Draft
Conceptual Channel Restoration Design and Hydraulic Modeling Report

for

Sunset Stables Restoration and Resource Management Plan Project

Prepared for

California Department of General Services
Real Estate Services Division
and
California Tahoe Conservancy

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Section 1 - Introduction

This report presents the results of two Sunset Stables Sub-Tasks under *Task 6 – Conceptual Restoration Design Plans*. First, as outlined in *Sub-Task 6A – Conceptual Channel Restoration Designs*, the procedures used to develop the new channel planform and cross-section geometry for the Sunset Stables Alternatives restoration designs are described. Then, as part of *Sub-Task 6B – Hydraulic Modeling for Alternative Channel Restoration Designs*, the methods and results of hydraulic modeling of the new channel designs in the three restoration Alternatives are presented.

Several previous reports describe the existing geomorphic conditions of the Upper Truckee (UT) River at Sunset Stables (CTC, CTC, 2004b; CTC, 2007a; CTC, 2007b; Simon et al., 2003; TRCD, 2003; USACE, 2000). This report incorporates the data and analyses presented in these previous reports with new analyses described below to outline the methods and assumptions used in the new channel design and hydraulic modeling.

Section 2 - Existing Channel Conditions

A brief description of geomorphology in the Sunset Stables reach is described here[1].

The UT River in Sunset Stables is predominantly a single-thread channel meandering through grassy meadow and lodgepole pine. Analysis of air photographs dating back to 1940 indicates little planform change over the past 60 plus years in Sunset Stables, with most change likely due to natural channel migration (Figure 1). The channel planform in this reach was not radically altered to accommodate airport construction as it was downstream in the City of South Lake Tahoe's (CSLT) channelized airport reach (Figure 2). The 1940 channel has a sinuosity of 1.55. The existing channel maintains a sinuous course throughout the project area similar to the 1940 channel. The average bed slope of the existing Sunset Stables channel is approximately 0.001, and channel sinuosity is 1.56. Typical of other channels also in watersheds producing weathered granite (Parker, 2004), the channel substrate is transitional between sand and gravel with the median bed surface grain size typically ranging from fine to medium gravel (4 mm to 16 mm) depending upon the geomorphic surface sampled.

Bedload measurements and modeling show that the channel has a mixed sand and gravel bedload, with coarse sand composing the greatest percentage of material transported (see Section 4 for an in-depth discussion of sediment transport).

The estimated 450 cfs channel-forming flow[2] has respective Log-Pearson Type III recurrence intervals of approximately 1.35 and 1.4 years based on peak annual flows at USGS gages #103366092 in Meyers and #10336610 in South Lake Tahoe (see discussion of hydrology in Section 3 below for more detail). Analysis of mean daily discharge records at gage #10336610 indicates that a 450 cfs discharge is equaled or exceeded

[1] See the March 23, 2007 report prepared by ENTRIX, Inc. for the CTC titled: *Results of Sunset Stables Sub-Task 4C1: Supplemental Geomorphic Assessment* for detailed studies on channel form, hydraulic modeling, and sediment transport in the Sunset Stables reach.

[2] See CTC, 2003 for a detailed account of the analyses undertaken to estimate the UT River's channel-forming flow. Many of the channel-forming flow analyses in the CTC (2003) report are updated with new data in this report.

approximately 15 days a year, on average. Based on HEC-RAS modeling of a 450 cfs flow in Sunset Stables, the mean channel top width is 65 ft, the mean hydraulic depth is 2.8 ft, mean boundary shear stress is 0.18 lb/ft^2, and the mean channel velocity is 2.5 ft/s (CTC, 2007a).

Channel banks in Sunset Stables can be stratified with fine-grained lacustrine sediment overlain by alluvial deposits. Bank sediment sampling by Simon et al. (2003) and ENTRIX (CTC, 2007a) documented the median bank material is primarily fine sand. In ENTRIX's bank sampling results, sand typically composes about 53% of the bank material, silt 38%, and clay 8%. The cohesive, fine-grained sediment composing the banks increases erosion resistance, which is exhibited in the limited natural channel migration detected since the 1940 air photograph.

Comparison of the existing top of bank profile in the project reach indicates that the top of bank is currently between 0.75 to 3 ft higher than the modeled 450 cfs existing water surface profile (CTC, 2007a). Where the channel is incised 0.75 ft, a flow of about 700 cfs is required for the channel to overbank onto the floodplain, which has a 2-year recurrence interval based on Log-Pearson Type III analysis of peak annual flows at gage s#10336610 and gage #103366093. Analysis of mean daily discharge records at gage #10336610 indicates that a 700 cfs discharge is equaled or exceeded approximately 5.7 days a year, on average. Where the channel is incised 2.5 to 3 ft, a 1,300 cfs flow is required for floodplain inundation, which has a 4.5-year to 5-year recurrence interval based on Log-Pearson Type III analysis of peak annual flows at gage #10336610 and gage #103366093, respectively. On a daily basis, a 1,300 cfs discharge is equaled or exceeded on average approximately 0.3 days a year at gage #10336610. Since the start of the gaging record in October 1971, the mean daily discharge has exceeded 1,300 cfs a total of 9 days. The low frequency of overbanking illustrates that the UT River's floodplain is rarely inundated in many areas, and increasingly hydrologically disconnected from the floodplain throughout the entire project reach.

