APPENDIX J

Comer Dam modification alternatives

Calabazas Creek
Miller Avenue to Comer Drive

Comer Dam Vicinity

Overview and
Sediment Transport Analysis

by

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Overview

The Comer Debris Basin is located on Calabazas Creek just downstream of Comer Drive Bridge. The District has identified three main feasible alternatives for replacing the Dam. See the summary of the three alternatives on page 33 of the Main Report.

Sediment Transport Introduction

Located just downstream of Comer Drive Bridge on Calabazas Creek, Comer Debris Dam was constructed in 1973. The dam is about 13 feet deep and extends a distance of roughly 35 feet along-stream, inclined downstream at an angle of about 20 degrees to the horizontal. The bottom 7 feet of the dam (and more than half of its along-stream distance) was covered with a mixture of rocks and sediment to conform to the existing bed such that only part of the dam was exposed. Between 1973 and 1992, the reach immediately upstream of the dam and beneath Comer Drive Bridge was routinely excavated to serve as a sediment trap for the high sediment loads carried from upstream. During this time, significant erosion was occurring on the downstream face of the dam such that the invert dropped an additional 7 feet and the entire length of the dam became exposed. After 1992, when the District halted its maintenance program, the channel in the vicinity of Comer Debris Dam reached a state of quasi-equilibrium. Although stable, the reach of creek around Comer Drive Bridge has decreased the depth of the opening under the bridge from about 10 feet to about 4 feet (Kennedy/Jenks 2002), which has raised community concerns about bridge overtopping during large storm events. In addition, the Dam is not aesthetically pleasing, provides poor habitat for vegetation, and makes the creek less accessible to the public for recreation as there is a deep, steep drop at the dam face.

In response to these concerns and others, the District has identified three main conceptual alternatives for replacing the Dam:

Alternative 1: The Dam is replaced completely with a section of channel of constant slope. District analysis concludes that the 'most stable' channel configuration would have a slope of about 1.3% and would extend from just upstream of Wardell Rd. to about 600 feet upstream of Comer Drive Bridge, a total of about 2700 feet.

Alternative 2: The Dam is replaced by a series of rock weirs or drops. The feasible design of this alternative includes 10 drops ranging in depth from 0.8 to 1.2 feet and in length from 60 to 170 feet. The slope between drops would be designed to be 1%, matching that in the reach immediately upstream of the project reach. About 750 feet of channel would be impacted in construction of Alternative 2.

Alternative 3: The Dam is partially removed. In this scenario, the top 5.6 feet of the existing dam would be removed. A stable slope of 1.25% would be established upstream of the dam. The impacted area would include about 500 feet of channel.

These conceptual alternatives were roughed out in a 2002 study by Kennedy/Jenks Consultants. This document summarizes the development of each conceptual alternative into a feasible one. This process involves several steps, including 1) the determination of stable channel cross-sectional shape for each alternative, 2) calibration of the model for the existing conditions, 3) development of an initial HEC-RAS model for each alternative to serve as the hydraulic inputs for the sediment transport analysis, and 4) iterative adjustment of the HEC-RAS geometry until sediment transport patterns produce a stable invert profile for the project reach.

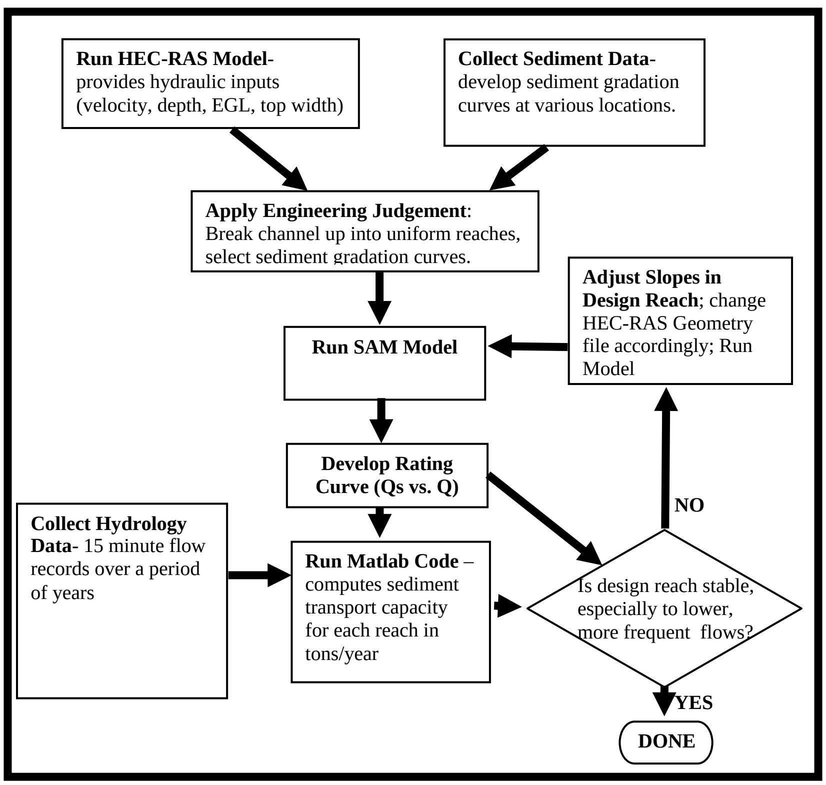
This document is organized into 6 sections, describing 1) the procedure for the sediment transport analysis, including necessary inputs, 2) selection of the inputs for this analysis, based on data collected and a calibration of the model for the existing conditions, and 4), 5), and 6) development of the feasible alternatives 1, 2, and 3, respectively.

1. Sediment Transport Analysis Procedure

The procedure for designing a stable invert slope for a reach of creek is summarized in Flow Chart 1.1 below.

Inputs for this analysis include sediment gradation curves representative of the reaches of interest, a HEC-RAS model of the creek, and either historical flow records (in this case) or a flow frequency analysis. The HEC-RAS model should extend a distance upstream and downstream of the project reach such that the sediment transport capacity of the surrounding reaches can be adequately computed (i.e., boundary conditions should be far upstream and downstream of the project reach).

Essentially, the procedure involves performing iterations between adjusting the slope(s) of the project reach (in the HEC-RAS model) and performing the sediment transport analysis until a balance of the sediment transport capacity is achieved between the project reach and the reaches immediately upstream and downstream of it. These reaches are defined by the engineer to contain relatively uniform geometry and hydraulics, so that the standard deviation of a given hydraulic parameter (e.g., velocity) is small within a reach.



Flow Chart 1.1 Describes the procedure for performing the sediment transport analysis.

2. Development of Inputs for Design of Feasible Alternatives

As mentioned in Section 1, several inputs are necessary for development of feasible alternatives at Comer Debris Basin. In this section, we present the results of the existing conditions calibration case. The calibration case is necessary for ensuring that our inputs to the sediment transport model are reasonable. Here, inputs include the division of the channel into hydraulically-uniform sub-reaches, the sediment gradation curves used for each reach and the flow data used for computing the annual sediment yield estimates. The calibration case also determines the sediment transport equation that will be used in the modeling process through comparison of results for different equations with known data (sediment removal data, in this case).

In addition, in order to generate stable geometry files for the three alternatives, it is necessary to identify a stable, geomorphic cross section shape that can be used for the new channel geometry in the HEC-RAS files. The important cross section parameters, determined from a site visit, are also presented in this section.

2.1 Existing Conditions: Calibration Case

2.1.1 Flow Data

Six years of recent historical flow data recorded at the Wilcox gauge station between 1999 and 2004 were used in this analysis. Calabazas Creek is dry most of the year, and the flow records are flashy. Figure 2.1.1.1 plots the non-zero flows for each water year. The data are plotted as flow rate in cfs vs. time in days as recorded at Wilcox gauge station, where zero flow rates (occurring most of the time) have been removed from the record so that the data are treated as continuous hydrographs, even though the flow data are not contiguous. Since the project reach (i.e., surrounding Comer Debris Basin) is located far upstream of Wilcox gauge station, the flow rates have been reduced accordingly by a factor of about 0.28. This adjustment factor is the ratio between District hydrology predictions of the 10-year flow event magnitude near Comer Debris basin to the value near Wilcox.

