operation strategies.

## Previous Studies Utilized

WEST Consultants (2000) recently developed a set of working hydrologic, hydraulic and sediment transport analytical tools to address water resource and sedimentation problems/issues on the Santa Margarita River watershed for the Army Corps of Engineers. The West (2000) study also provided a review of prior hydrologic, hydraulic and sediment studies for the Santa Margarita River watershed. Hydrologic peak flood data, a hydraulic model, and results from a sediment transport model developed from the WEST (2000) study were used in Reclamation's hydraulic and sedimentation assessment of modifications to the O'Neill Diversion Weir. Bed-material size gradation analysis from Appendix D of a 1995 study by Simons, Li & Associates was provided by Camp Pendleton staff and was utilized in the West (2000) study to characterize the bed-material.

A recent Stetson Engineers (2002) report provides an analysis and results of a feasibility level study that investigated the implementation of a conjunctive use project between the Fallbrook Public Utility District (Fallbrook PUD) and the Camp Pendleton. This study investigated two sources of supply to be applied for beneficial use: naturally occurring streamflow and tertiary treated wastewater. As part of this effort, initial designs were developed to modify the existing O'Neill Diversion Weir.

## Methodology

Information from previous studies and readily available aerial photography were utilized to assess the general characteristics of the watershed and Santa Margarita River. Discussions with Camp Pendleton water resources staff and observations made during a June 2004 site visit were used to assess the current sedimentation issues associated with the existing diversion structure, and the desired goals of implementing a new structure. During this meeting, Camp Pendleton staff stated that the goals of the proposed diversion and canal modification project are:

1. Increase diversion capability to 200 ft<sup>3</sup>/s (currently about 60 ft<sup>3</sup>/s).
2. Design system to allow flushing of sediment during floods to retain natural stream gradient.

3. Design system to minimize deposition in front of diversion gate or entering diversion canal to keep canal relatively free of sediment and debris
4. Design system to be able to maximize the amount of water that can be diverted within water right and have minimal maintenance and sedimentation issues.

The Stetson (2002) report evaluated several alternatives for modification to the existing diversion facilities to meet these goals. This Reclamation report focuses on main channel river processes related to the preferred alternative to replace the existing sheet pile diversion dam with an Obermeyer spillway gate system. This alternative would also increase the existing instantaneous capacity of the head gate and ditch facilities from approximately 60 ft<sup>3</sup>/s to 200 ft<sup>3</sup>/s.

Because flushing of sediment is one of the highest priorities, an assessment of existing sediment characteristics and transport potential was accomplished by using some indicative tools and results from the West (2000) sediment transport model. These computations along with past experience from similar sedimentation studies were used to evaluate the feasibility of the proposed structure to meet the goals of Camp Pendleton related to sedimentation issues.

The WEST (2000) hydraulic model was modified for this study to include the proposed replacement structure, as supplied by the Reclamation designers. The model was then used to develop feasibility level design estimates of 100-year flood elevations and hydraulic properties in the vicinity of the structure. This model is not currently calibrated, and, therefore, the accuracy of flood stage predictions is not known. Additional data would need to be collected during future floods to calibrate the model. The existing topography available does not include detailed channel geometry. Additionally, once the new structure is installed and operating, the channel topography would be different than the existing conditions due to the new sluicing capabilities. Therefore, channel geometry for modeling the 100-year flood during proposed conditions was estimated based on predicted natural channel gradients and channel widths.

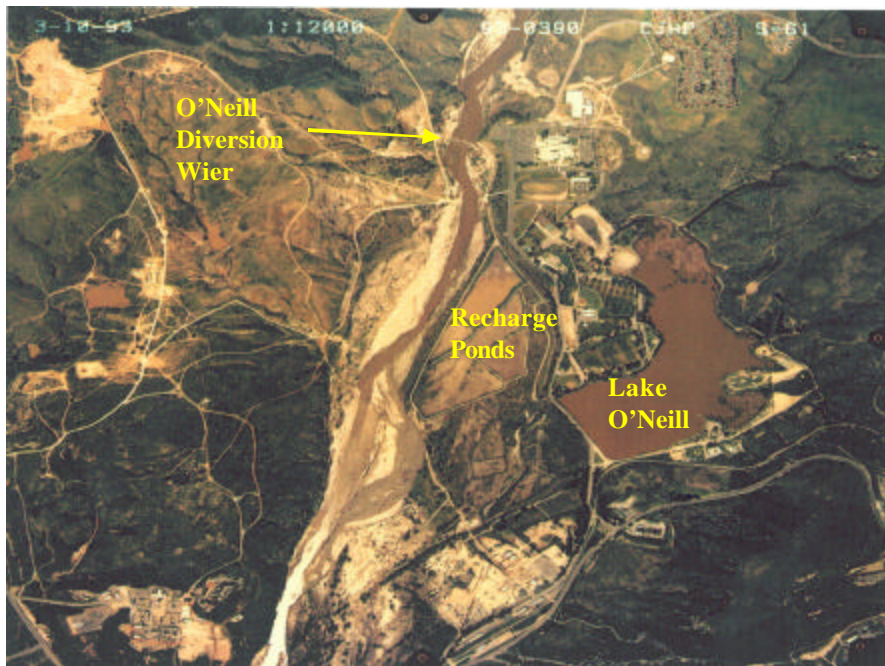
No new data collection was done for this initial level of study. Recommendations for additional data collection and analysis are suggested at the end of the report that may be useful for final design stage.

## Site Description

The Santa Margarita River has a drainage basin of 744 square miles (Stetson, 2002) that is dominated by rainfall generated storms, although some snowfall occurs in the upper portion of the watershed. Over 60 square miles of the basin are located in the southern portion of Camp Pendleton (Stetson, 2002). The river is approximately 29 miles long from the confluence of the Murrieta and Temecula Creeks to the mouth at the Pacific Ocean.

### O'Neill Diversion Structure

The existing O'Neill Diversion Weir diverts surface flow of the Santa Margarita River on the east side into the O'Neill Ditch, which carries the water to five ground-water recharge ponds (capacity of 260 AF) and the man-made Lake O'Neill (capacity of 1,200 AF) (Figure 2). The amount of water diverted in past years has been dependent on available supply and required demand. During the diversion season, a series of control structures and measuring devices allows Camp Pendleton staff to manage, control and measure the diversion to each of the different facilities (Stetson, 2002).



**Figure 2. 1993 Aerial photograph of O'Neill Diversion Weir, recharge ponds, and Lake O'Neill. River flow in photograph is moving from top of photo to bottom. Flood of record near 100-year flood level occurred on January 16, 1993 at 44,000 ft<sup>3</sup>/s, approximately two months prior to this photograph taken at a flow of 211 ft<sup>3</sup>/s. Photo provided by Camp Pendleton.**

The original diversion dam used to divert water to the recharge ponds was located slightly downstream where a road crossing now exists. It is estimated that the original dam was constructed in the late 1950s based on drawings and local knowledge on the Base that the recharge ponds were first filled during this same time period (personnel communication with Mike Malloy at Base). According to the Stetson report (2002), the ground-water recharge pond system was constructed between 1955 and 1962 and Santa Margarita River diversions to the recharge ponds were first recorded in October 1960 (Figure 2). It is not known exactly when the dam was moved upstream to its current location, but it is known it was originally composed of large rock. The rock weir repeatedly got washed out during floods and had to be reconstructed. It was known to be last rebuilt with rock in December 1980, and washed out a short time afterwards (personnel communication with Mike Malloy at Base). In 1982, the dam was replaced with steel sheet-pile.

The existing sheet-pile dam stretches across a 280-foot section of active river channel, and continues an unknown distance up the armored west (right) bank and into the floodplain (Figure 3). According to the 1982 construction drawings, the sheet piles in the river are 30 feet in height (length) and were driven to a depth that fixed the weir crest elevation at about 116.6 feet. According to new June 2004 survey data from which new topography will be developed, the weir crest measured at 117.0 feet in one location and 117.5 in another. More survey work needs to be done to verify this information. Water impounded behind the sheet pile weir may be diverted through a 60-inch by 48-inch (span by rise) slide gate mounted on a concrete headwall on the eastern bank of the river (Stetson, 2002). The slide gate is manually operated to pass river diversions through a 45-foot long section of arch corrugated metal pipe (CMP) having dimensions of 65-inches by 40-inches. The invert elevation of the arch CMP at the entrance of the diversion is 112.1 feet according to the 1982 construction drawings. The capacity of the arch CMP diversion pipe is estimated to be 75 cubic feet per second ( $\text{ft}^3/\text{s}$ ) with a water surface elevation at least 3.4 feet (115.5 feet - 112.1 feet) above the pipe inlet.

Historic diversions have generally captured the majority of low and normal flows up to the capacity of the diversion structure. With current operation of diverting during low flow, water is allowed to pass downstream of the weir through small holes in the sheet pile. During a flood, the operator would watch the river to determine when the majority of debris (human caused and natural) had passed and the turbidity in the river settled down. When the water appeared clear, personnel would start diverting to fill the recharge ponds. During floods, small stop logs can be used to sluice sediments at two locations along the left side of the dam. The stop logs are small in length relative to the total sheet-pile structure, and, therefore, can only sluice a small portion of sediment in the upstream channel during floods.



**Figure 3. Looking from east bank across diversion weir after dredging of upstream reservoir was accomplished to reduce sedimentation. Dredge piles were placed on both sides of the river. Several rocks were estimated to have been placed at the toe of the dam after the 1993 flood. These rocks have since been washed a small distance downstream during subsequent floods as observed in this photo (2-8-99). Photo provided by West Consultants.**

According to Camp Pendleton staff, sediment routinely deposits in the diversion canal and recharge ponds. During low and normal flows, the main path of the river flows directly into the canal with very minimal water released in the downstream river channel (Figures 4 and 5). Additionally, a large volume of sand-sized sediment is trapped upstream of the sheet-pile dam that provides a continual supply. Because there is a large supply of sediment and the majority of flow is diverted, sediment is transported directly into the canal even during low flows which normally would not mobilize sediment in the main channel area. The canal is dredged approximately every 2 years and quickly refills with sediment following dredging operations. Dredging of the main river would occur in the vicinity of the structure in the 1980's and 1990's, but hasn't occurred in recent years. During dredging, sediment would be pushed to the right and left side of the channel into large piles, which is still present today in part (see Figure 3). During subsequent floods, the sediment quickly refills the river channel upstream of the dam. Several million dollars are also spent approximately every 10 years to dredge sediments that have deposited in Lake O'Neill from the diversion operations.