If the base level for the UT River in Sunset Stables remains unaltered, the future geomorphic evolution of the channel is likely to include local downcutting in some areas, while in many areas channel bar growth in combination with cut-bank erosion will lead to continued incipient floodplain development at a lower elevation than the historic broad floodplain above. Further expansion of the incipient floodplain will lead to decreased hydrologic connectivity with the historic floodplain since it will require higher and higher flows to exceed the conveyance capacity of the incipient floodplain and inundate the historic floodplain. In addition to reducing the potential floodplain inundation area, growth of the incipient floodplain will continue to promote erosion of the steep channel banks and input of fine-grained sediment into the channel.

The focus of this channel restoration is on designing a channel form that supports natural geomorphic processes and will be sustainable under the existing hydrologic and sediment regimes. Since the hydrology and sediment loads in the UT River are critical to the channel and floodplain restoration design, they are described below.

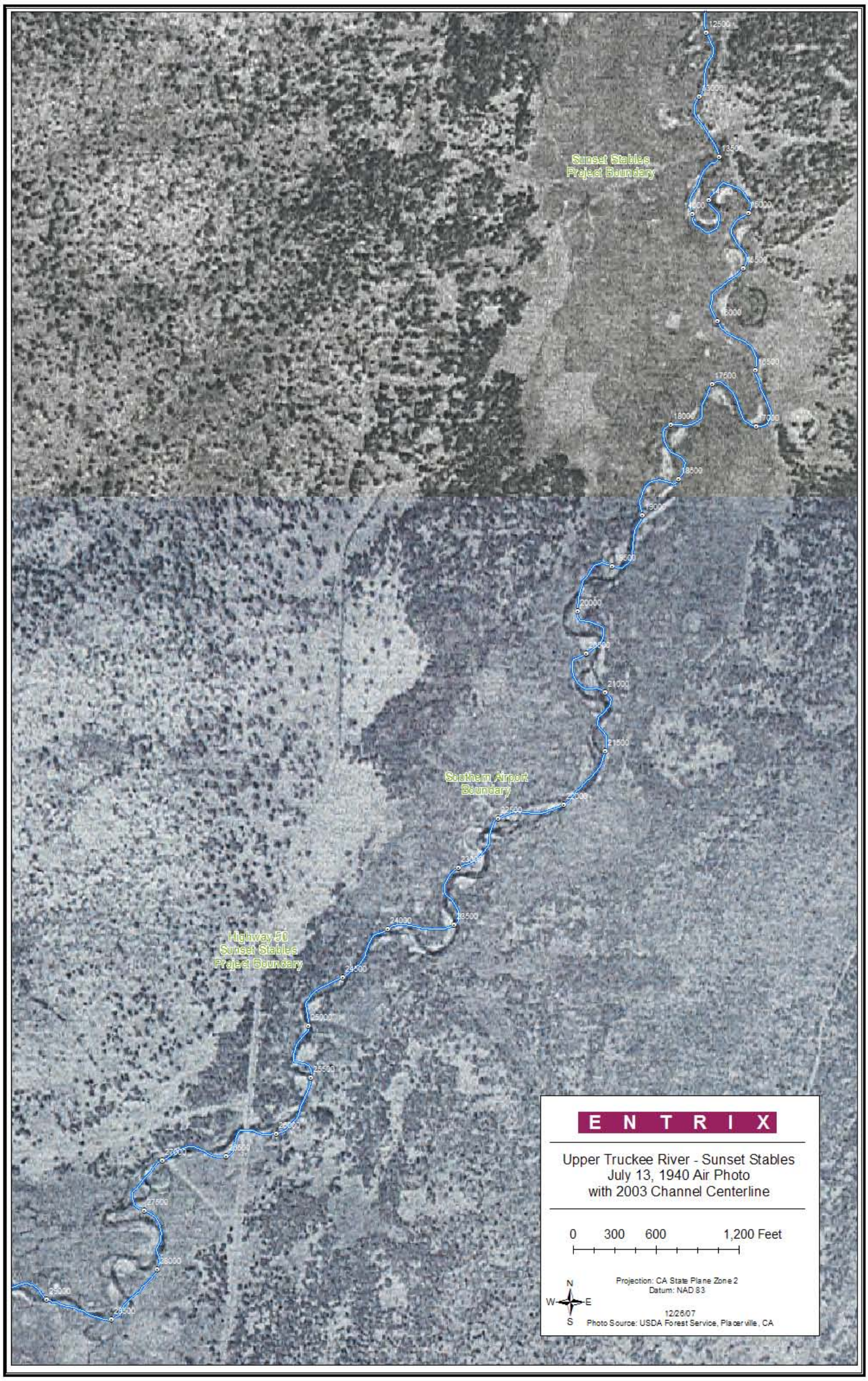


Figure 1. July 13, 1940 air photo of the UT River in the Sunset Stables project area with the 2003 UT River channel centerline displayed in blue. Source: USDA, U.S. Forest Service office, Placerville, CA