As will be shown later in the calibration case, these 6 years of flow data yield reasonable estimates of the sediment transport capacity in the reach upstream of Comer Debris Basin relative to available sediment removal data there.

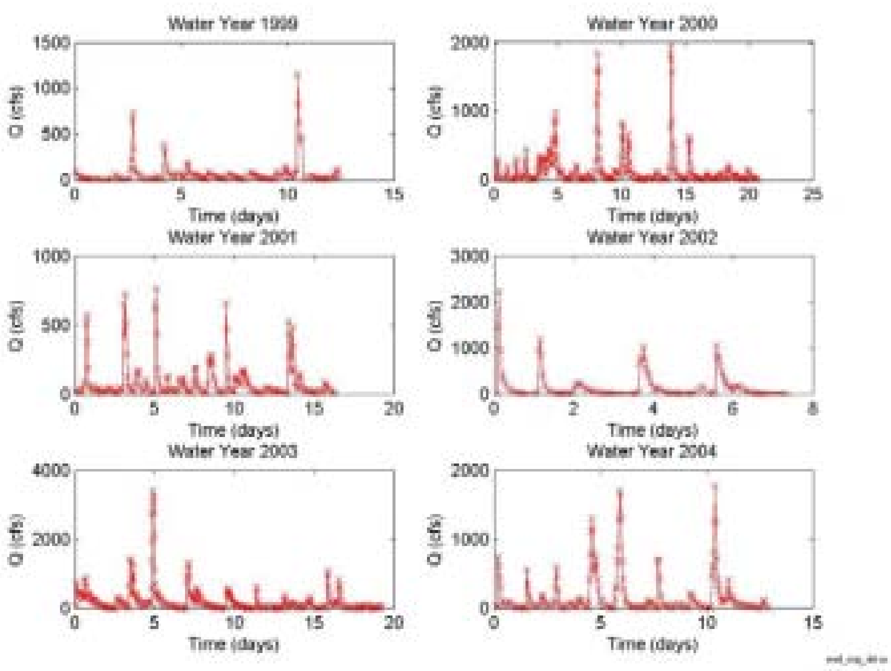


Figure 2.1.1.1 Flow Data at Wilcox Gauge Station for water years 1999 – 2004. Zero flows have been removed from the record, which is treated as a continuous flow record for the purposes of our sediment transport analysis.

2.1.2 Sediment Data

Sediment samples were collected in the vicinity of Comer Debris Dam in November of 2005. The sites were spaced anywhere from 200 to 1200 feet apart. At each sample site, three samples were collected- one from the armoring layer, one from the pond, and one from the subsurface layer beneath the point bar. Six samples between Wardell Rd. and Padero Rd. Bridges were used in the sediment transport analysis.

As explained in the section 1, the sediment transport analysis involves computation of and comparison between the sediment transport capacity for each reach. This procedure includes selection of a representative sediment gradation curve for each reach. After considerable deliberation, the project team decided to use a single sediment gradation curve for the sediment transport analysis in all of the reaches.

The final sediment gradation curve used was computed as the average of the seventeen bags of sediment collected for the six samples used in the analysis (three samples at each of the six sites except for one, which lacked an armoring layer). Figure 2.1.2.1 shows the sediment gradation curves sampled for the six sites, plotted together with their curve average. Figure 2.1.2.2 shows the average for each of the six samples on the same figure along with their average. The sediment gradation curve used in this study- the average of all 17 samples- is the solid blue line with circle markers in Figure 2.1.2.2.

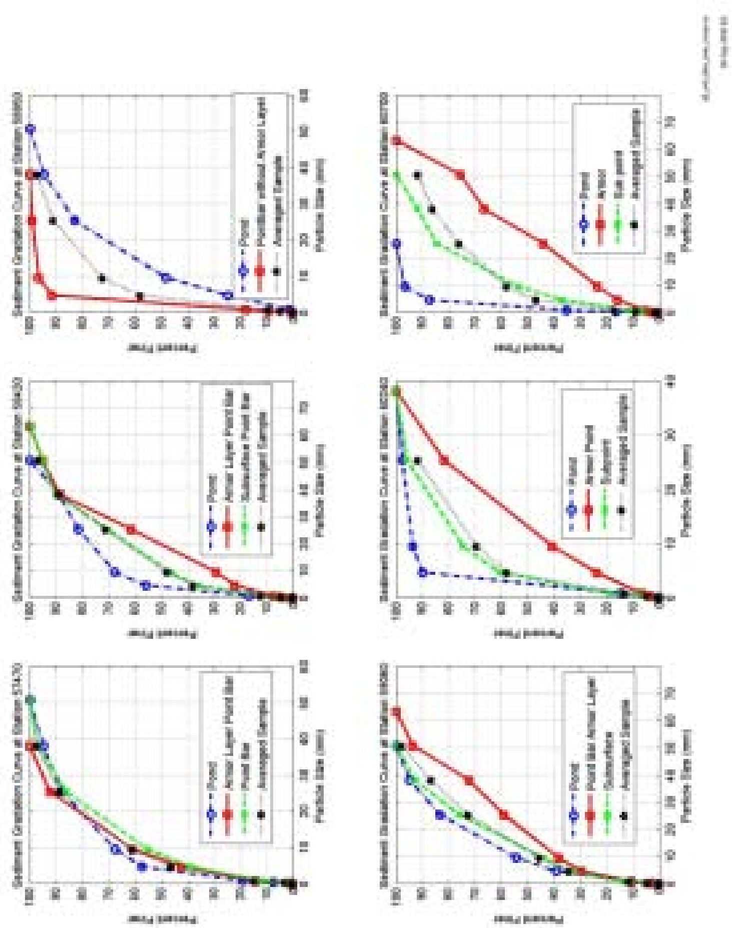


Figure 2.1.2.1 Sediment samples collected at various stations along Calabazas Creek, in the vicinity of Comer Debris Dam.

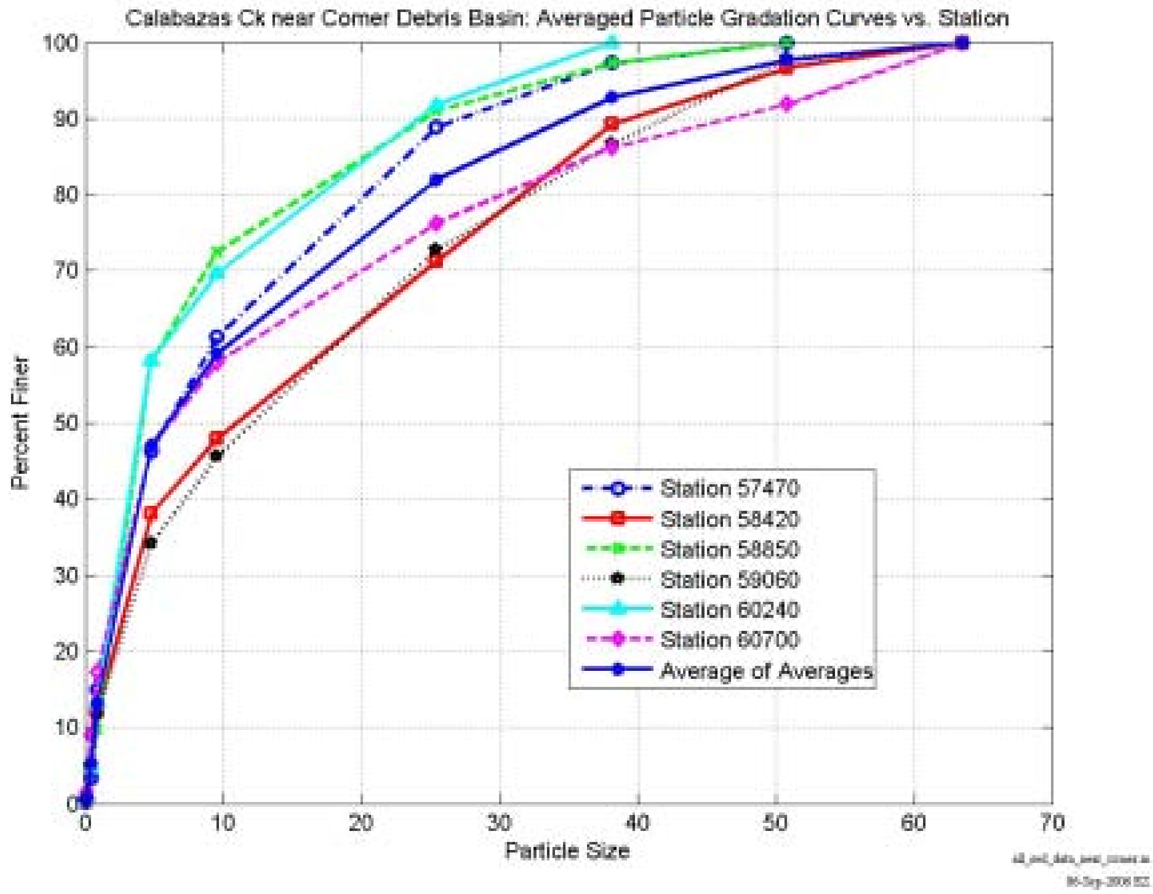


Figure 2.1.2.2 Averaged sediment profiles at 6 stations near Comer Debris Dam. The "average of averages" (i.e., the average of the 17 samples collected near the project site) profile was used in calculations of sediment transport capacity for each reach of channel.