**Figure 4. Looking upstream at active channel just upstream of O'Neill Diversion Weir (June 2004).**



**Figure 5. Looking across from left bank at O'Neill Diversion Weir and majority of river flow shown in Figure 4 being diverted into canal.**

During the June 2004 field visit, the sheet-pile in the west floodplain was mostly buried with fine sediment, but could be observed at least 100 feet to the west of the main river channel. The depth of the sheet-pile in the floodplain is not known. Only about three to four feet of sheet pile was exposed above the downstream river bed (distance between crest and toe of structure) and sediment was generally piled up to near the elevation of the sheet-pile crest on the upstream side (Figure 6). Following the 1993 flood, about 8 feet of sheet-pile was exposed above the downstream river bed according to Camp Pendleton staff (downstream toe scoured out). The 1993 flood also washed out the entire embankment around the intake, and generally washed out most of the vegetation that had built up in the channel. After this flood, large rock riprap was placed at the toe of dam. In 1995 and 1998, two additional floods occurred that were near the 10-year flood level. The rock was partially eroded during these floods, and more rock was subsequently placed at the toe according to Camp Pendleton staff (see Figure 3). Much of this placed rock that has been washed out could still be observed during the June 2004 site visit just downstream of the dam below the vegetation.

Future diversion operations are not established at this time, but it is proposed to increase the diversion capacity to 200 ft<sup>3</sup>/s and diversion may need to occur during all flows to meet higher water supply demands. With the proposed modifications listed previously in the methodology section of this report, Camp Pendleton hopes to reduce the current sedimentation issues and reduce the need for future dredging operations in the river, diversion canal, and recharge ponds.



**Figure 6. Looking downstream along middle of channel at sediment piled up against existing sheet-pile diversion weir (6-8-2004).**

## Hydrology

The hydrology is highly variable with typical low flows occurring during the summer months and extreme peak events occurring during winter rainfall events. Annual precipitation amounts measured at the Lake O'Neill since 1882 average around 13.9 inches, but have a wide range from a minimum of 4.2 inches in 1961 to as much as 40 inches in 1993 (Stetson, 2002).

Three reservoirs are located within the watershed (WEST, 2000):

- 1) Vail Dam, built in 1948, controls 320 square miles or 43% of the total watershed area
- 2) Skinner Lake, finished in 1974, controls 51 square miles or 7% of the total watershed
- 3) Domenigoni Reservoir, currently under construction, will control approximately 17 square miles or 2% of the watershed

The Ysidora gaging station operated by the U.S. Geological Survey (USGS) provides the best available data for evaluating river flows in the vicinity of the O'Neill Diversion Weir (Bullard, 2004). The WEST (2000) study developed peak flood frequency estimates based on the Ysidora gage for river miles 12 to the mouth of the Santa Margarita River, taking into account the regulation by Vail Dam, and for upstream river reaches based on additional gaging data (Figure 7 and Table 1). The Ysidora gage has been operated since 1923, but has had several locations and the peak flood records are only considered "fair", and mean daily discharges "poor". WEST (2000) noted that the changes in gage location are not expected to impact the frequency analysis because the change in contributing area for the different locations is small. The flood of record was on

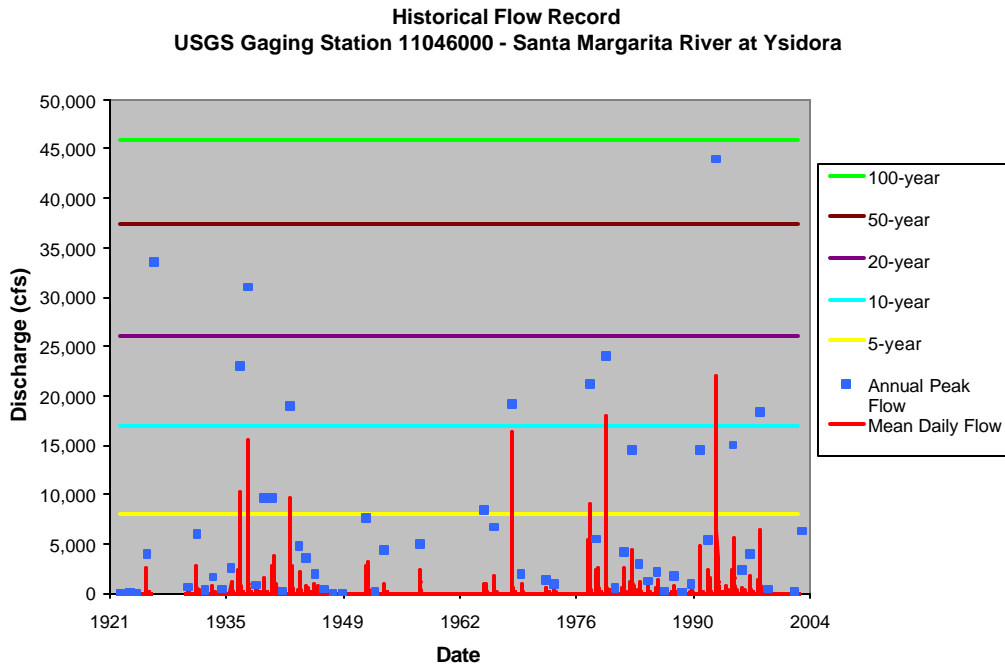


January 16, 1993 at 44,000 ft<sup>3</sup>/s, slightly less than the 100-year flood prediction. The most recent flood occurred in 1998 at 18,400 ft<sup>3</sup>/s, slightly greater than the 10-year flood.

**Table 1. Return period flows from West (2000) report.**

Return Period (Years)	Flow (ft <sup>3</sup> /s) for Specified River Miles (RM)		
	RM 29 to 22	RM 22 to 12	RM 12 to 0
5	7,200	8,000	8,000
10	14,000	17,000	17,000
20	21,400	26,000	26,000
50	29,000	36,000	37,500
100	35,000	44,000	46,000

Data compiled from USGS mean daily flow records using software developed by Hydrosphere Data Products, Inc. (2001) was analyzed to determine typical flow ranges on a monthly basis (Table 2). Between January to March, average daily flows range between 100 to 166 ft<sup>3</sup>/s. During April and May, average flows drop off to below 50 ft<sup>3</sup>/s, and on average remain below 20 ft<sup>3</sup>/s for the remainder of the year.



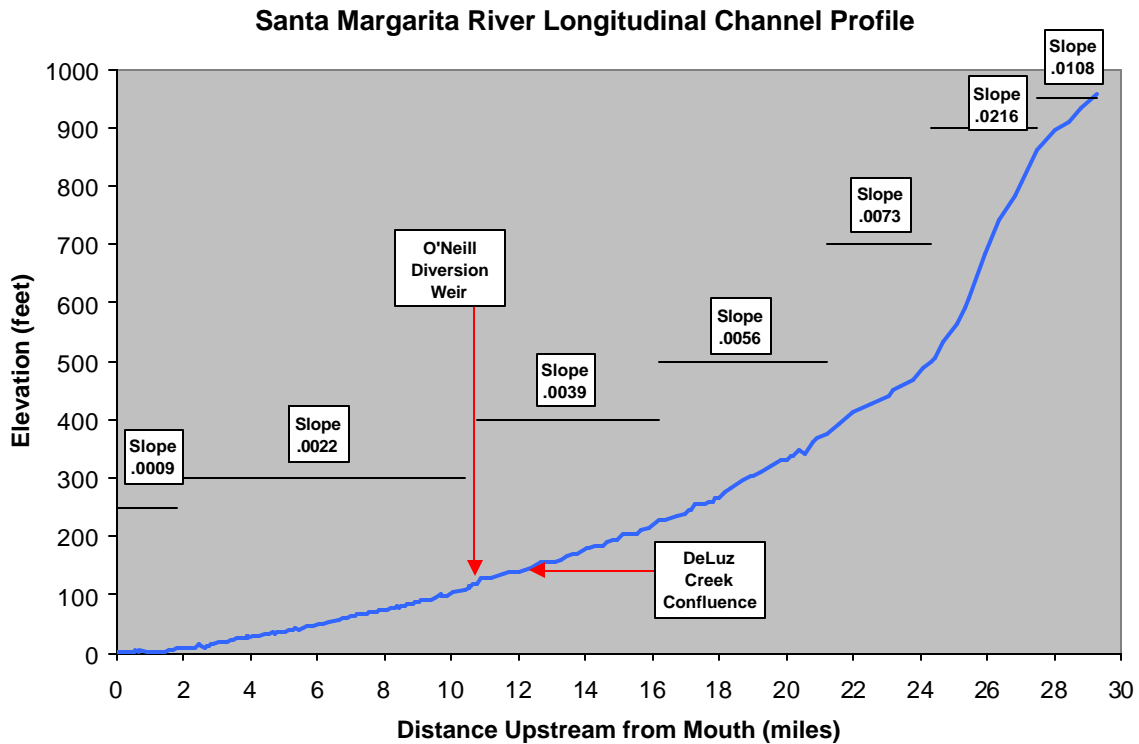
**Figure 7. Historical gage data and flood frequency estimates from West (2000) report for Ysidora gaging station (11046000). Flood frequency values represent river mile 12 to the mouth (see Table 1).**

**Table 2. Monthly statistics of mean daily flow from Ysidora USGS gaging station (11046000).**

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
# Days	2,232	2,031	2,232	2,160	2,232	2,160	2,232	2,232	2,160	2,232	2,160	2,232	26,295
Avg Day	100	166	145	43	12	4	1	1	1	2	7	19	41
Max Day	22,000	18,000	15,500	2,650	477	75	19	97	165	276	1,080	2,770	22,000
Min Day	0	0	0	0	0	0	0	0	0	0	0	0	0
# Months	72	71	72	72	72	72	72	72	72	72	72	72	71
Sdev Month	299	391	301	83	24	8	2	4	2	6	15	30	70
Skew Month	6	4	3	3	3	3	3	7	4	5	3	2	2
Min Month	0	0	0	0	0	0	0	0	0	0	0	0	0
Max Month	2,261	2,249	1,730	465	121	37	10	32	14	39	66	141	337
<b>Exceedance</b>	<b>Jan</b>	<b>Feb</b>	<b>Mar</b>	<b>Apr</b>	<b>May</b>	<b>Jun</b>	<b>Jul</b>	<b>Aug</b>	<b>Sep</b>	<b>Oct</b>	<b>Nov</b>	<b>Dec</b>	<b>Year</b>
1%	2,214	3,423	2,600	520	117	47	15	15	9	27	86	244	665
5%	235	538	548	172	67	22	7	4	3	11	22	56	106
10%	92	247	280	106	32	12	4	2	1	4	12	34	42
20%	36	79	110	50	15	6	1	0	0	1	3	15	14
50%	9	13	12	7	2	0	0	0	0	0	0	1	0
80%	0	0	0	0	0	0	0	0	0	0	0	0	0
90%	0	0	0	0	0	0	0	0	0	0	0	0	0
95%	0	0	0	0	0	0	0	0	0	0	0	0	0
99%	0	0	0	0	0	0	0	0	0	0	0	0	0

## Channel Characteristics

The average channel slope generally decreases in the downstream direction throughout the basin (Figure 8). The O'Neill Diversion Weir is located in a transition zone where a break in slope occurs from an average channel slope of .0039 to .0022.