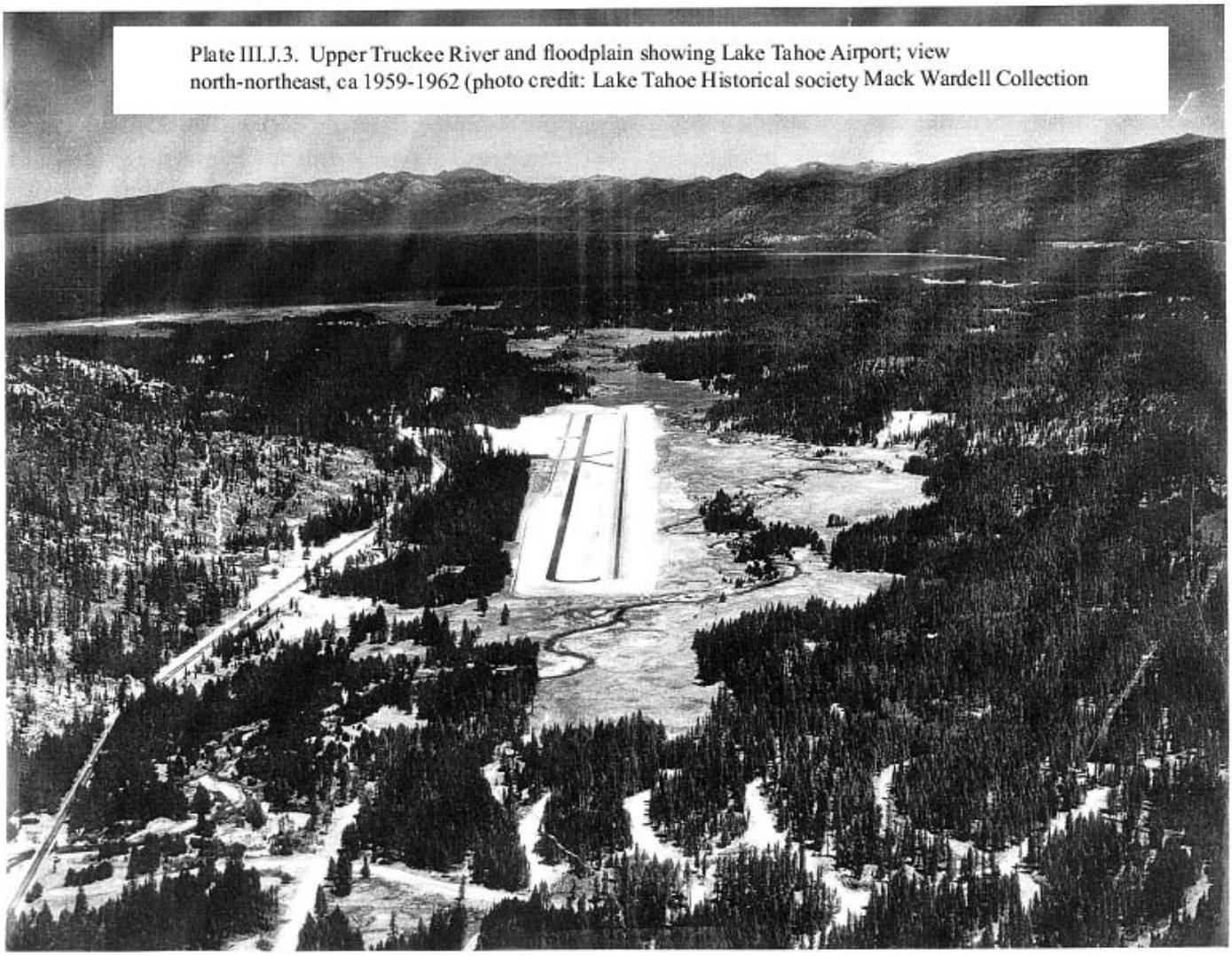


Figure 2. Oblique view of the UT River in the Sunset Stables project area prior to completion of the airport runway expansion in the late 1960s. Source: TRCD, 2003

Section 3 - Existing Surface Hydrology

A review of the UT River's surface hydrology, updated with the most recent hydrology data, is presented below since knowledge of the hydrologic regime is instrumental in the channel design procedures described in Section 5. Streamflow (discharge) is gaged by the U.S. Geological Survey (USGS) at four locations on the UT River (Table 1). The Upper Truckee at South Lake Tahoe gage (#10336610) located at the pedestrian bridge near South Lake Tahoe Blvd. is the closest gage, located approximately 2.5 miles downstream of the Sunset Stables project area boundary. Steamflow data from this gage is continuous back to 1972 except for gaps between 10/01/1974 to 09/30/1976, 07/01/1977 to 09/30/1977, and 07/01/1978 to 02/28/1980.

Table 1. USGS streamflow gaging stations within the UT River watershed

Station Name	*USGS Station ID*	*Period of Record (Water Years)*	*Contributing Drainage Area (Sq. Mi.)*	*Percent of Watershed Gaged*
Upper Truckee at South Lake Tahoe	10336610	1972–1974 1977-1978 1980-current	54.0	97.2
Upper Truckee River at Highway 50 above Meyers	103366092	1990-current	39.2	68.8
Upper Truckee near Meyers	10336600	1961-1986	33.2	58.6
Upper Truckee So. end of So. Upper Truckee Rd.	10336580	1991-2001	14.1	25.0

Source: (NHC, 2002; Rowe and Allander, 2000)