The decision criteria for selecting the sediment gradation curve formed by averaging the 17 samples between Wardell Rd and Padero Rd. for the sediment transport analysis were:

1) There was no discernible pattern in the way that sediment size varied with distance downstream.

2) For each sample site, three samples were collected from regions with different particle characteristics (i.e., one each from the armoring layer, the pond, and the subsurface below the point bar). There is a reasonable chance that these samples were not statistically representative of the material at that site.

3) The sediment gradation curves are used to estimate the total sediment moved over a number of years. For this long time period, use of a mixture of sediment samples from the armoring layer, the pond, and the subsurface region below the point bar seems reasonable.

4) The selected sediment gradation curve, along with the choice of the Toffaleti-MPM method, produced sediment transport capacity values in the Reach upstream of Comer Drive which matched well with sediment removal data. See the Section 2.1.4 for more information.

2.1.3 Existing Conditions: Division of Channel into Hydraulically Uniform Reaches

We begin with a description of the length of channel located between Wardell Rd. Bridge at its downstream end and Padero Rd. Bridge at its upstream end, which is the extent of the HEC-RAS model used in this analysis. This reach both extends between two control structures and is sufficiently large to characterize the areas downstream and upstream of each alternative. The length of this reach is about 4500 feet long, or about 0.85 miles. Comer Debris Dam is located about halfway through the reach. There are two bridges within this reach- Comer Drive Bridge, located about 200 feet upstream of Comer Debris Basin, and a Foot bridge, located about 1100 feet upstream of Comer Drive Bridge.

The slope, channel roughness, and cross section shape vary considerably in the reach extending from Wardell Rd. Bridge at its downstream end to Padero Rd. Bridge at its upstream end. For the purposes of simplifying the sediment transport analysis, this reach has been broken up into five sub-reaches with relatively constant slopes, channel shape, and roughness characteristics- three downstream and two upstream of the dam. The reaches are outlined in Figure 2.1.3.1 and their major characteristics are summarized in Table 2.1.3.1 below. These sub-reaches, referred to herein as Reaches 0-4, were used in the existing conditions calibration case. The hydraulic inputs from each were averaged and input into SAM for computing the sediment rating curves.

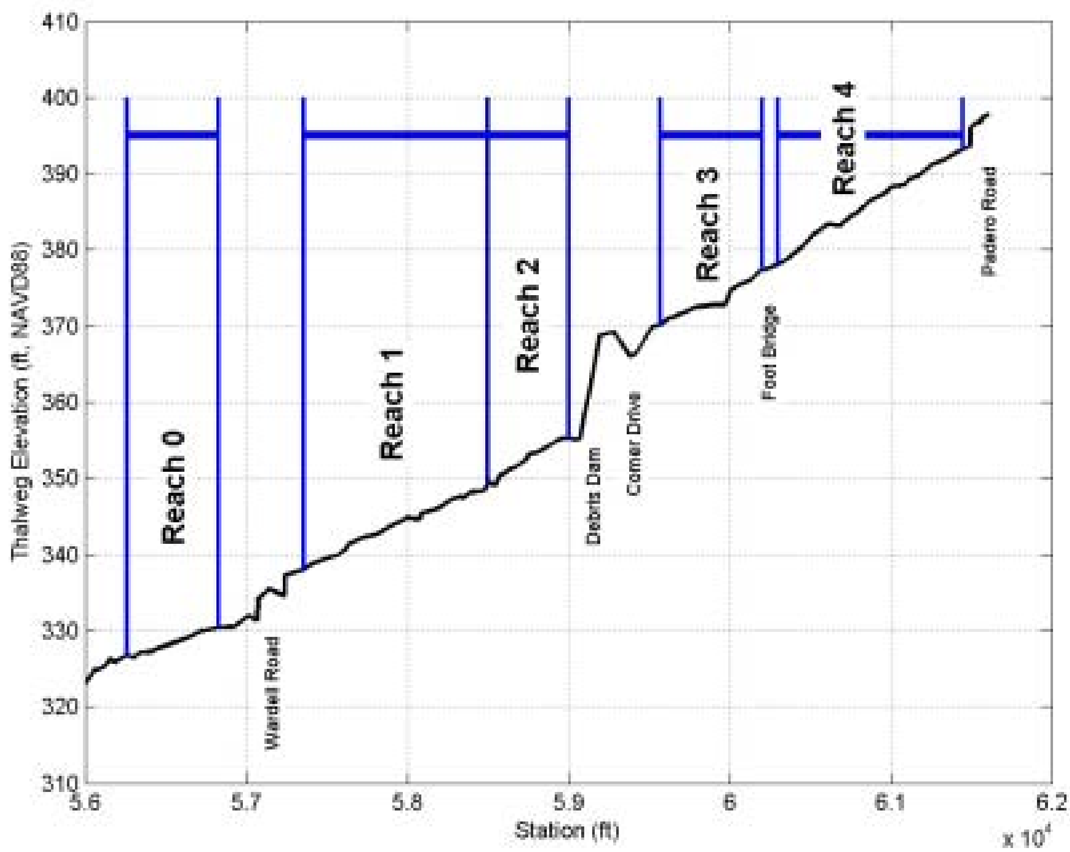


Figure 2.1.3.1 Definition of reaches 0- 4 relative to the thalweg profile for the existing conditions. Thalweg elevation data were obtained from the 2006 HEC-RAS model.

Table 2.1.3.1 Characterization of Channel Geometry in Reaches 1-4

	Length of Reach (Ft)	Average Percent Slope of Reach (ft/ft)*100 %	Description of Channel Cross Section Shape	Roughness of Low Flow Channel
Reach 0	560	0.7 %	Cross sections are uniform, narrower than Reach 1, and deep. 100-year flow is contained	n ~ 0.036
Reach 1	1140	0.9 %	Similar to Reach 2, but less incised	n ~ 0.03 – 0.047
Reach 2	500	1.3 %	Somewhat incised, well vegetated	n ~ 0.04 – 0.047
Reach 3	630	1.1 %	Wide flood plain (relative to Reach 4 upstream)	n ~ 0.034
Reach 4	1150	1.3 %	Deeper and narrower than Reach 3	n ~ 0.034

2.1.4 Calibration for Existing Conditions- Selection of a Sediment Transport Function

The SAM package includes 20 different sediment transport equations. Although several were developed for computing sediment transport in gravel bed streams, we tested only the MPM (bedload only) and MPM-Toffaleti (combined bed- and suspended load) methods, because the MPM method produced reasonable results in previous calculations of the sediment transport rate in Calabazas Creek between Miller Avenue and Homestead.

The sediment transport capacity values averaged over the six years of flow data for each reach are given in Table 2.1.4.1 below. Not surprisingly, the two related methods produce different magnitudes, since the MPM method computes only the bedload whereas the MPM-Toffaleti computes the combined bed- and suspended load. However, they produce similar patterns from reach to reach, since they were given the same hydraulic inputs. Essentially, the transport capacity increases with distance upstream.

Table 2.1.4.1 Computed Values of Average Annual Sediment Transport Capacity (tons/year) using Different Sediment Transport Equations. Values are rounded to the nearest 5.