**Figure 8.** Average channel slope of the Santa Margarita River (blue line) developed from 5-yr water surface elevation from West (2000) hydraulic model. Note the break in slope near where the O'Neill Diversion Weir is located.

The active channel represents the unvegetated channel area that transports water frequently enough that mature vegetation cannot establish. The average active, unvegetated channel width was measured on the 1997 aerial photograph to be just under 200 feet (Figure 9) for about a 1.5 mile reach in the vicinity of the O'Neill Diversion Weir. Immediately upstream of the O'Neill Diversion Weir the active channel is slightly wider, about the width of the weir which is 278 feet.

The bankfull channel includes the active channel and low elevation vegetated areas (active floodplain) that are between the binding terraces or valley walls. The bankfull channel is the area where the majority of sediment is transported by the river, and, therefore, the bed is most often reworked. Flood flows can overtop the banks of the bankfull channel and spread out onto the floodplain. The average bankfull channel width

is more than twice that of the active channel, about 550 feet. Side channel paths that convey a small amount of water and pass through the vegetated active floodplain can be seen in this reach in the 1997 aerial photograph and were also observed in the field in June 2004. Note that the area of bankfull channel that is vegetated is highly dependent on flood occurrence and, therefore, changes over time.

Based on the slope-discharge relationship in the vicinity of the O'Neill Diversion Weir (using 5-year discharge), the Santa Margarita River active channel can be characterized as in the transition zone between a braided and meandering channel (Knighton, 1998). Based on aerial photography, the active channel in this reach is slightly meandering for a short distance upstream of the O'Neill Diversion Weir until the confluence with De Luz Creek (Figures 9 and 10). Upstream of the confluence, the channel is more confined and runs fairly straight with the configuration of the valley planform (Figure 11).

A thorough field inventory or historical analysis was not done, but based on the flood history and aerial photographs from 1993, 1994, and 1997 and ground photographs from 1999, some likely qualitative conclusions can be drawn about influence of vegetation on channel conditions. If a large flood occurs such as the flood of record in 1993 or the near 10-year flood in 1995 and 1998, the majority of the vegetation in the bankfull channel appears to be washed out. During periods when no large floods occur, the floodplain area begins to be vegetated, as it is currently in 2004, and only the low flow channel remains unvegetated (see Figures 6, 10 and 11). Therefore, it is estimated that vegetation may help stabilize the sediment stored in the river channel during low and intermediate flows, but during floods it is scoured out and does not have a large influence on channel hydraulics or availability of sediment to be transported downstream.

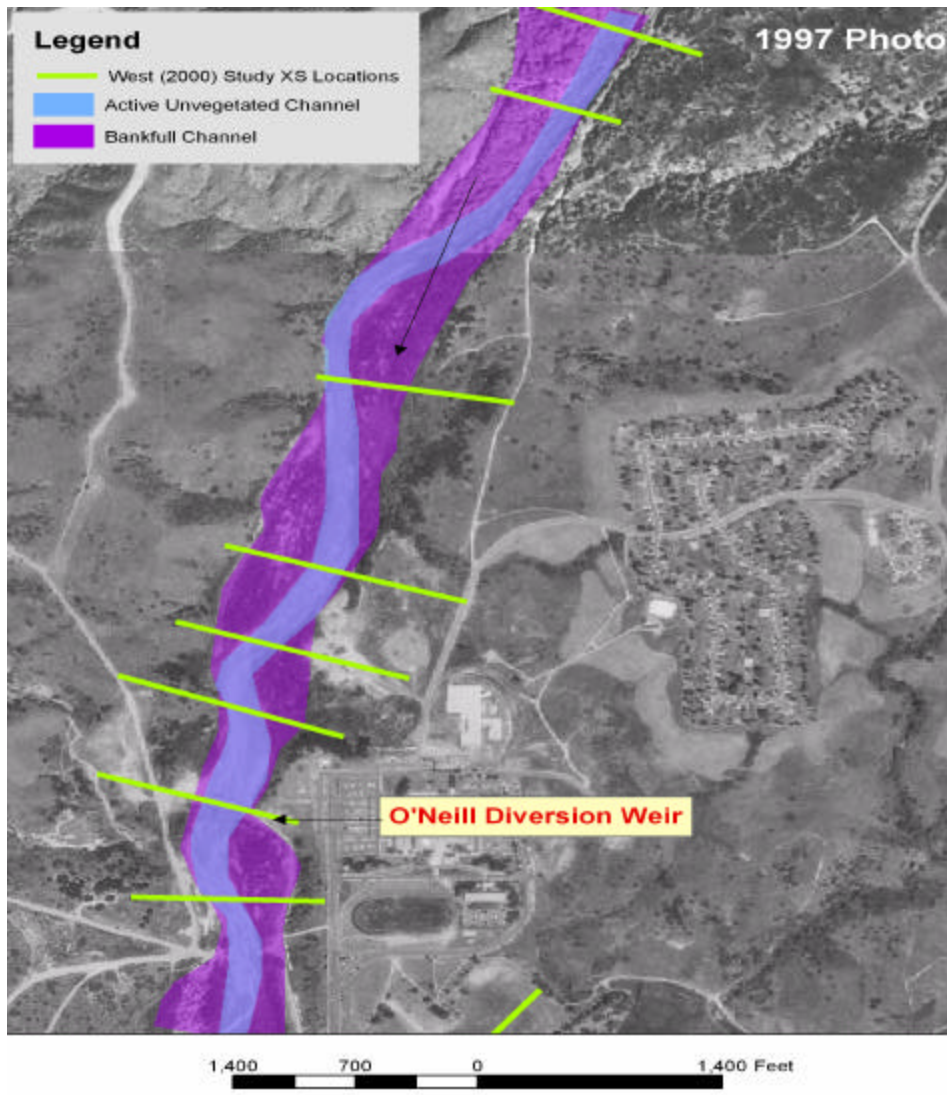


Figure 9. Mapping of active and bankfull channel on 1997 aerial photograph.



Figure 10. Looking upstream at active channel and floodplain about 1/2 mile upstream of the O'Neill Diversion Weir.



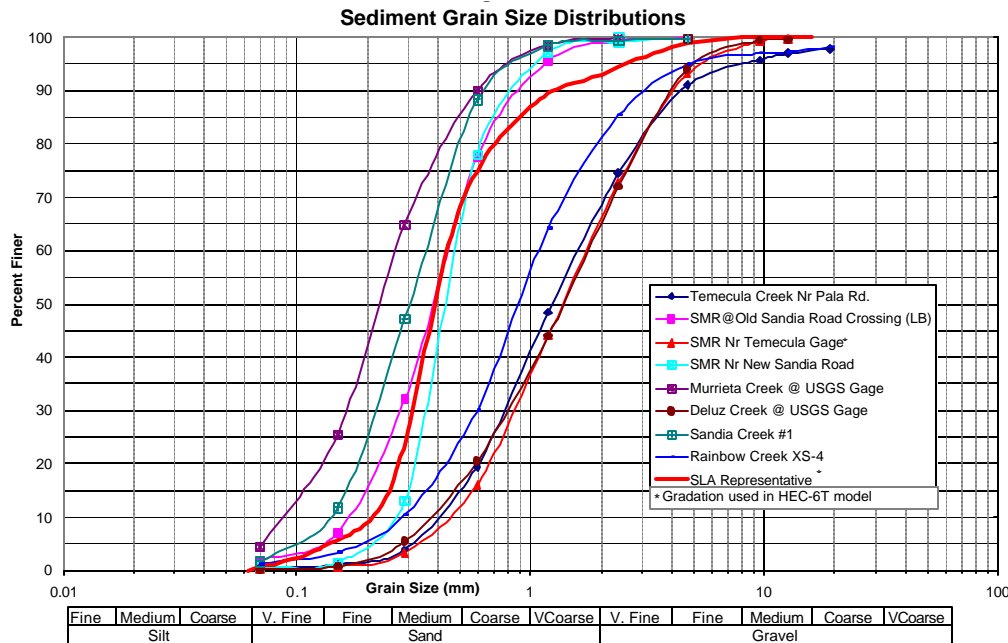
Figure 11. Looking upstream at Santa Margarita River at road crossing located just upstream of confluence with DeLuz Creek.

# Sediment Characteristics

Sediment is supplied to the O’Neill Diversion Weir reach by reworking sediment in the upstream channel bed and transporting it downstream, from upstream tributaries entering the Santa Margarita River, and by erosion of river banks on either side of the channel to the extent it occurs during floods.

## Bed-Material Sizes

Based on sediment data collected by Simons, Li & Associates (SLA) in January 1994 (1995 Report), the active channel bed in the vicinity of the O’Neill Diversion Weir is generally composed of sand-sized sediment with a small percentage of fine gravel and coarse silt. Downstream of river mile 24.3, West (2000) determined a representative particle size-gradation from the SLA (1995) study was appropriate to characterize the sediment being mobilized along the river bed. Based on the representative particle size gradation, the median particle diameter ( $d_{50}$ ) of the bed material is 0.4 mm, which represents a coarse-sized sand particle (Figure 12). During a site visit by Reclamation in June 2004, visual observations of the bed indicated that the SLA (1995) representative sediment gradation was still applicable to current conditions and appropriate for the level of analysis being accomplished for this study. Upstream tributaries and the upper portion of the Santa Margarita River have slightly coarser bed-material sizes due to the more naturally constricted river channel in these reaches.



**Figure 12. Particle size gradations measured in various sections of the Santa Margarita River and tributaries by SLA (1995). Note that the thick red line labeled “SLA representative” was used by WEST (2000) to characterize the bed particle size gradation in the reach including the O’Neill Diversion Weir for sediment modeling efforts.**

## Sediment Transport Capacity

Camp Pendleton is proposing a new structure that allows flushing of sediment during floods to maintain the natural stream gradient upstream of the O'Neill Diversion Weir. To predict the capability of the proposed structure to flush sediment, the existing sediment transport capacity of the river must be understood. Previous sediment transport modeling, a decrease in average channel slope, stream power computations, and natural topographic constrictions were evaluated. This indicated the reach of river containing the O'Neill Diversion Weir is prone to deposition.