Mean Daily Streamflow

The USGS gaging station continuously takes water stage measurements of the UT River at 15-minute intervals. All of the 15-minute measurements taken in one 24-hour period are converted to a streamflow (cfs) and averaged by the USGS to determine the mean daily streamflow for that day. This calculation is done for every day of the year. Mean daily streamflows for the South Lake Tahoe gage are graphed in time-series since 1972 in Figure 3. The river displays large annual and seasonal flow variation typical of an unregulated Sierra Nevada snowmelt river. Seasonal snowmelt creates annual maximum mean daily flows generally in May or June. Seasonal low flows occur in the summer and fall, usually between July and November. The typical snowmelt season streamflow on the UT River ranges from 200 to 500 cfs, with fall low flows under 25 cfs. Climate-driven cycles can produce extreme highs and lows during a single year and from one year to the next. The extreme high flows are mostly, but not entirely, associated with winter rain-on-snow conditions. Mean daily streamflows associated with major rain-on-snow events (e.g., Feb 1963, Dec 1964, Jan 1997) are often 3 times greater than snowmelt flows on the UT River. The mean daily streamflow of January 2, 1997 was measured at

3,150 cfs. In addition to longer duration snowmelt runoff peaks in spring, short duration peak runoff events with lower volumes occur in summer months from thunderstorms that typically last only a few hours (USACE, 1999).

The UT River flow duration curve (Figure 4) is a statistical analysis (based on the Weibull plotting position method) of all the mean daily streamflows on record since the 1972 water year. The curve describes the percent probability on any given day of the year that a given streamflow will be equaled or exceeded. For example, on any given day, there was a 50 percent probability the streamflow was equal to or greater than 38 cfs. If expressed in terms of number of days, the flow was equal to or greater than 38 cfs for half the days of the year. Table 2 displays the number of days per year that UT River flows were equal to or exceeded for selected streamflow values.