Reach Identified	Sediment Transport Capacity MPM (1948), tons/year	Sediment Transport Capacity MPM-Toffaleti, tons/year
Reach 0	1580	5055
Reach 1	2010	5850
Reach 2	2540	6610
Reach 3	2140	6115
Reach 4	3415	9225

Apart from conceptual understanding of sediment transport patterns arising from observation of erosion and deposition over the years, the main data available for calibration of the sediment transport capacity model are found in maintenance records of sediment removal data for Comer Debris Basin. Sediment was removed from the basin in 1974, 1975, 1978, 1982, 2983, 1985, 1986, 1990, 1991 and 1992 before maintenance was halted. After each removal, the excavated area was filled back up with sediment from upstream. Thus, the amount of sediment removed annually (expressed as tons/year) at the Debris Basin represents a lower limit on the sediment *transported* annually by the reach upstream of Comer Debris Basin, which, in turn, is a lower limit to the annual sediment transport *capacity*.

Annual sediment removal rates at Comer Debris Basin range in value from about 2200 tons/year in a dry year to a maximum value of 13,700 tons/year in a wet year. The average value over the 10 years of data is about 5600 tons/year (Kennedy/Jenks 2002). The results presented in Table 2.1.4.1 are analyzed by comparing annual removal rates at Comer Debris Basin with the average annual capacity for Reach 3, located just upstream of Comer Debris Basin. The transport capacity of Reach 3 represents the upstream supply of sediment for the debris basin. Clearly, the average sediment transport capacity computed for Reach 3 with the MPM method of 2140 tons/year is too low, producing a number which is about 40% of the historical average sediment removal rate by the District of 5600 tons/year. The MPM-Toffaleti equation yields an average annual transport capacity of 6115 tons/year, which is only 10% larger than the annual removal rate.

Kennedy/Jenks 2002 estimated that the sediment supply in Reach 3 should be about 8960 tons/year on average. This estimate is based on a combination of past field measurements of sediment load which showed that sand and gravel constituted about 40% of the total load and the assumption that only sand and gravel were deposited in the debris basin. Both assumptions are reasonable, and the value we show here seems to be about 30% low.

However, this is not the case. The smallest point on the sediment gradation curve entered into SAM was that 0.3% of the sample was finer than 0.07 mm. This resulted in an aggregate sample which, according to SAM, contained only particles which were larger than 0.0625 mm, the cut off for silt. [This has to do with the way SAM parses the entered gradation curve]. For now, we simply note that our estimates of sediment transport capacity include only transported sand and gravel and should, then, be on the order of the sediment removal data estimate of 5600 tons/year, rather than the 8960 tons/year estimated by Kennedy/Jenks. Thus, our calculation of 6115 tons/year of sand and gravel transport capacity, only about 10% larger than the removal rate of 5600 tons/year, is very reasonable.

More work could have been done to determine what percentage of silt and clay comes from upstream of the project reach. This is deemed unnecessary, since a reasonable estimate of the sand and gravel transport rate has been made, and silt and clay particles tend to remain suspended in the wash load. Furthermore, the remainder of the analysis attempts to balance the transport rates between the project reach and its neighboring upstream and downstream reaches- so that differences in this value between reaches are more important than magnitudes.

2.2 Identification of Geomorphic Cross Section: for development of Alternatives at Comer Debris Basin

The shape of the channel cross section in the reach extending very roughly 400 feet downstream of Comer Debris basin differs significantly from the reach of channel extending upstream. Immediately downstream of the debris dam, the channel is well vegetated, somewhat incised, and has a relatively steep slope of about 1.3%. Immediately upstream of the dam, the channel is characterized by less vegetation, a wider floodplain and a milder slope of about 1%. Because of the channel incision downstream (and the locale of the alternatives), the design channel cross section shape was based on channel conditions upstream of Comer Debris Basin, where the channel is natural and stable.

The geomorphic design for the channel cross section shape to be used in the feasible alternatives was based on measurements made in the summer of 2006 of the existing channel dimensions upstream of Comer Dam. Measurements of the bankfull channel- (depth, bottom and top widths) and flood plain- (depth and width) dimensions were made at several locations where a geomorphic, stable channel was identified to have formed. Measurement locations spanned from a distance upstream of Comer Drive Bridge to locations upstream of the Footbridge. In addition, HEC-RAS was run for several flow rates to determine whether one of the flow rates consistently filled the bankfull channel identified in the field. It turned out that the 1.5 year flow event (about 200 cfs) satisfied this criterion reasonably well, which falls within normal range for the frequency of the effective flow rate. After these calculations were performed, District staff calculated the effective discharge for Calabazas Creek to have a recurrence interval of 1.1 years (based on data from the flow gauge far downstream at Wilcox.

The channel dimensions for the bankfull channel are summarized below in Table 2.2.1. The prototypical channel shape is also plotted together with a surveyed cross section located about 500 feet upstream of Comer Bridge to show that it is similar to the existing channel in shape.

Table 2.2.1 Recommended Stable Cross Section Dimensions for Calabazas Creek near Comer Debris Basin	
Bankfull Channel Width (Ft)	12
Bankfull Channel Cross-channel Slope, Horizontal:Vertical (Ft/ Ft)	2.5:1
Bankfull Channel Depth (Ft)	2.5
Flood Plain Slope, Vertical/Horizontal (Ft/Ft)	0.002
Bank Slope, Horizontal:Vertical (Ft/Ft)	2:1

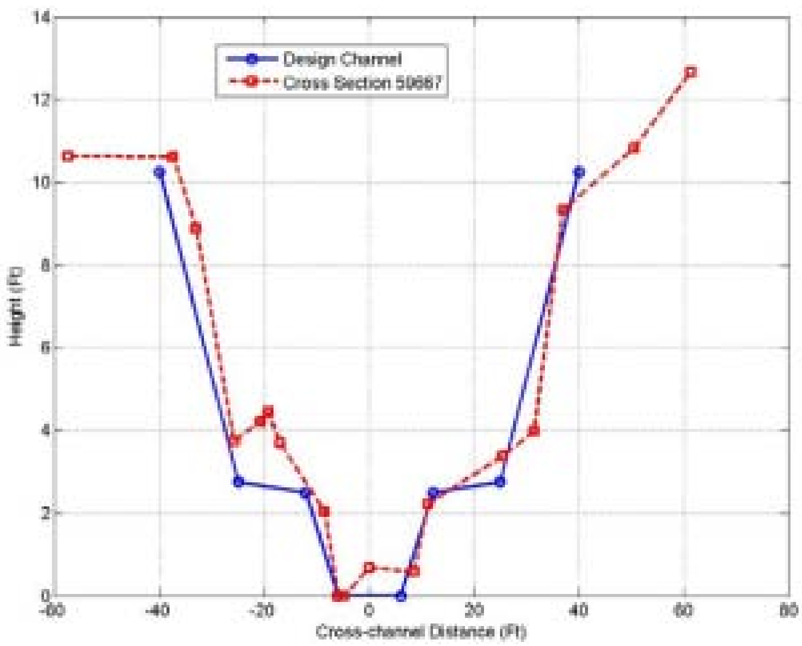


Figure 2.2.1 A channel cross section located about 500 feet upstream of Comer Dam superimposed on the design channel.

3. Feasible Design of Alternative I

The design for Alternative 1 involves complete removal of the Comer Debris Dam and its wingwalls. The channel profile would be modified to have a consistent, stable slope formed by a combination of excavation (primarily upstream of the dam) and fill (primarily downstream of the dam). Removal of the dam would significantly improve the habitat around Comer Debris Basin for establishment of vegetation and aquatic life, but these improvements would be initially offset by the large impact area necessary for its construction. The main purposes of Alternative I then are the improved flood protection at Comer Drive, which would be achieved by excavation under the bridge to a depth of at least 7 feet to provide the 100 year flood protection, and the improved habitat for aquatic life.

Design Parameters for Alternative I

The main design parameters for Alternative I are the depth of clearance beneath Comer Drive Bridge and the consistent channel slope. In the following analysis, we identify the most feasible channel slope which also provides the needed clearance below Comer Drive Bridge.