### Existing Sedimentation Issues

Currently, sediment has completely filled the reservoir area upstream of the O'Neill Diversion Weir. Local accounts note that even after dredging has occurred in the past, sediment quickly refills the reservoir area and diversion channel bed. This indicates that the trap efficiency of the reservoir created by the weir is near or at 100%. Because the majority of flow is currently being diverted, a channel is scoured upstream of the diversion channel down to the elevation of the diversion gate opening. The sand-sized sediment trapped upstream of the weir are not cohesive and are easily eroded and transported by the river. As water is diverted, sediment is also continuously diverted. Because there is plenty of sediment stored in the reservoir, even when dredging of the canal occurs, sediment is quickly re-transported into the canal and fills it again. This process will continue under the current situation of limited sluicing and diversion of the majority of flow. When large floods occur, it can only be sluiced in a small section of the weir, and therefore the majority of sediment remains as a source for being diverted into the canal during lower flow periods during the remainder of the year.

### Flows that Mobilize Bed Sediment

An indicator of when the majority of sediment is mobilized on the Santa Margarita River can be computed based on the gage data at the Ysidora gage. By taking the square or cube of the annual peak discharges, it was estimated that over 95% of sediment transport occurs during the 10-year flood and greater, and over 99% of sediment transport occurs during the 5-year flood and greater. The exception is where sediment is stored upstream of the weir and can be mobilized at lower flows due to the gradient created between the diversion gate and the river channel when the majority of river flow is diverted rather than transported downstream.

### WEST (2000) Sediment Model Results

The following paragraphs provide documentation of the WEST (2000) sediment transport model results, which were obtained using the sediment transport model HEC-6T (USACE, 1993b; Thomas, 1999). WEST (2000) notes that these results should be verified by collecting additional topographic and sediment data, because only limited data were available to work with in this basin. Based on the model results using the Yang sand and gravel sediment transport equations (Yang, 1973, 1984), WEST (2000)

estimated that the average annual sediment load at the Interstate 5 crossing (Station 3075) of the Santa Margarita River is 36,000 to 51,000 tons per year.

The West (2000) report noted that “from Station 49000 upstream past the O’Neill Diversion Weir to Station 56780, the model results show channel deposition (up to 2 feet of aggradation) resulting from most of the flood events. The exception to this statement is the scour shown in the cross section just downstream of the weir at River Station 54830. Interestingly enough, the average bed elevation is not substantially lowered here except for the 100-year balanced hydrograph.”

From the Basilone Bridge crossing (Station 49000) downstream to Station 18460 WEST (2000) results show the channel is “in a degradational state during all flood hydrographs greater than the 2-year event. The average bed elevations are lowered by as much as five feet in this reach. It appears that the principal reason for the degradation is the construction of the flood protection works along the left bank of the river. The increase in variability is due not only to the increased frequency of cross sections but also to the increased detail of the cross sections (greater number of ground points) in this reach.”

From the Interstate 5 bridges upstream through the Ysidora Basin (River Station 18460) the WEST (2000) model results show “the channel as being relatively stable. A few notable features in this reach are deposition between the Interstate 5 and Stuart Mesa Road bridges, scour at the latter bridge, and some degradation in the area of “the narrows” (approximately cross section 1100 to 14640). With the exception of changes in two cross sections during the 100-year hydrograph, neither aggradation nor degradation surpassed 2 feet in this reach.”

From the Ocean to Interstate 5 (River Stations 0 to 3075), the WEST (2000) model predicts “lowering of the average bed elevation (degradation) ranging from 0 for the 2-year hydrograph to 3 feet from the 100-year hydrograph. These results may be suspect because: (a) the HEC-6T model cannot simulate tidal influence in this reach (unsteady flow), (b) cannot simulate two-dimensional flow patterns, (c) does not simulate the added resistance to erosion of existing cohesive sediments in the estuary; and (d) the current model does not include non-cohesive sediment which may deposit in the estuary. However, the degradation computed during these flow events could also be filled in during subsequent lower flows. Results for this reach also show that the 10- through 100-year events cause erosion at the bridge opening due to the constriction of flow.”

### **Stream Power**

The O’Neill Diversion Weir is located in a transition area where the average channel slope changes from 0.0039 to 0.0022. As slope decreases in the downstream direction, the ability of the river to transport sediment can decrease, which may result in the area being naturally prone to deposition. However, as discharge increases in the downstream direction from tributaries or runoff, an increase of the river’s ability to transport sediment can increase. Stream power is a computation that is used to indicate the relative balance of energy in the river on a reach scale level. The higher the stream power computation, the greater the sediment transport capacity.



Both total and unit stream power showed a decrease in the downstream direction from river mile 24 to the mouth (Figure 13). It is interesting that just downstream of the diversion weir the unit stream power is noticeably lower than the trendline. Total stream power is computed by multiplying the reach-averaged discharge (Q) by reach-averaged slope (S) for given locations along the river. Because the discharge does not vary greatly in the lower Santa Margarita River, this computation generally shows a reduction in stream power in the downstream direction to the reduction in slope.

Using the existing WEST (2000) hydraulic model, unit stream power can also be computed by multiplying the depth-averaged channel velocity (V) by the average reach slope (S) along the river. In addition to accounting for changes in discharge and slope, unit stream power also takes into account changes in channel morphology and bed roughness because velocity is dependent on these parameters. The topography used in the hydraulic model is not very detailed in the O'Neill Diversion Weir area, but the computations should show if there is any relative trend in unit stream power. Unit stream power was computed for the 5-year flood because it is the lowest flood frequency developed, and should represent something slightly greater than a typical bankfull condition. Bankfull floods can be a good indicator of sediment transport capacity because they frequently occur and contain enough water to fill the active channel and begin reworking sediment along the channel bed.

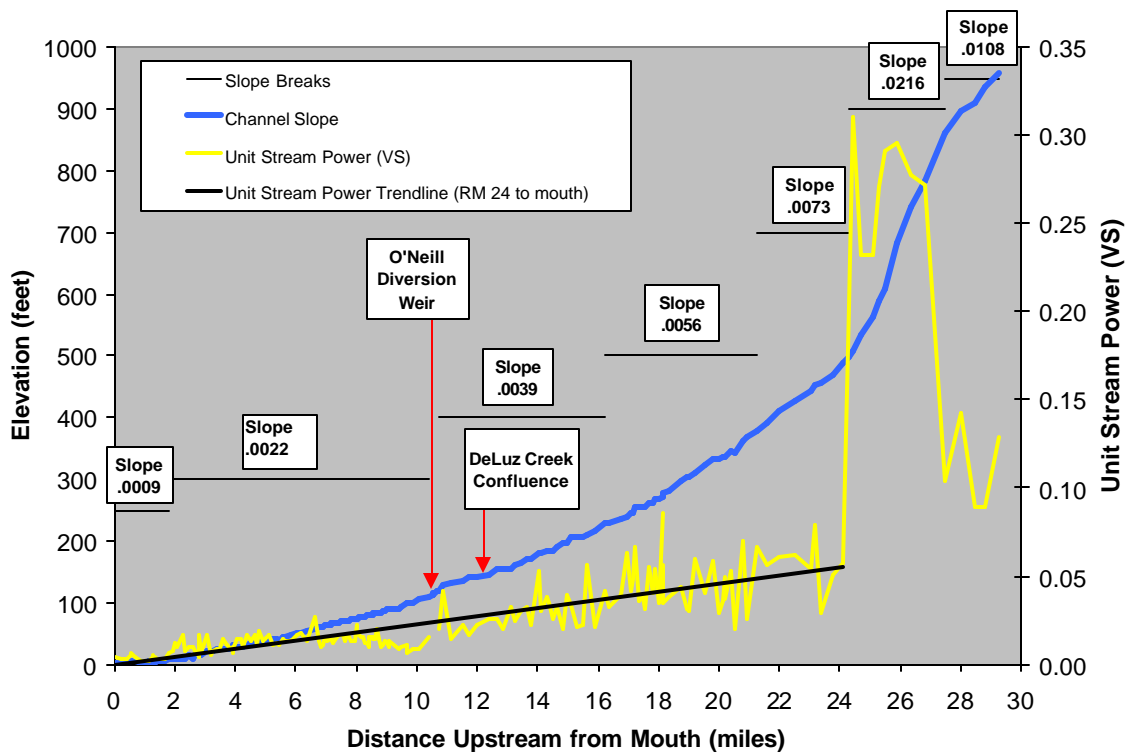


Figure 13. Unit stream power, an indicator of sediment transport capacity, plotted against average channel slope for the Santa Margarita River.

## Reservoir Sediment

In an ideal case the amount of sediment trapped in the reservoir can be determined by using a combination of pre-dam survey data (natural river bed), existing survey data of sediment deposit, and drilling data when needed to determine depth to original channel bed from the top of deposit. Because this data does not exist for this project, the amount of sediment trapped by the weir was estimated for this report by determining where the existing sediment delta intersects with the natural river bed. Based on these computations, the sediment stored upstream of the weir is located within 3600 feet upstream and is estimated to range between 84,000 yd<sup>3</sup> to 102,000 yd<sup>3</sup> (Figure 14). About 50% of the total sediment volume is estimated to be located in the first 1000 feet upstream of the weir. The sizes of sediment trapped upstream of the weir were visually observed to be similar to the representative sediment particle gradation described by SLA (1995), mostly coarse sand sizes (see Figure 12). Assuming a specific weight of the reservoir sediment of 100 lb/ft<sup>3</sup>, this volume would be roughly 2 to 4 times the average annual sediment load at the I-5 bridge crossing based on estimates provided by WEST (2000) modeling efforts.