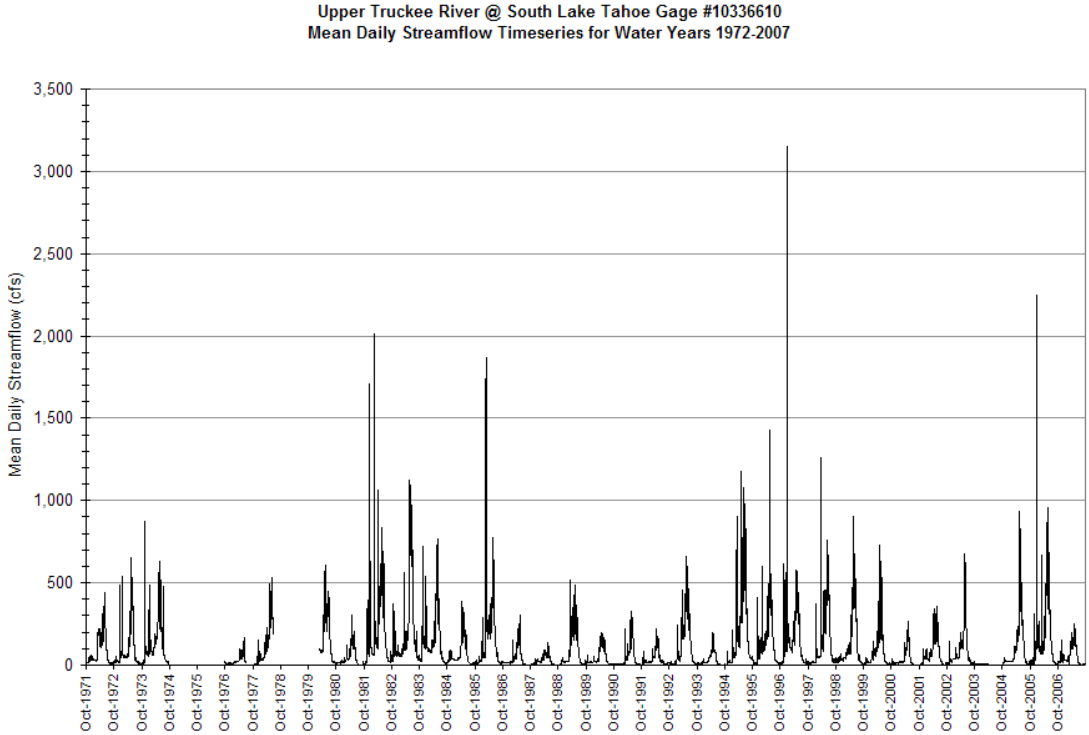


Figure 3. UT River mean daily streamflow timeseries from 1972 to 2007

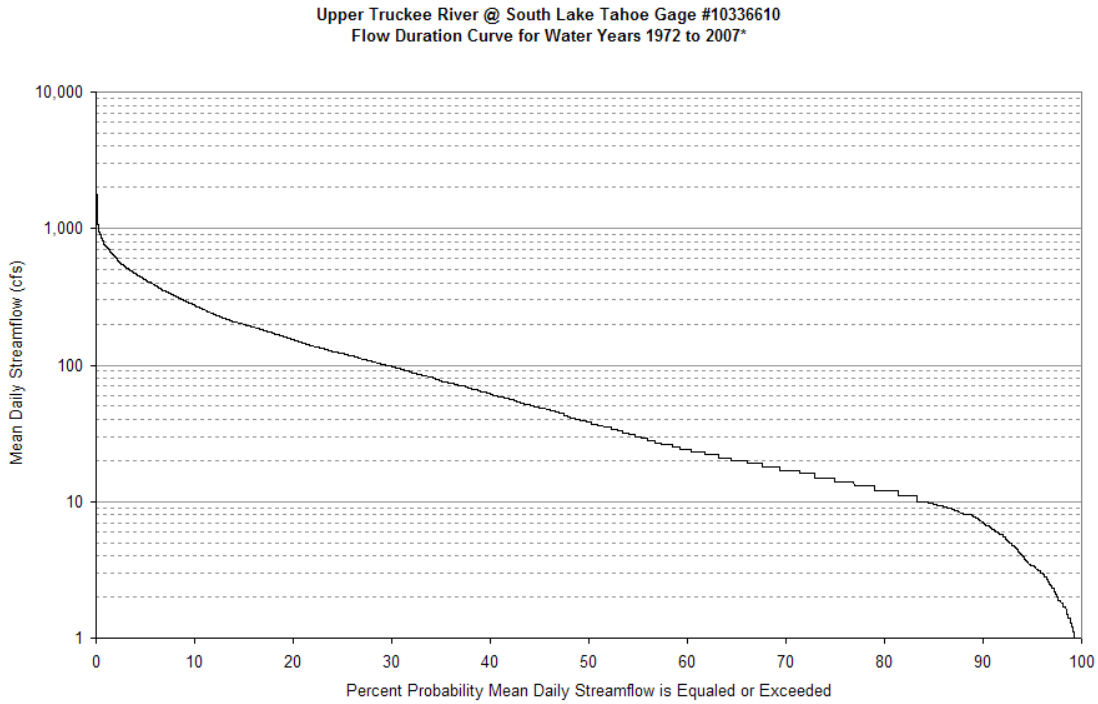


Figure 4. UT River mean daily flow duration curve from 1972 to 2007

Table 2. Mean daily streamflow statistics generated from UT River flow duration curve (water years 1972 – 2007)

Mean Daily Streamflow (cfs)	*% Probability Streamflow was Equaled or Exceeded*	*# Days/Year Streamflow was Equaled or Exceeded*
1,000	.2	0.8
800	.7	2.7
600	2.1	7.7
500	3.4	12.4
400	5.6	20.4
300	8.9	32.8
100	29.6	108
50	44.5	162.4
25	59.2	216.1

While the flow duration curve is useful for statistical analysis to determine how many total days per year a given streamflow can be expected to be equaled or exceeded, it does not describe changes to streamflow exceedence probabilities within a year due to seasonal flow fluctuations. Therefore, monthly exceedences have been calculated for the same mean daily streamflow period of record to describe the probability that a particular streamflow was equaled or exceeded on a monthly basis (Table 3). Statistically, September's median flow is 10 cfs, while in May the median flow is 257 cfs.

Table 3. Monthly streamflow statistics for mean daily discharges at USGS gage #10336610

Upper Truckee River @ South Lake Tahoe Gage #10336610
Monthly Streamflow Statistics for Water Years 1972-2007*

		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
	Maximum	3,150	2,010	1,870	1,060	1,430	1,090	829	165	204	369	875	2,250
	Minimum	8	8	13	25	36	11	1.7	0.07	0.01	0.94	4	6
	Average	66	67	105	165	311	252	82	19	12	15	37	52
	Median	32	45	84	145	257	203	35	11	10	11	16	22
Exceedence Probability	**Return Interval**												
1%	100-yr	448	469	491	444	935	937	696	137	56	60	350	518
2%	50-yr	314	324	320	402	846	858	506	116	46	41	250	340
4%	25-yr	202	208	261	344	734	733	377	77	39	34	156	190
10%	10-yr	122	103	196	280	588	527	203	42	24	26	74	94
20%	5-yr	79	79	140	216	472	424	123	25	14	20	38	59
50%	2-yr	32	45	84	145	257	203	35	11	10	11	16	22
80%	1.25-yr	18	21	45	97	138	57	11	3	3	5	10	14

* No gage records available for periods: 10/1/1974 - 9/30/1976, 7/1/1977 - 9/30/1977, and 7/1/1978 - 2/28/1980

Peak Annual Sreamflows

Peak instantaneous streamflows measured by the USGS Upper Truckee at South Lake Tahoe gage #10336610 from 1972 to 2005 are plotted in Figure 5.