Of course, the channel shape and roughness also play a strong role in determining sediment transport capacity and are parameters which can be varied to achieve a stable design channel. For this analysis, it was deemed reasonable to use the prototypical stable and geomorphic channel identified for this area of Calabazas Creek as described in Section 1.1. This shape is used in the excavation of channel cross sections, with deviations from the design shape according to its ability to tie into the existing banks at some locations.

SAM Analysis

In this section, we describe the process for determining the most stable slope for alternative I.

The first step in this process involves making an educated guess of channel configuration which may be stable, which provides at least 7 feet of clearance beneath Comer Drive Bridge, and which attempts to minimize the impact area. The slope guess is necessary for the development of initial HEC-RAS geometry input, which is modified for each iteration of the SAM analysis (usually by changing the invert slope of the project reach).

The project reach for alternative 1 must extend both upstream and downstream of Comer Drive Bridge to allow for complete removal of the Dam. Its slope should be on the order of the slopes of the surrounding reaches. A slope of 1.5% with about 9 feet of clearance beneath Comer Drive Bridge was estimated to be a good first guess at a stable slope. This slope provides the necessary clearance under the bridge and has a relatively minimal impact area extending about 1110 feet downstream and 590 feet upstream of Comer Drive Bridge, so that it is situated between existing reaches 1 (downstream) and 4 (upstream). In addition, the project reach has quite a long stretch of channel for which the existing floodplain and channel shape is both wider and shallower than Reach 4 upstream. A steeper slope than that for Reach 4 of 1.3% is potentially called for in order to balance the sediment transport capacity upstream.

In order to perform the SAM analysis for the 1.5% slope channel case, hydraulically-uniform reaches for computing the average sediment transport capacity had to be identified. These were based on the reaches identified for the existing conditions case. As it turns out, the project reach for the 1.5% case includes all of reach 2 and most of reach 3. Thus, the choice of reaches was obvious- one for characterizing the project reach, Reach 1 shortened on its upstream end by about 300 feet for characterizing the downstream reach, and Reach 4 extended by about 300 feet on its downstream end for characterizing the upstream reach. These new Reach 1 and 4 definitions did not differ significantly in channel geometry or hydraulics from their definitions for the existing conditions case.

The results of the 1.5% case indicated that the slope choice of 1.5% was too large. Figures 3.1 and 3.2 show the sediment transport capacity and rating curve results from the SAM analysis. Although the transport capacity of the project reach is only about 6% larger than that of the reach upstream, it is also

40% larger than that of the downstream reach. This situation could lead to significant deposition downstream in Reach 1.

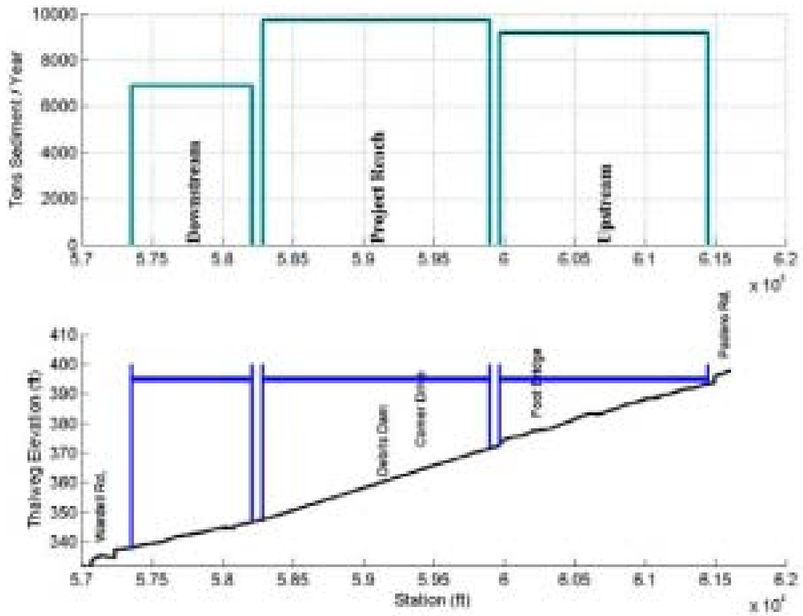


Figure 3.1 Sediment Transport Capacity Calculations for a version of Alternative I with a 1.5% invert slope.

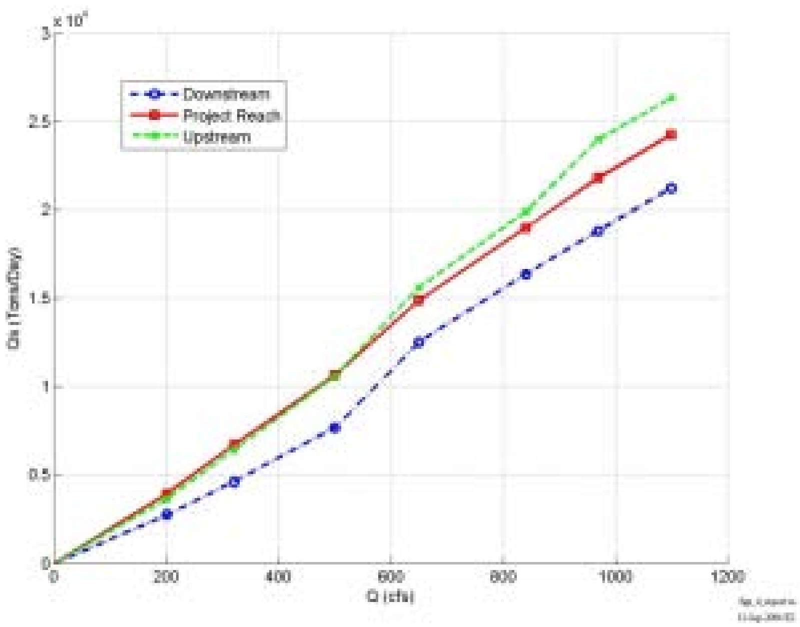


Figure 3.2 Sediment Rating Curves for a version of Alternative I with a 1.5% invert slope.

A more stable channel configuration for Alternative 1 should have a smaller slope, allowing for the transport capacity of the project reach to lie between the O(9000) tons/year of the reach upstream and O(5000) tons/year of the reach downstream. It would also have at least 7 feet of clearance and a minimal impact area. The impact area can be easily assessed by plotting lines of different slopes which go through the points below the bridge at fixed distances greater than or equal to 7 feet from the Comer Drive soffit. The upstream and downstream locations where these lines intersect with the existing thalweg profile (if they do at all) define the limits of impact. Figure 3.3 shows these lines for slopes with 1.2%, 1.3% and 1.4% for the 8 ft clearance case. Seven, nine, and ten feet clearance cases were also considered. The 1.3% slope case was deemed to be the most feasible (with a lower slope and reasonable impact distance) for 7 and 8 feet of clearance. The 8 ft clearance was selected because it provides extra freeboard.

For the 1.3%/ 8-ft of clearance case, the existing uniform reaches downstream and upstream of the proposed project reach are Reaches 0 and 4, respectively. The sediment transport capacities for these neighboring reaches are 5055 tons/year for Reach 0 and 9225 tons/year for Reach 4. Thus, the aim for the project reach should be to provide a transport capacity somewhere within this range.

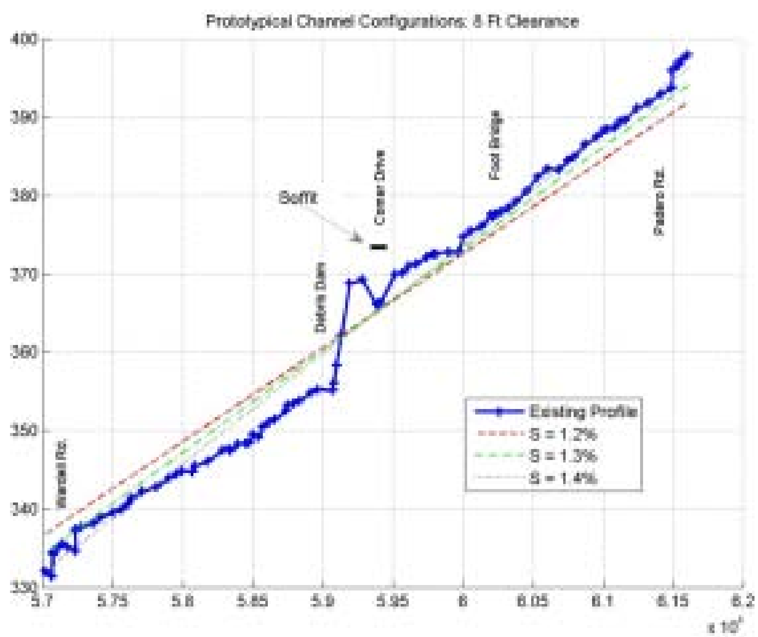


Figure 3.3 Constant-slope lines that go through the point 8 feet below the bridge opening plotted to show locations of their intersection with the existing thalweg profile.