Estimate of Sediment Stored Upstream of O'Neill Diversion Weir

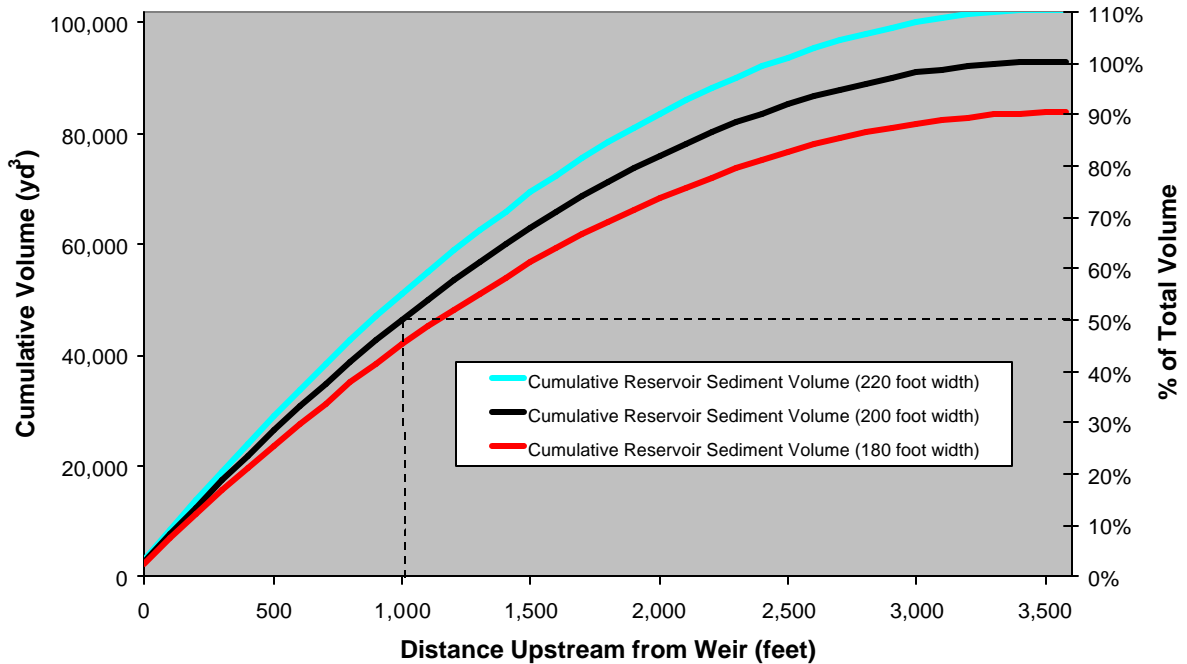
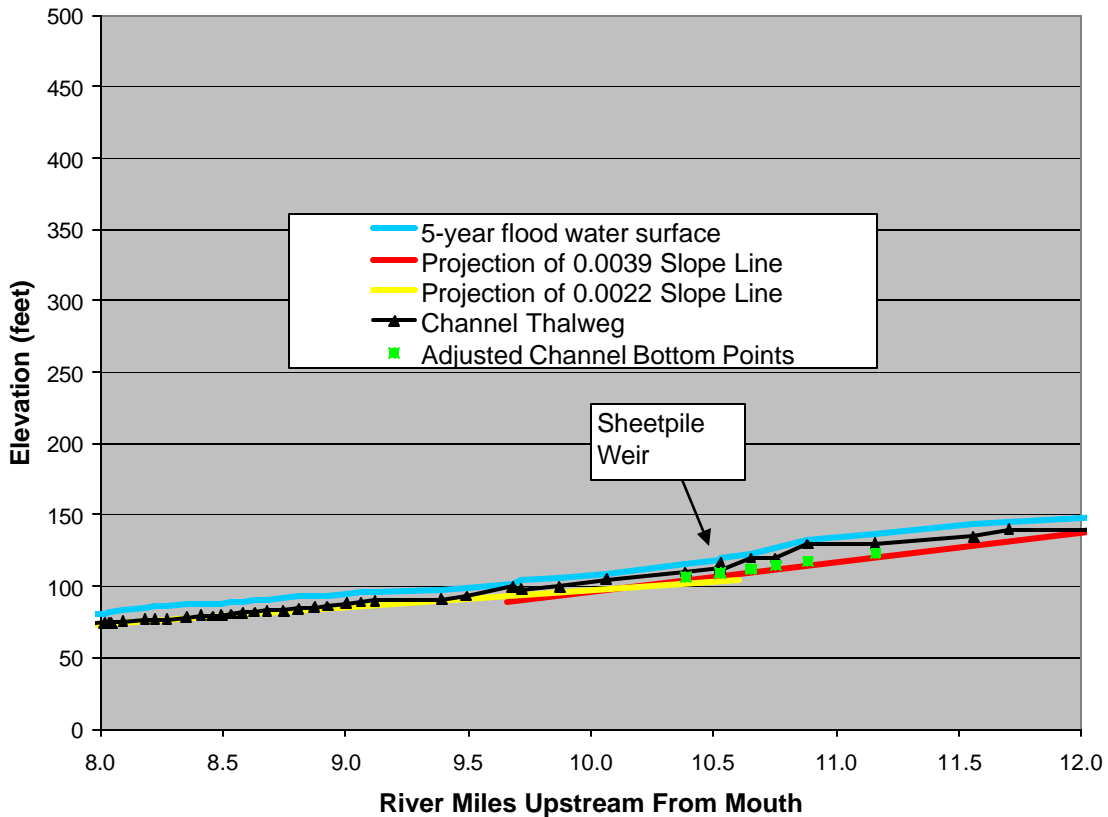


Figure 14. Estimated volume of sediment stored upstream of the O'Neill Diversion Weir plotted by distance upstream from weir. Three assumptions of active channel width were made to provide a reasonable range of possible volume estimates.

In 1993 during the flood of record, the downstream channel was estimated to have scoured down to elevation 109 feet. The channel slope in the reach downstream of the weir is .0022, and upstream of the area influenced by the weir is .0039. By extending these two lines to find their point of intersection, a rough estimate of the natural channel bed elevations can be made (Figure 15). By projecting the upstream slope downstream to the weir, the predicted natural river bed elevation was at about elevation 109, which matches local accounts of where the bed was scoured down to after the 1993 flood. The existing bed is slightly higher than the 0.0022 slope projection line (yellow) for a short distance downstream of the weir. This may represent a transitional area where the slope gradually flattens out to 0.0022. It may also be an artifact of connecting the more detailed survey data downstream of river mile 9 with the 5-foot contour data that exists upstream of this point. Although there is limited detail in the channel, the 5-foot contour data is likely adequate to estimate the average slope of the channel over long reaches. The cross sections that would likely need to be adjusted to represent the natural channel bed extend about 3400 feet upstream of the weir (see green markers in Figure 15).



**Figure 15. Estimate of natural river channel prior to the construction of the O’Neill Diversion Weir. Also shown are sections where the channel bed elevation was lowered to model the sluiced channel condition after installation of the proposed Obermeyer weir structure.**

Another method developed by Strand and Pemberton (1982) is to estimate the top of delta slope based on one-half of the natural bed gradient and determine where that slope would intersect the existing bed based on the height of the structure above the bed. Based on

this method the delta slope would be .00196, and therefore the majority of trapped sediment would extend about 3600 feet upstream. Approximately 575 feet of the delta in the low flow channel upstream of the dam was measured in a June 2004 survey. This slope was .001837, very close to the Strand and Pemberton prediction, and would result in a sediment area extending 3800 feet upstream.

Based on the above methods, it is estimated that the majority of sediment deposition occurs within about 3600 feet upstream of the weir. The volume of trapped sediment can be estimated by multiplying the upstream extent of deposition, a typical active channel width, and the depth of sediment above the natural bed. The depth of sediment at the weir was computed by taking the difference between the measured channel bottom from the June 2004 survey at 116 feet and the estimated pre-dam channel bottom at 109 feet (7 feet). At the upstream end the depth of sediment is near zero. The average active channel width was estimated as 200 feet and is fairly consistent upstream of the weir in this reach. To provide a range of estimates, a volume based on a 180 foot and a 220 foot active channel width were also computed.

## Hydraulic Modeling

A one-dimensional hydraulic model, HEC-RAS Version 2.2 (USACE, 1998b) was developed by WEST Consultants for a study of the Santa Margarita watershed (WEST, 2000). This model was provided to Reclamation for use in the analysis of the proposed diversion weir modifications. The HEC-RAS model was not calibrated or validated by WEST because there are no known historical flood elevations or measured data to utilize. Therefore, the biggest concern in estimation of hydraulic properties and flood stage using the existing model developed by WEST (2000) is the lack of detailed topographic data to represent existing conditions in the vicinity of the O'Neill Diversion Weir and the lack of data of the natural (pre-dam) topography.

The existing conditions hydraulic model would be improved by obtaining more detailed survey data of the existing topography. New photogrammetric data was collected in June 2004, but was not yet available at the time of this report. This data could be used to further refine the model in future design phases, and would also provide a baseline condition of topographic conditions during full sediment fill prior to the new structure. The photogrammetric data was collected during a time period when vegetation was extensively growing in the channel. This may limit the ability of the data to provide detailed topography in the channel and floodplain areas. This data or additional ground survey data would be very useful in the future during monitoring of the new structure to improve operations and evaluate its effectiveness in sluicing sediment. Estimation of pre-dam conditions had to be accomplished using engineering judgment and relationships developed from similar study areas.

## Existing Conditions Model

WEST (2000) notes that although the roughness values were not calibrated, they “are reasonably conservative for determining water surface elevations for flood inundation purposes.” Roughness values used in the model appear reasonable based on a scoured channel (fairly unvegetated) condition. This assumption of a scoured channel is appropriate for 10-year and higher floods based on aerial photographs taken following the near 100-year flood in January 1993 and after the 1998 flood which show very little vegetation remaining in the channel. WEST (2000) did state that increasing roughness could impact water surface elevations by increasing them 3 to 4 feet, which may be applicable for conditions where multiple years without major floods occur such that mature vegetation can establish in the floodplain.

The cross section geometry created for the model in the general vicinity of the O'Neill Diversion Weir was generated by WEST from topographic maps with 5-foot contour data created from photogrammetric data collected in 1994. WEST (2000) notes that the 5-foot contour data used for about 1.4 miles downstream of O'Neill Diversion Weir may have

some deficiencies in its accuracy, and upstream of the weir it is unknown what the accuracy of the 5-foot contour data is. The cross sections have widely spaced ground points and for most cross sections a constant elevation across the channel and overbank area occurs. This would indicate the 5-foot topography does not accurately represent the detailed geometry of the channel. In addition, in vegetated overbank areas the elevations may characterize the top of vegetation rather than the actual ground elevation. This is very common in data created from aerial photography (photogrammetry) where vegetation is dense and/or water is flowing in the channel area.

When compared to June 2004 survey data collected in the low flow channel at a location about 600 feet upstream of the existing weir, the cross section developed from the 5-foot contour data showed a constant channel bed about 3 feet higher than thalweg (low point) of the existing low flow channel. Average bed elevation created by 5-foot contour data may be somewhat comparable to actual conditions, but this can't be verified without more survey data. If the 5-foot contour data in the hydraulic model represents a higher bed elevation than exists, it would result in a higher flood stage for a given model run. For low flows, lack of detail in the channel area makes it difficult to predict the hydraulics with confidence.

## Proposed Conditions Model

The existing (WEST Consultants) HEC-RAS model was adjusted to develop the channel topography for proposed conditions with the sheet pile weir replaced by Obermeyer gates and a sluiced channel condition. This means that during each flood, it is assumed the Obermeyer gates have been utilized to flush the sediment. Modeling was also done as a sensitivity test with the reservoir filled with sediment, which assumes the Obermeyer gates were closed for a long enough duration during previous floods to cause the reservoir to fill to capacity. These adjusted model results result in a maximum of 5.6 feet +/- 1 foot drop over the proposed structure when the Obermeyer gates are closed (up) at elevation 117.1 at the 5-year flood level (energy grade drop is slightly less) (see Figures 16 to 19 and Tables 3 and 4).