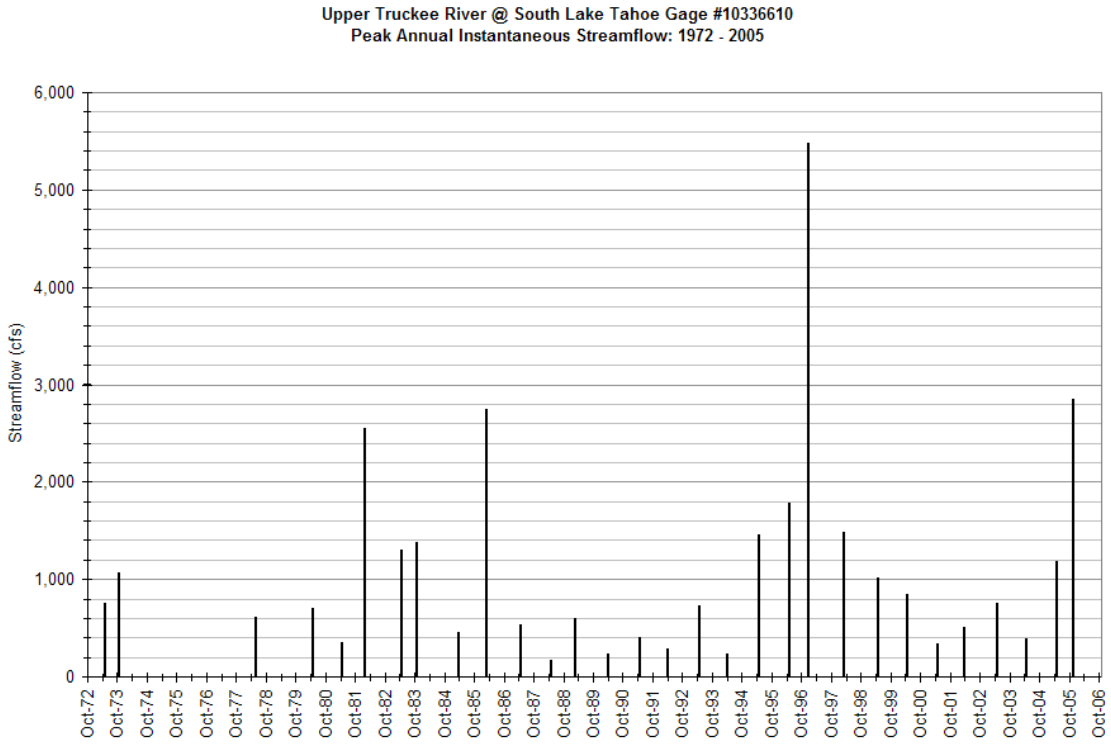


Figure 5. UT River peak instantaneous streamflows measured from 1972 to 2005

The January 2, 1997 rain-on-snow flood event is the largest UT River flood on record. The peak flow was measured at the UT River at Highway 50 above Meyers gage (#103366092) as 5,120 cfs. The gage at Highway 50 in South Lake Tahoe (#10336610) failed during the flood event and was unable to measure the peak instantaneous streamflow.

Because the 1997 flood event was of such large magnitude and peak flows have only been measured at the Highway 50 in South Lake Tahoe gage (#10336610) since 1972, flood recurrence statistical analysis is highly affected by this flow event. Due to the importance of the 1997 event, several estimates have been made of the flood's peak streamflow at Highway 50. The USGS estimated the peak flow at Highway 50 in South Lake Tahoe to be 5,480 cfs. This was based on an extension of the rating curve up to a high water mark within the shelter of their recording equipment at Highway 50 (Rockwell, April 5, 2001 Personal Communication). The USACE (2002) estimated the 1997 peak flow at 8,200 cfs through frequency curve comparison and correlation with Blackwood Creek, and the upper reaches of the UT River and Sagehen Creek flow sites. The large difference between the two peak flow estimates may be a result of flow blockage at the Highway 50 bridge, which may have influenced the lower USGS estimate. Both the USACE and USGS had noted backwater (damming) and possible constricted flow at the Highway 50 bridge during the 1997 event. Flow was observed to cross over Highway 50 near Longs Drugs suggesting a peak flow potential in excess of the bridge design capacity.

ENTRIX calculated flood recurrence statistics using both the USGS and USACE estimates of the 1997 flood event. To increase the number of years on record available

for the peak flow recurrence analysis, ENTRIX correlated the peak annual streamflow record at gage #10336610 with the upstream UT River near Meyers gage (#10336600) to extend the record back to 1961. ENTRIX applied a log-Pearson Type III analysis to the extended record for both the USGS and USACE 1997 flood estimates and developed the flood recurrence intervals listed in Table 4. The analysis resulted in 100-year[3] peak flow estimates of 6,600 cfs and 7,650 cfs for the UT River.

Table 4. ENTRIX flood recurrence interval estimates based on log-Pearson Type III analysis with extended streamflow record of USGS gages #10336610 and #10336600

Return Period (years)	*Upper Truckee Using USGS 1997 Peak of 5,480 cfs (cfs)*	*Upper Truckee Using USACE 1997 Peak of 8,200 cfs (cfs)*
1.5	533	530
2	768	760
5	1,620	1,660
10	2,430	2,550
25	3,790	4,130
50	5,090	5,690
100	6,660	7,650
200	8,560	10,100

Because the previously noted problems of streamflow blockage at the Highway 50 bridge likely affected the USGS 1997 flood estimate of 5,480 cfs, ENTRIX considers the USACE (USACE, 2002) estimate of 8,200 cfs more probable. Therefore, it is recommended that flood recurrence statistics based on the 1997 flood estimate of 8,200 cfs be used.

Moderate magnitude flooding, like a 10-year to 20-year event, on the UT River occurs as peak flows from spring snowmelt events and large winter rainstorms or rain-on-snow events. Summer thunderstorms rarely produce flooding in the CSLT study area because they tend to produce short duration precipitation events with little volume and flashy peaks (USACE, 1999). Major flooding, as in the 1997 event, usually results from large winter rainstorms where antecedent snowpack conditions have produced saturated ground conditions. Flows from spring snowmelt events tend to be less extreme than winter rain events, because the snowpack melts gradually over various elevation zones of the watershed moderating the peak flow.

Section 4 - Sediment Loads

The following discussion of UT River sediment transport is taken from ENTRIX's report written for the CTC (CTC, 2007a) for the Sunset Stables restoration project. It represents the most detailed and recent analysis of the nature of the river's sediment transport through the project area. In combination with the hydrology analysis, the sediment transport relationships derived from this analysis provided a foundation for the design of the new channel dimensions in the Sunset Stables project reach described in Section 5.

[3] Since less than 40 years of peak streamflows are on record, the 100-year flood estimate is based on an extrapolation of data. Another way to express the 1997 event is to call it the flood-of-record.