In order to determine a preliminary estimate of the sediment transport capacity for this case, a HEC-RAS model was run with uniform channel cross sectional shape based on the design channel shape outlined above and a constant 1.3% slope, with normal depth boundary conditions specified on the downstream and upstream ends. The SAM analysis was then performed, and results estimated a sediment transport capacity for that reach of about 8630 tons/year. This capacity estimate value falls between those for Reaches 0 and 4, as required for the development of a stable channel. This estimate is very rough, however, because it does not take into account changes in geometry (e.g., in the width or floodplain) which occur over the length of the project reach.

Alternative 1 may be the least desirable of the three alternatives because it impacts the largest stretch of creek. In addition, preliminary calculations indicate that the sediment transport capacity of the project reach would be increased significantly relative to existing capacities computed for Reaches 1, 2 and 3 (as identified for the existing case). This is due to the fact that the large drop at the existing debris basin allows for milder slopes extending upstream and downstream of the dam than could exist if the drop were replaced with a consistently sloped channel invert. This increased transport would likely result in an eventual slope adjustment over years, most likely causing deposition downstream of the project reach.

If this alternative were to be adopted as the preferred one, it is highly recommended that a HEC-6 type analysis be performed to compute the long term bed evolution.

4. Feasible Design of Alternative 2

Alternative 2 involves removing the dam and wingwalls in their entirety, and replacing them with a series of small drops. This alternative impacts a smaller distance of about 750 feet of the creek than alternative 1, but also requires construction of the drops and the purchase of large rock for their construction.

The design of Alternative 2 includes 1) a channel realignment to straighten the channel slightly in the vicinity of Comer Dam, and 2) construction of 10 drops with the distance between drops ranging from 60 – 170 feet. The drops are located away from bends and the bridge entrance at Comer Drive; i.e., in reaches where the flow is relatively uniform. In addition, care will be taken to allow for preservation of the Oak tree near Comer Debris Dam. In construction, we assume that the drops themselves will be constructed of rock, and a rock-lined pool about 15 feet long will be constructed downstream of each drop. The portion of the channel bed between each drop which is natural will also be covered with an armoring layer similar to that which exists upstream of the dam, yielding similar roughness to the channel as the upstream. The channel shape is based on the design channel defined in section 1 above, where the design channel cross section will be positioned to tie into the existing banks and the bankfull channel will be centered around the new channel alignment.

In this case, the project reach containing the 10 drops was flanked by existing reaches 2 on the downstream end (almost all of it) and 3 on the upstream end (all of it), which have approximately equal sediment transport capacities. The project reach was divided into 9 different sub-reaches, each including 2 or more cross sections. These sub-reaches were located between adjacent drop structures and outside of the influence of drawdown on the downstream end where the flow goes critical. There is one fewer sub-reach than the number of drop structures because the areas downstream of the most downstream drop and upstream of the most upstream drop were absorbed into reaches 2 and 3. Reaches 1 and 4 of the existing channel were also included in this analysis. The values for Reach 0 are unchanged from the existing conditions case, and have been included in Table 4.1 below for reference (but not its accompanying Figure 4.1).

The initial geometry had a slope of 1.25% between drops, with drop heights constant at about 0.75 feet. This produced sediment transport capacities in the project reach which were about 10- 20% larger than the transport capacity immediately upstream and downstream of the reach (omitting the large transport capacity in the reach named Drop 7 - 8, see the next paragraph).. This could lead to significant erosion in the project reach. Therefore, the slope between drops was adjusted to the smaller value of 1% by changing the drop heights to range between 0.8 and 1.2 feet tall. [This was achieved efficiently in the HEC-RAS geometry file by translating entire sections up or down with a maximum elevation change of 1.2 feet]. The results of these two analyses are shown in Table 4.1 below. [Note that the transport capacities for Reaches 2 and 3 are slightly different values from those provided in the Table 2.1.4.1, but that values are the same for Reaches 0 and 4. In Reach 2, the difference is due to the slightly shortened reach definition. In Reach 3, the difference is due to the changed downstream flow conditions of the project reach (i.e., Alternative II).]

The goal is for the sediment transport capacities of existing reaches 2 and 3 (highlighted) to match those along each of the drops. For both cases, drop reach 7 - 8 has a much larger capacity than all other drops- this is due to the fact that the drop reach 7 - 8 goes beneath Comer Drive Bridge, which has a lower roughness value. The average transport capacities across drops (but excluding drop reach 7 - 8) are about 6496 tons/year and 7517 tons/year for the 1% slope and 1.25% slope cases, respectively. The 1% slope case average transport capacity is about 4% lower than the downstream capacity and 0.7% higher than the downstream capacity, whereas that for the 1.25% case is 11 to 15 percent higher than capacities of reaches 2 and 3.. Therefore, the 1% slope in the project reach should yield a more stable channel. The results for the 1% slope case are shown in Figures 4.1 and 4.2, showing the sediment transport capacity for each reach as a bar plot and the sediment rating curves developed for each reach as line plots.

The choice of the 1% slope also makes sense physically. As described in Table 1.2.1, Reach 3 has a smaller slope (1% vs. 1.3%), a less rough channel bottom (n ~ 0.034 vs n ~ 0.04), and a wider floodplain than Reach 2. Combined, these factors act to balance the sediment transport between Reaches 2 and 3—i.e., the smaller roughness (acting to increase channel velocity) combats the wider floodplain and smaller slope (acting to decrease channel velocity). With the sediment transport capacities of the reaches immediately upstream and downstream of the project reach balancing, the goal is to find a slope which yields the same transport capacity as that of those two reaches. In this case, since the design channel shape and roughness are dictated to be similar to that of the upstream reach, the obvious choice for the prototypical slope is about 1%. Figures 4.1 and 4.2 support this argument.

Table 4.1 Comparison of sediment transport capacities for two versions of Alternative 2 (1% and 1.25% slopes). Values are rounded to the nearest 5.

Reach Idenfication	Sediment Transport Capacity in tons/year	
	Alternative 2 with 1% Slope Between Drops	Alternative 2 with 1.25% Slope Between Drops
Reach 0	5055	5055
Reach 1, Existing Channel	5875	5875
Reach 2, Existing Channel	6775	6775
Drop 1-2, Project Reach	6410	7605
Drop 2-3, Project Reach	6755	8000
Drop 3-4, Project Reach	6495	7360
Drop 4-5, Project Reach	6470	7590
Drop 5-6, Project Reach	6155	7250
Drop 6-7, Project Reach	6870	7730
Drop 7-8, Project Reach	8240	9800
Drop 8-9, Project Reach	6125	7050
Drop 9-10, Project Reach	6685	7550
Reach 3, Existing Channel	6540	6540
Reach 4, Existing Channel	9225	9225