The existing weir is estimated to be able to cause deposition upstream for a maximum of 3600 feet. Therefore, cross sections in this range were lowered to the estimated "sluiced" channel condition that will be maintained under the proposed condition based on the slope of the upstream channel that is not significantly influenced by the structure (see Figure 15). To model the natural topography restored, geometry had to be estimated using best available data. Typically, a slightly trapezoidal cross section was generated by lowering a portion of the existing bed represented as a constant elevation. The cross sections immediately downstream of the new weir were also lowered down to 109.2 ft to reflect the elevation of the proposed apron. The sheet pile weir was then replaced with Obermeyer gates for a distance of 280 feet across the channel, and a 10-foot wide sluice gate at the left end of the Obermeyer gates. Side slopes on either side of the structure were developed from existing topography.

The relative drop in water surface elevation is dependent on the input geometry and roughness values in the model. Given the fixed elevation of the downstream apron, and the assumed scoured out channel upstream, a relative drop in elevation over the proposed structure was computed for the 5- to 100-year flood. Results were calculated assuming the sluice gates on the 10-foot channel were fully open, and the Obermeyer gates were either fully open or fully closed.

These adjusted model results result in a maximum of 5.6 feet drop over the proposed structure when the Obermeyer gates are closed (up) at 117.1 at the 5-year flood (energy grade drop is slightly less). This drop represents the approximate change in water surface from just upstream of the structure before it begins to pass through critical depth, to the section just downstream of the structure where the concrete apron would be located. Given the limitations of the one-dimensional model, this result may be +/- 1 foot within the absolute water surface elevation drop over the structure. The upper limit of hydraulic drop should not be expected to exceed the height of the structure, which is about 7.4 feet (117.1 – 109.75ft). To determine the highest possible drop in elevation over the structure, another model was run with the upstream channel filled in, the new structure in place with gates up, and the downstream channel scoured down to the apron. This resulted in the same magnitude of hydraulic drop in water surface across the structure of about 6 feet for the 5-year flood. It is recommended that a final modeling effort be done when more detailed survey and final designs are available. The new structure causes a maximum backwater effect of about 2,000 feet when the Obermeyer gates are up (closed). A small backwater is still created when the gates are down (open), likely due to the constriction of the channel from about 550 feet down to 300 feet in this area.

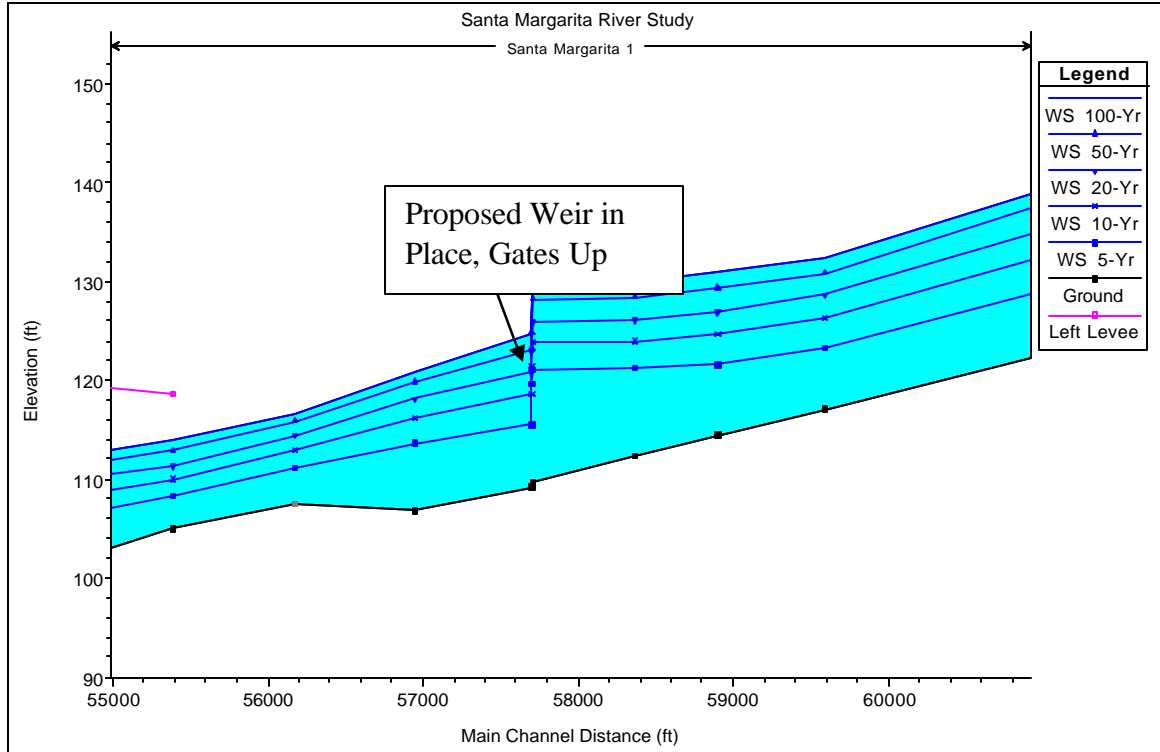


Figure 16. This graph shows the water surface profile for the various floods modeled with the sluiceway open and the 280 feet of Obermeyer gates up at elevation 117.1 feet.

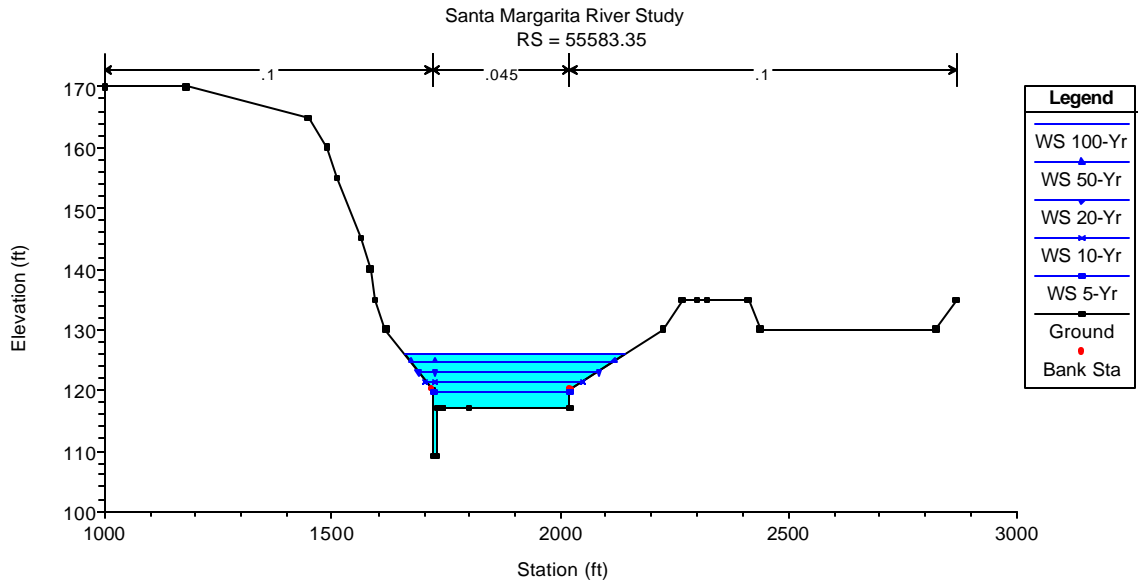


Figure 17. Plot of proposed weir section modeled with 10-foot wide open sluiceway and Obermeyer gates up (closed) resulting in a weir elevation at 117.1.



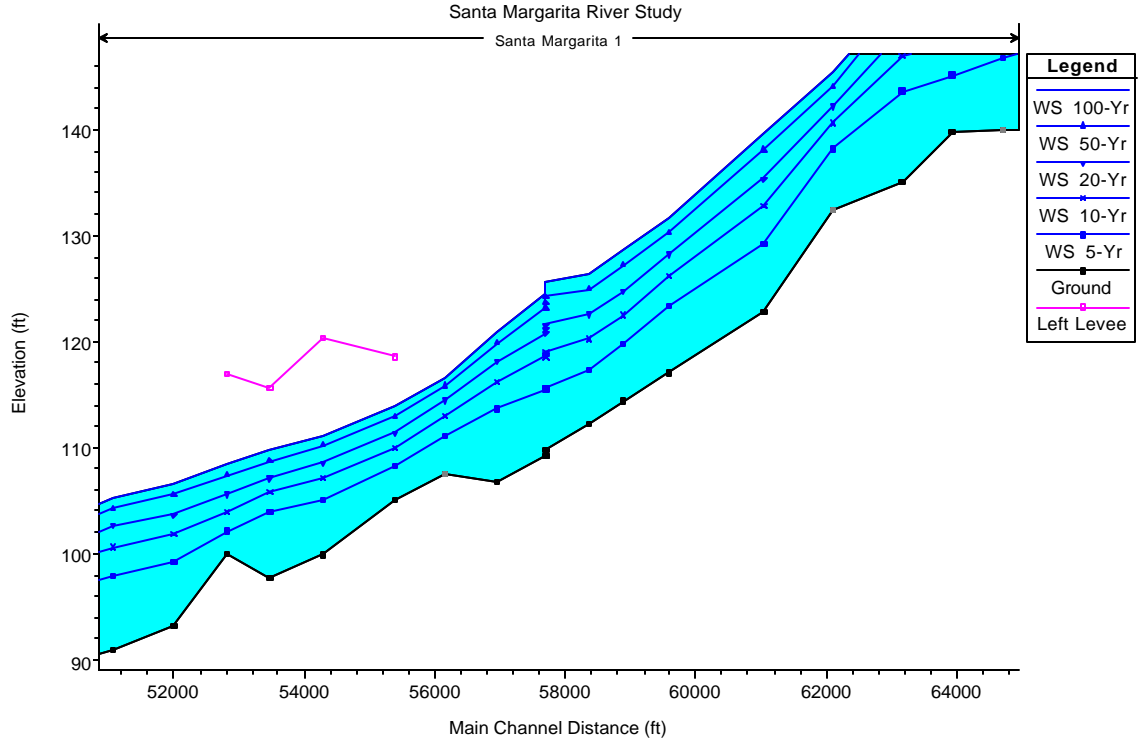


Figure 18. Profile plot of floods with Obermeyer gates down. A very small backwater is created from the narrowing of the active channel down to 300 feet even when the gates are down.