Suspended Sediment

The USGS collects suspended sediment samples at its UT River gages in South Lake Tahoe (#10336610, dating back to 1971) and in Meyers (#103366092, dating back to 1989). Suspended sediment rating curves created by ENTRIX for data collected at both gages since 1989 are shown in Figure 6. The trend lines for the data show that the UT River in South Lake Tahoe consistently has a higher suspended sediment load than the gage upstream in Meyers, which has a smaller contributing watershed area with fewer tributary inputs. At about 100 cfs the trend lines for the Meyers and South Lake Tahoe gages start converging on each other until the two are nearly the same at a large overbank discharge of about 1,000 cfs.

In addition to reporting the suspended sediment concentration, in the past the USGS has periodically analyzed the suspended sediment of the South Lake Tahoe samples for particle size gradation. The USGS analysis is limited to gradational analysis within the sand size sediment range, and does not distinguish between silt and clay fractions. ENTRIX used the USGS data to develop cumulative particle size gradation curves (Figure 7). These curves represent the typical grain size distributions of the UT River's suspended sediment for discharges ranging from 15 cfs to 600 cfs (analysis was limited to this range because of the availability of data points to regress trends). The largest particle sizes transported as suspended sediment for the 15 cfs and 450 cfs discharges range from about 0.1 mm to 1 mm, respectively. At a 15 cfs discharge about 75% of the suspended sediment is finer than sand (0.063 mm). At a 450 cfs discharge about 37% of the suspended sediment is finer than sand, and at a 600 cfs discharge about 35% of the suspended sediment is finer than sand. The silt/clay to sand ratios of the higher 450 cfs and 600 cfs discharges are similar to the average grain size distributions of the Sunset Stables bank samples, of which the banks contain 57% sand, 36% silt, and 7% clay. This correlation is not unexpected since streambank erosion has been identified as a large contributor of suspended sediment to the UT River (Simon et al., 2003).

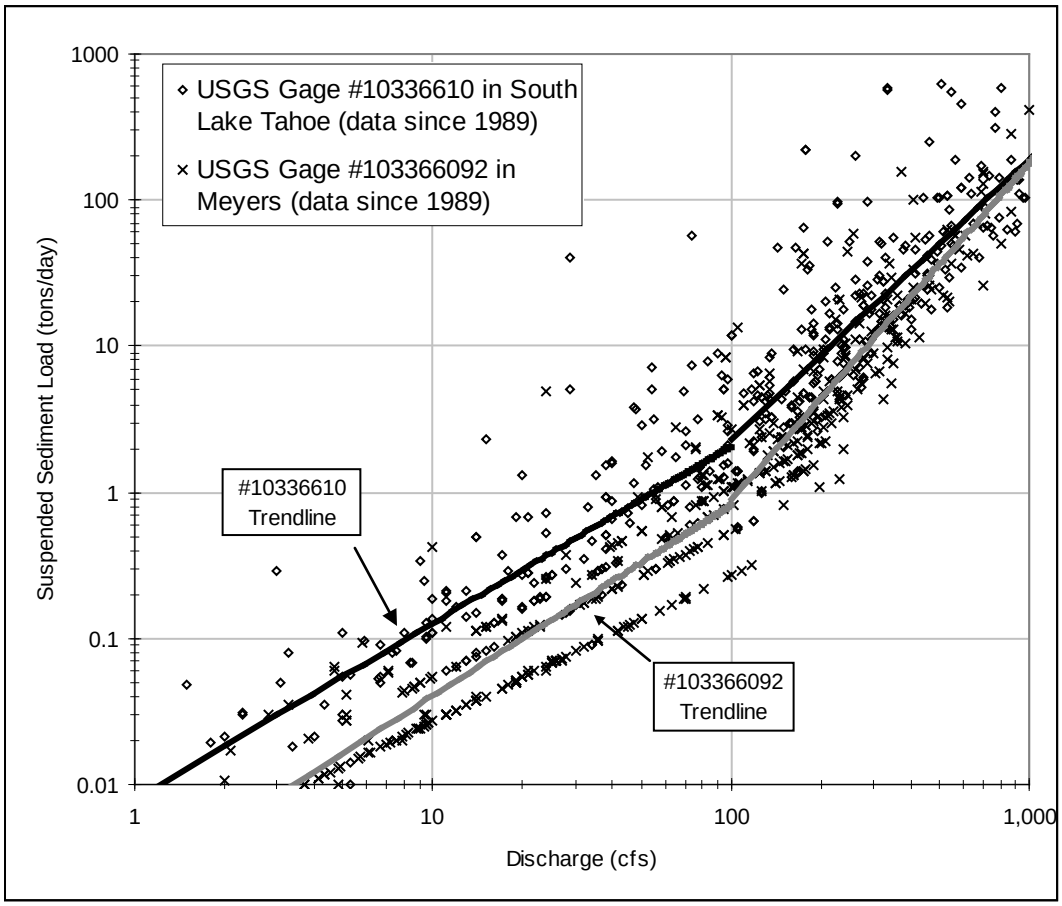


Figure 6. UT River suspended sediment loads since 1989 measured by the USGS at gages in Meyers and South Lake Tahoe