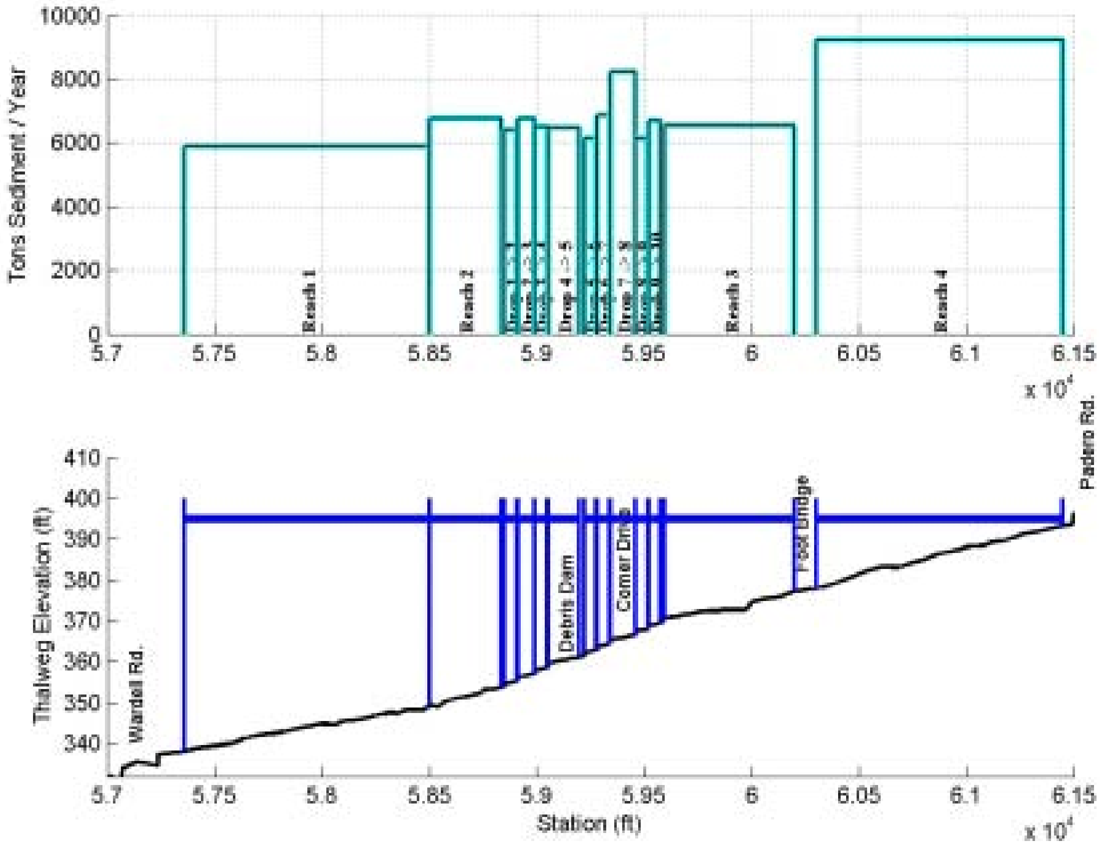


Figure 4.1 Average annual sediment transport capacity computed from 6 years of historical flow data, in tons/year (above), for Altrernative 2. Definition of reaches over which hydraulic parameters were averaged (below). The sediment gradation curve used in these calculations is shown above, in Figure 1.3.2.

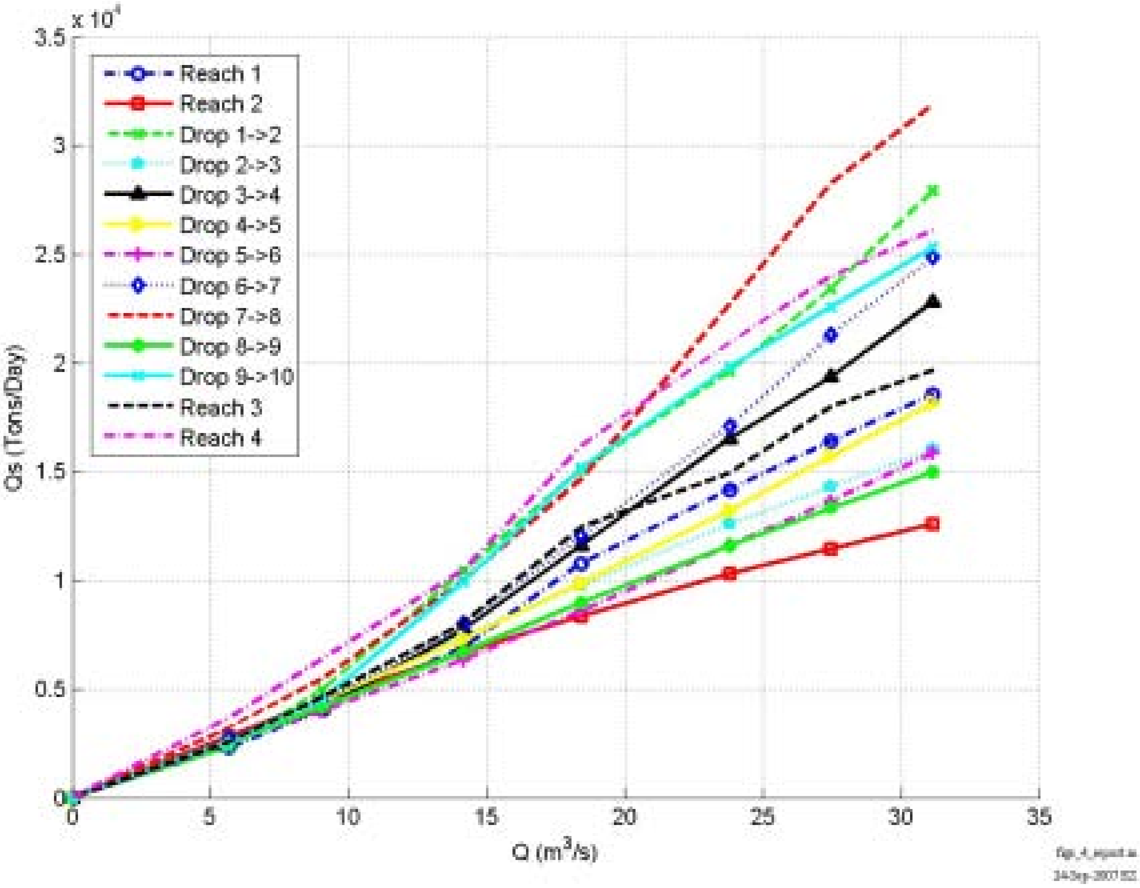


Figure 4.2 Rating curves computed with SAM, using hydraulic input from HEC-RAS and the sediment gradation curve shown in Figure 1.3.2, for the reaches in Alternative 2 with a 1% slope between drops.

5. Feasible Design of Alternative 3

Alternative 3 would partially remove Comer Debris Dam, lowering the existing drop to increase clearance beneath the bridge, but preserving its lower portion. The main benefit from construction of Alternative 3 would be the improved flood protection at Comer Drive Bridge. Secondary benefits would include a generally improved environment for the establishment of vegetation and for aquatic life. However, these secondary improvements would be to a lesser extent than those for Alternatives 1 or 2 because of the preserved steep drop.

Three geometry configurations were considered for Alternative 3. All versions of Alternative 3 extend from Comer Debris Dam about to a distance of about 175 feet upstream of Comer Drive Bridge, for a total impact distance of about 500 feet. The configurations tested for stability included three cases, called Alternative 3a, Alternative 3b and Alternative 3b1. Alternative 3a has a 1.5% slope case with removal of the top 8 feet of the dam (including a drop of 0.85 feet at the downstream end of Comer Drive Bridge). Alternative 3b has a 1 % slope with removal of the top 5.6 feet of the dam (with no drop). Alternative 3b1 has a 1.25% slope with removal of the top 6.4 feet of the dam (with no drop). All versions of Alternative 3 conform to the existing channel a distance of about 175 feet upstream of Comer Dr. Bridge. The channel slope between the downstream face of the bridge and the conforming point is about 1.7% for all three cases.

The HEC-RAS model results showed that the flow was close to critical depth upstream of the bridge. In order to determine whether this was due to the steep 1.7% slope there, some tests were performed by using a milder (1.25%) slope upstream of the bridge and conforming to the existing channel with a drop. The results of these tests indicated that the cause of the near-critical flow was due to changes in channel width, and not the slope.

Construction of alternatives 3a, 3b and 3b1 includes excavating sediment over the distance 175 feet upstream of the bridge to the downstream face of the bridge to form a uniform, 1.7% slope. This procedure would provide an additional three feet of clearance (for a total of seven feet of clearance) beneath the bridge. From the downstream face of the bridge, sediment would be further excavated with consistent slopes of 1% and 1.25%. The 1.5% case tested is different and would include an additional 0.85 ft drop at the downstream face of the bridge, downstream of which sediment would be excavated to form a uniform 1.5% slope.

Table 5.1 Average Annual Sediment Transport Capacity for Different Versions of Alternative 3. Values are rounded to the nearest 5.