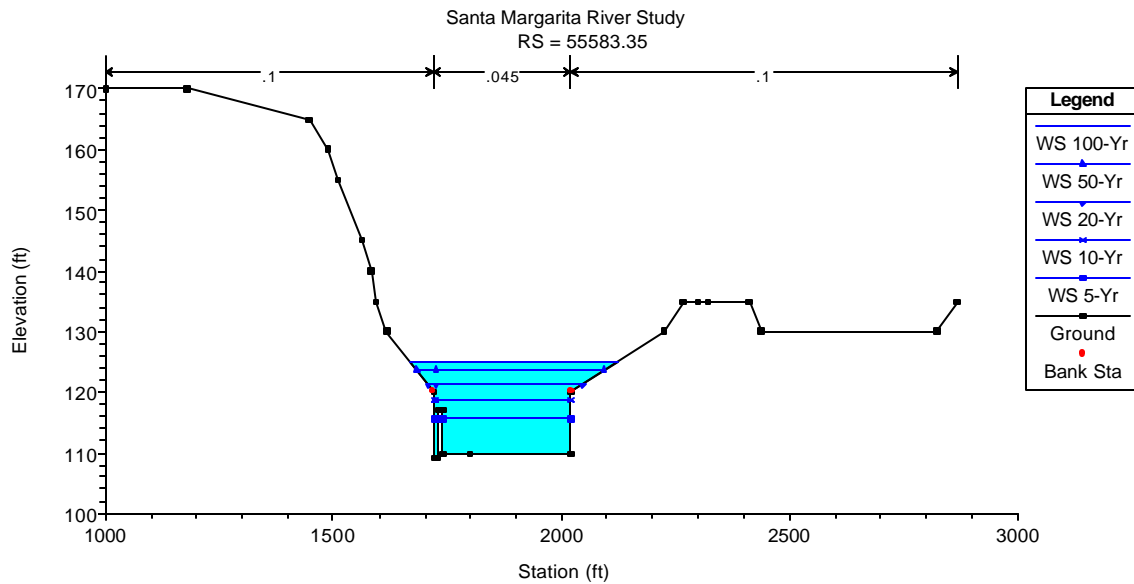


Figure 19. Plot of proposed weir section modeled with 10-foot open sluiceway and Obermeyer gates down (open) resulting in a weir elevation at 109.75.

**Table 3. Model results with Obermeyer Gates Up at Elevation 117.1, Sluiceway Open at Elevation 109.25.**

River Station	Location	Profile	Q Total (ft <sup>3</sup> /s)	Min Ch Elevation (ft)	W.S. Elevation (ft)	Depth (ft)	E.G. Elevation (ft)	Velocity Channel (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Channel	Drop (ft)
55587	Just Upstream	100-Yr	46000	109.8	129.6	19.8	130.1	5.8	8946.2	614.2	0.2	
55587	Just Upstream	50-Yr	37500	109.8	128.2	18.4	128.6	5.2	8071.0	591.5	0.2	
55587	Just Upstream	20-Yr	26000	109.8	125.9	16.1	126.2	4.2	6778.4	556.3	0.2	
55587	Just Upstream	10-Yr	17000	109.8	123.8	14.0	124.0	3.2	5633.6	523.1	0.2	
55587	Just Upstream	5-Yr	8000	109.8	121.1	11.3	121.2	2.0	4280.5	480.9	0.1	
55583	Weir	100-Yr	46000	109.3	125.9	16.6	129.8	16.2	3245.9	481.4	1.0	
55583	Weir	50-Yr	37500	109.3	124.8	15.5	128.3	15.3	2729.0	447.2	1.0	
55583	Weir	20-Yr	26000	109.3	123.0	13.8	125.9	13.8	2005.2	394.2	1.0	
55583	Weir	10-Yr	17000	109.3	121.5	12.2	123.8	12.1	1430.9	346.4	1.0	
55583	Weir	5-Yr	8000	109.3	119.6	10.4	121.0	9.6	834.6	300.9	1.0	
55579	Apron	100-Yr	46000	109.2	124.6	15.4	126.5	12.2	5719.0	568.6	0.6	5.0
55579	Apron	50-Yr	37500	109.2	123.1	13.9	124.8	11.3	4912.6	551.8	0.5	5.1
55579	Apron	20-Yr	26000	109.2	120.8	11.6	122.1	9.8	3660.0	524.6	0.5	5.1
55579	Apron	10-Yr	17000	109.2	118.5	9.3	119.4	8.0	2702.9	352.9	0.5	5.3
55579	Apron	5-Yr	8000	109.2	115.5	6.3	116.0	5.8	1684.3	315.3	0.4	5.6

**Table 4. Model results with Obermeyer Gates Down at Elevation 109.75, Sluiceway Open at Elevation 109.25.**

River Station	Location	Profile	Q Total (ft <sup>3</sup> /s)	Min Ch Elevation (ft)	W.S. Elevation (ft)	Depth (ft)	E.G. Elevation (ft)	Velocity Channel (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Channel	Drop (ft)
55587	Just Upstream	100-Yr	46000	109.8	125.8	16.0	126.6	7.5	6684.6	553.6	0.4	
55587	Just Upstream	50-Yr	37500	109.8	124.2	14.4	124.9	6.9	5838.0	529.1	0.3	
55587	Just Upstream	20-Yr	26000	109.8	121.7	11.9	122.2	6.0	4553.1	489.7	0.3	
55587	Just Upstream	10-Yr	17000	109.8	119.0	9.2	119.5	5.3	3318.5	445.5	0.3	
55587	Just Upstream	5-Yr	8000	109.8	115.7	5.9	116.0	4.3	1939.5	383.4	0.3	
55583	Weir	100-Yr	46000	109.3	125.0	15.7	126.5	10.1	4894.6	454.3	0.5	
55583	Weir	50-Yr	37500	109.3	123.6	14.3	124.9	9.1	4287.8	411.1	0.4	
55583	Weir	20-Yr	26000	109.3	121.3	12.0	122.2	7.6	3424.2	340.3	0.4	
55583	Weir	10-Yr	17000	109.3	118.8	9.6	119.4	6.4	2657.2	300.9	0.4	
55583	Weir	5-Yr	8000	109.3	115.6	6.4	116.0	4.7	1716.0	291.4	0.3	
55579	Apron	100-Yr	46000	109.2	124.6	15.4	126.5	12.2	5719.0	568.6	0.6	1.2
55579	Apron	50-Yr	37500	109.2	123.1	13.9	124.8	11.3	4912.6	551.8	0.5	1.1
55579	Apron	20-Yr	26000	109.2	120.8	11.6	122.1	9.8	3660.0	524.6	0.5	0.9
55579	Apron	10-Yr	17000	109.2	118.5	9.3	119.4	8.0	2702.9	352.9	0.5	0.5
55579	Apron	5-Yr	8000	109.2	115.5	6.3	116.0	5.8	1684.3	315.3	0.4	0.2

# Sediment Considerations for Future Diversion Operations

This section discusses sediment considerations regarding the initial flushing of sediment and the design and operation of the proposed sluiceway and Obermeyer weir structure.

## Initial Flushing of Reservoir Sediment

A general description of the sedimentation processes in the reservoir area following dam removal is given by Doyle et al. (2003), which is a modification of the geomorphic head cut model of Shumm (1984). This description can be applied to how the initial flushing of sediment during the first flood following construction of the Obermeyer gates may occur. This would also be applicable for future sluicing operations if sediment is allowed to build up behind the weir. The various stages are shown in Figure 20 and a summary of the descriptive model of Doyle et al. follows. The figure shows sluicing originating from the center of the channel, where as in the case of the O'Neill Diversion Weir it would begin from the left side of the channel near the intake structure.

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 highest of any stage. Depending upon the grain sizes present in the reservoir and the magnitude of the initial drawdown, this erosion may proceed as a head cut, or may be primarily fluvial. The erosion is not expected to cut below the original bed elevation. The initial width of the channel formed by this erosion will be governed by the stability of the material in the reservoir.

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

Stage E. Sediment from the upstream reach starts to be supplied to the previously inundated reach. Some of this sediment is deposited in the reach as the degradation and widening processes have reduced the energy

slope within the reach. Some additional widening may occur during this stage, but at a reduced rate as compared to Stage D.

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

The width of the erosion in the reservoir will be of similar width to the natural river channel. Therefore, some of the sediment in the reservoir may remain in the reservoir for a long period of time and perhaps indefinitely. The river may migrate over the reservoir area and eventually erode most of the stored sediment, but this process may be slow.

If the existing reservoir sediment is sluiced downstream following construction of a new structure with Obermeyer gates, it is of interest to know where the sediment would likely deposit in the downstream river channel. If a 10-year or great flood occurs while the Obermeyer gates are fully open, the sediment would likely be quickly eroded and transported downstream (Morris and Fan, 1998). Existing information indicates that the sediment would initially deposit in the approximately one-mile reach just downstream of the dam where the floodplain widens substantially and the river bed is prone to deposition. During subsequent floods, the sediment would eventually be flushed downstream. If the reaches downstream of Station 18,460 are degradational as the WEST (2000) model predicts, the sediment should gradually be transported to the mouth but may cause temporary aggradation until they are completely flushed out. Detailed assessment of the downstream impacts from the initial flushing of reservoir sediment was beyond the scope of this study.

At the O'Neill Diversion Weir and at another location about 500 feet downstream, two topographic constrictions exist that cause "pinch points" in the bankfull channel (see Figure 9). These constriction points may cause some upstream backwater to occur during flood flows. If backwater does occur, these upstream areas would be prone to more deposition than the more un-constricted, wider areas of channel, particularly during bank full floods and larger. Interestingly, the computation of unit stream power does show a locally lower than normal area just downstream of the O'Neill Diversion Weir (see Figure 13).

According to the WEST (2000) model results, about one-mile downstream of the weir the river switches to a degradational state until about 3 miles upstream of the mouth, where it switches to a stable channel and then is slightly degradational at the mouth. This is not apparent in the stream power results, which are based more on average reach conditions. The WEST (2000) report notes that flood protection structures and bridges may be resulting in the model results showing degradation. Further evaluation would be necessary to confirm these model predictions and evaluate the potential of the river downstream of the O'Neill Weir to transport sediment.