	Average Annual Sediment Transport Capacity (tons/year): Alternative 3a (1.5% Slope)	Average Annual Sediment Transport Capacity (tons/year): Alternative 3b (1 % Slope)	Average Annual Sediment Transport Capacity (tons/year): Alternative 3b1 (1.25% slope)
Reach 1	5850	5845	5845
Reach 2	6550	6515	6515
Project Reach for Alternative 3	14310	5220	6625
Reach 3	7320	7310	7310
Reach 4	9185	9565	9565

The average annual sediment transport capacity results for the alternatives 3a, 3b and 3b1 are shown in table 5.1 above. Of the three versions of Alternative 3 tested, alternative 3b1 provides an acceptable sediment transport capacity for the project reach. The project reach capacity for alternative 3b1 is only about 2% larger than that downstream, and 9% smaller than that upstream. By contrast, the project reach capacity for the 1% slope is about 20% lower than that downstream and 29% lower than that upstream. The transport capacity for Alternative 3a is significantly higher than upstream or downstream, because the steep slope induces supercritical flow in the project reach for most flow rates. The reach layout and sediment transport capacity variation for Alternative 3b1 are shown graphically in figure 5.1 below. Because Alternative 3b1 is the most stable version of alternative 3, it will now be referred to simply as Alternative 3.

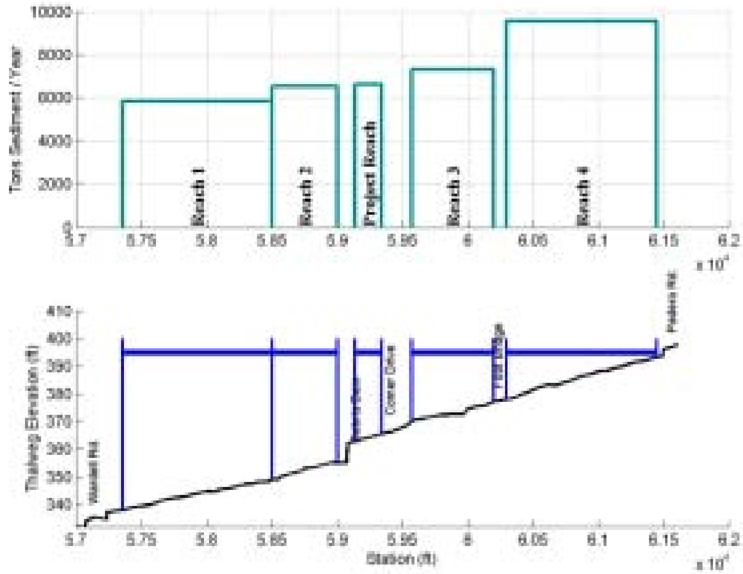


Figure 5.1. Average annual sediment transport capacity (computed with 6 years of flow data from 1999-2004) for Alternative 3 (version b1).

For this simple SAM analysis, it is important to investigate how the sediment transport rating curves (i.e., sediment transport rate vs. flow rate) vary between reaches in addition to a sediment transport capacity analysis. This second step is necessary because it always possible for average annual sediment transport capacities between two reaches to balance in cases where the sediment rating curves are quite different (e.g., the curves could cross).

The sediment transport rating curves for Alternative 3 are shown in Figure 5.2 below. The rating curves are very similar between reaches. The results presented in this figure close the loop, proving that the cause of the balanced sediment transport capacity values, is, indeed, the similarity in rating curves between neighboring reaches. Because the low flows are more frequent, it is particularly important that the rating curves match at lower flows (which they do).

The sediment transport of Reach 4 looks to be significantly higher than that of all reaches downstream. In the field, there is evidence of erosion in Reach 4, but it is localized and does not occur to the extent that would be expected from the differences in sediment transport capacity. From observations (from site visits) of relatively minimal erosion to the invert profile, it is likely that the actual amount of sediment transported in Reach 4 is significantly smaller than that of it downstream neighbor Reach 3.

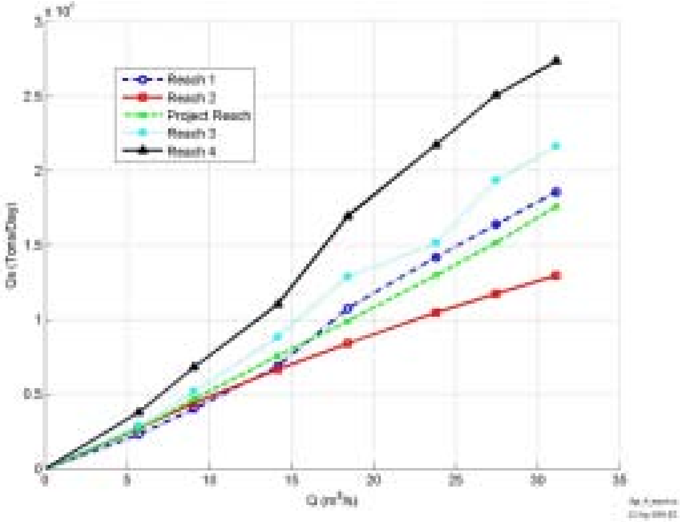


Figure 5.2 Sediment transport rating curves developed for each reach for Alternative 3.

6. Summary

The results of the sediment transport analysis are summarized in Table 6.1 below.

Table 6.1 Summary of the sediment transport analysis for each alternative

	Alternative I	**Alternative II**	**Alternative III**
Extent of Impact Area	2700 Ft	750 Ft	500 Ft
Brief Description of the alternative geometry	Dam is completely replaced with a channel of consistent slope of about 1.3% or smaller.	Dam is replaced with a series of 10 drops of 0.8 – 1.2 feet high, with lengths between drops raning from 60 – 170 feet.	Top 5.6 feet of the dam is removed. Channel is excavated to have a consistent slope of 1.25% extending from the Dam to Comer Drive; of 1.7% extending from the Comer Drive to a

			distance 175 feet upstream to conform to the channel.
Level of Detail in Study	With its large impact area, a number of simplifying assumptions were used in the design of the prototypical stable channel geometry.	Final prototypical geometry design for Alternative II was detailed, robust, and stable.	Final prototypical geometry design for Alternative III was detailed, robust, and stable.
SAM Analysis Results	Alternative I can be stable, but details of stable channel geometry need to be flushed out for this alternative.	Alternative II is stable according to SAM analysis.	Alternative III is stable according to SAM analysis.

7. Recommendations

From an invert stability perspective, Alternatives II and III have been shown to be equally stable according to this steady-state SAM analysis. More work is required to design the stable geometry for Alternative I because of its large impact area, but it should be possible to do so. Once an alternative has been selected, a HEC-6 style model should be performed to ensure that the design geometry is stable as well.

From an impact area perspective, Alternative I has the largest impact area, followed by Alternatives II and III. Alternatives II has an impact area which is about 185 feet longer than Alternative III.

From an environmental and aesthetic perspective, Alternative II is probably preferred. Compared with Alternative III, it allows for better fish passage, entirely removes the dam, and has a reasonable impact area. Compared with Alternative I, its impact area is much smaller. Alternative I may eventually provide the same environmental benefit as Alternative II after the vegetation in the creek has re-established itself, and would provide easier fish passage than Alternative II, due to its lack of drop structures.

From a cost perspective, Alternative II is probably more costly than Alternative III because it involves more earth, the additional construction of step-pools (incurring both labor and rock materials costs), and the complete (as opposed to partial) removal of Comer Debris basin. The costs of Alternatives I and II cannot be easily compared until estimates have been completed. Both involve complete removal of the Dam. Alternative I requires more earth work than Alternative II, but Alternative II requires construction of drop structures (including labor and rock materials costs) whereas Alternative I lacks drop structures.

The results of this study are focused mainly on the channel invert stability and should be combined with the results of the geomorphology studies before choosing the preferred alternative.

8. References

Kennedy/Jenks Consultants, 2002. "Comer Debris Basin Engineering Feasibility Study Final Reaport," prepared for Santa Clara Valley Water District.

Philip Williams and Associates, 2005. "Santa Clara Valley Water District (SCVWD) Stewardship Initiative: Calabazas Creek Tier One Fine-Scale Geomorphic Assessment," prepared for Tetra Tech.