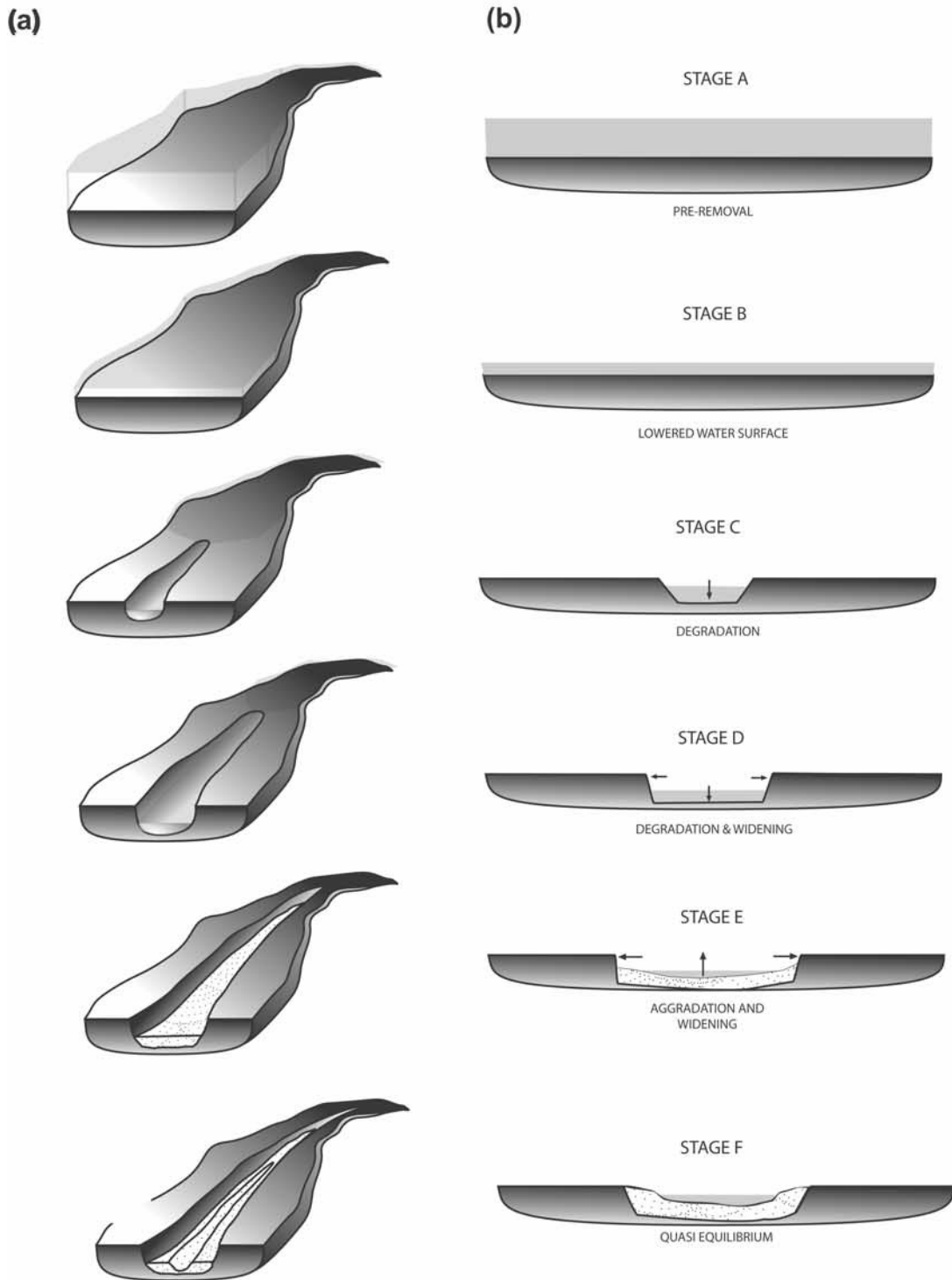


Figure 20. Schematic description of reservoir erosion process through delta deposits, from Doyle et al. (2003). (a) oblique view, (b) cross section view. This figure shows sluicing originating from the center of the channel, whereas in the case of the O'Neill Diversion Weir it would begin from the left side of the channel near the intake structure.

## Diversions and Sluicing Operations

A 10-foot wide sluiceway with two gates is proposed to be installed adjacent to the diversion channel intake with an invert elevation of 109.25 ft. The diversion channel intake will have a small sill at the intake location. The sill is proposed to be 1-foot higher than the invert elevation of the sluiceway to discourage sediment from entering the diversion channel. The Obermeyer weirs will be installed across a 280-foot section, with an elevation of 109.75 when the gates are down and a weir elevation of 117.1 ft when the gates are up. The question has arisen as to how the design can maximize sediment sluicing capability in the main river channel while minimizing the potential of allowing sediment to enter the diversion canal.

### Low Flow Periods

About 50 % of the time the mean daily flow on the Santa Margarita River at the Ysidora gage is below 20 ft<sup>3</sup>/s, and about 80 % of the time it is below 110 ft<sup>3</sup>/s (see Table 2). The proposed diversion capability of 200 ft<sup>3</sup>/s is greater than these amounts. It is estimated that during the majority of low flow periods, the majority of water would be diverted, and only a small portion of the flow would be allowed to pass downstream, currently estimated at 3 ft<sup>3</sup>/s. For these low flow periods, the sluiceway would be sufficient for passing flow downstream and it is assumed the Obermeyer weirs would remain up (closed).

### Flood Flows

The majority of sediment stored in the channel and floodplain is mobilized during the 10-year flood and greater, which is referred to as flood flows for purposes of this discussion. If all of the Obermeyer gates are down (open) when the flood begins and there is no sediment trapped upstream, the natural channel gradient would be maintained. However, if sediment was trapped upstream since the last sluicing, a channel would be cut through the deposit and continue to erode and expand as flows increase during the flood. As long as all of the gates are down and there is high enough flow, the structure will continue to effectively sluice sediment downstream. The exact timeframe to sluice the sediment will require additional modeling or could be determined through a monitoring program following construction. As soon as a portion or all of the gates are shut (put in the upright position), the new structure will begin to trap sediment in the backwater area created upstream of the portion of channel in which the gates are up.

### Intermediate Flows

During the winter months, typical river flows range between 100 to 166 ft<sup>3</sup>/s, but small floods can occur that range between 1,000 to 3,000 ft<sup>3</sup>/s. For these intermediate flows up to the 10-year flood, a smaller width of sluicing capability would be rather beneficial.

## Sluiceway and Diversion Intake Design

The sluiceway is proposed to be located near the natural left bank of the river. The diversion channel intake should be located as close as possible to the left side of the channel that will be maintained (scoured) from the sluiceway operation. If the diversion channel intake is located too far back from the left bank of the sluiceway channel, then a new channel will essentially be scoured out between the upstream river and the intake location, as exists with the present sheet-pile operation.

The sluiceway alignment should also be parallel (i.e. sluice gates oriented normal) with the alignment of the typical river flows during which it would be utilized (low and intermediate flow periods). This will maximize the sediment transport capacity by not having any bends in the sluiceway that would cause a reduction in energy and a subsequent reduction in transport capacity. The channel downstream of the weir appears to be a depositional area. If only minimal flows are allowed to pass downstream and the majority of flow is diverted, this may accelerate the depositional trend in this area if sediment is sluiced downstream of the weir but does not have enough flow to be transported further downstream.

## Obermeyer Gate Design

If there is a large amount of sediment stored in the reservoir area, the sediment will be easily transported into the diversion canal because it has no other place to go, particularly when the majority of flow is being diverted. By maintaining the natural gradient and limiting the amount of sediment deposited upstream, the amount of sediment that can be accessed and diverted into the canal should also be limited. Floods equal to and exceeding the 10-year flood mobilize the most sediment and also are large enough to rework the entire floodplain width. To reduce the amount of sediment deposited upstream of the weir, Obermeyer gates should be installed across the complete width of channel area, about 280 feet. If the Obermeyer gates were only installed across a portion of the channel, the sediment that deposited upstream of the weir would likely be transported into the diversion canal during low flow periods when the majority of flow is being diverted.

For low flows up to a couple thousand  $\text{ft}^3/\text{s}$ , the sluiceway will likely be effective in maintaining a channel and a pathway for sediment to be transported downstream. However, as flows increase up to the 10-year flood, a flexible sluicing width would be beneficial that is larger than the sluiceway but does not extend across the entire channel width. The Obermeyer gates are available in 20-foot sections and can be installed such that they operate all as one unit or individually in sections. The question was posed as to what section breaks would be useful given the flow hydrology and diversion operations. The more sections that are built into the system, the more flexibility the operators would have in future operations designed to balance maximizing diversion with minimizing



sediment deposition. Given the 10-year and greater floods require sluicing capability across the entire channel width and low flows do not mobilize much sediment, it would make sense to design a structure with maximum width flexibility for typical intermediate flows. If it is critical during these intermediate flows to maintain a higher than natural upstream water surface elevation, smaller incremental width additions may be more useful than large sections.

Some sensitivity testing was done with the hydraulic model to determine what flows might be contained in various section widths. This cursory analysis indicated that a 20-foot Obermeyer section (in addition to the 10-foot sluiceway being open) could pass flows up to 2,500 ft<sup>3</sup>/s. By increasing the Obermeyer gate width to 40-feet the section could pass 5,000 ft<sup>3</sup>/s, and by going up to an 80-foot section the 5-year flood (8,000 ft<sup>3</sup>/s) may be contained in the opening. Based on this approach, one option would be to have incremental width options as follows:

1. 10-foot sluiceway – Low flows
2. First Obermeyer section – 20 feet (Station 0 to 20)
3. Second Obermeyer section – 40 feet (Station 20 to 60)
4. Third Obermeyer section – 80 feet (Station 60 to 140)
5. Fourth Obermeyer section – 140 feet (Station 140 to 280)

The current proposal includes two options for Obermeyer gate operations. The first option would incorporate a 280 foot section of gates that operate uniformly such that all gates would always be up or down at the same time. The second option allows one 60-foot section and one 200-foot section that can operate separately or together.

During the peer review process for this report it was noted that an operational strategy that might be worth considering for very low flow operation is periodic sluicing (as opposed to continuous sluicing of a small discharge). This might be accomplished either by manual operation or by automation of the sluice control gates. Irrigation districts along the South Platte River in northeast Colorado diverting under similar conditions suggest that this “batch” flushing technique effectively allows sediments transported to the diversion mouth to be passed on down river with a lower volume of flow than is accomplished with a continuous small sluicing bypass flow.

## Recommendations

Additional analysis would be useful for final design that would more accurately model the potential sedimentation downstream of the weir after the initial flushing of reservoir sediment. Additional modeling could also help characterize the effectiveness of sluicing during a variety of operational strategies. Modeling could consist of some simplified numerical models, or physical modeling of the

proposed structure. Physical modeling has been used in similar design cases to provide guidance for final design of intake and sluiceway areas that maximize sediment transport in the river channel while minimizing sediment diversion.

An adaptive management plan should be integrated into the implementation plan that would allow monitoring and adjustment of operations based on observations and data gathered. This may be the best tool for striking the proper balance between maximizing diversion flows and minimizing sedimentation issues.

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