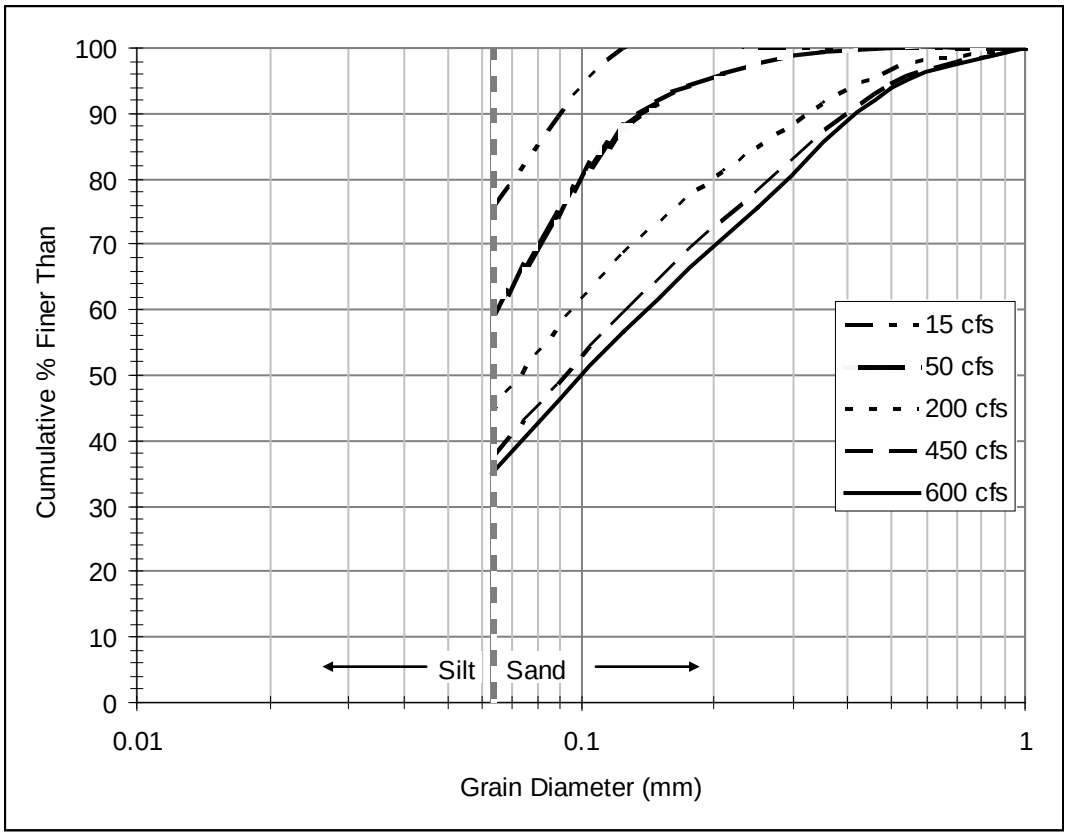


Figure 7. Cumulative particle size gradations of suspended sediment samples collected by the USGS at gage #10336610 in South Lake Tahoe

Bedload Measurement

ENTRIX collected seven bedload measurements on the UT River during the receding limb of the snowmelt hydrograph over discharges ranging from 115 cfs to 260 cfs in late June and early July 2006. Bedload was measured with a hand-held Helley-Smith sampler at a cross-section (RS 25+670) at the water quality monitoring station downstream of the Elks Club in Sunset Stables. This sampling site was selected because it is near the upstream boundary of Sunset Stables and would provide information on the bedload discharge entering the project reach. Furthermore, the sampling cross-section is located on a relatively straight section of channel with near uniform flow conditions and the wide channel width enabled safe wading conditions at high discharges.

Each bedload composite sample consisted of 20 vertical samples spaced evenly across the approximate 50-foot channel. The sampler's net was positioned for two minutes on the bed at each vertical for a total sampling time of 40 minutes per sample. A flow meter was used to measure several discharges at the sampling site over the course of the bedload sampling and a stage-discharge curve was developed. In addition, at the 250 cfs discharge point velocities were measured throughout the water column at several verticals to calculate shear velocity and grain stress values (Wilcock, 1996). These data were used to verify the grain stress values predicted in the bedload modeling described below.

All bedload samples were sent to a geotechnical laboratory to be dried, weighed, and sieved for particle size gradation analysis. The median grain diameter of all the bedload samples is around 1 mm (coarse to very coarse sand). As shear stress on the bed rises with increased discharge, the percent sand content in the bedload samples decreases (range of 69% to 94%) since more gravel is in transport. Rouse number calculations based on grain diameter fall velocities and channel hydraulics were performed at the bedload measurement cross-section (RS 25+670) to determine the grain sizes expected to be transported in full suspension (suspended load), in incipient suspension (saltating load), or no suspension (traction bedload). Based on results presented in Figure 8 for discharges over the range of the bedload measurements (115 cfs to 260 cfs), grain sizes finer than approximately 0.3 mm (sediment up to the fine sand size) are transported in full suspension, and therefore are considered to be part of the suspended load, and not the bedload. The Rouse number calculations support the suspended sediment gradation analysis described above (Figure 7) in which approximately 90% of the sediment load is finer than 0.3 mm for discharges measured during the bedload measurements. All sediment finer than 0.3 mm trapped in the Helley-Smith sampler net was excluded from the bedload calculation, which accounts for approximately 2 percent of the total sediment volume trapped in the sampler net for each sample. Measured bedload transport values in tons per day are displayed in Figure 9.

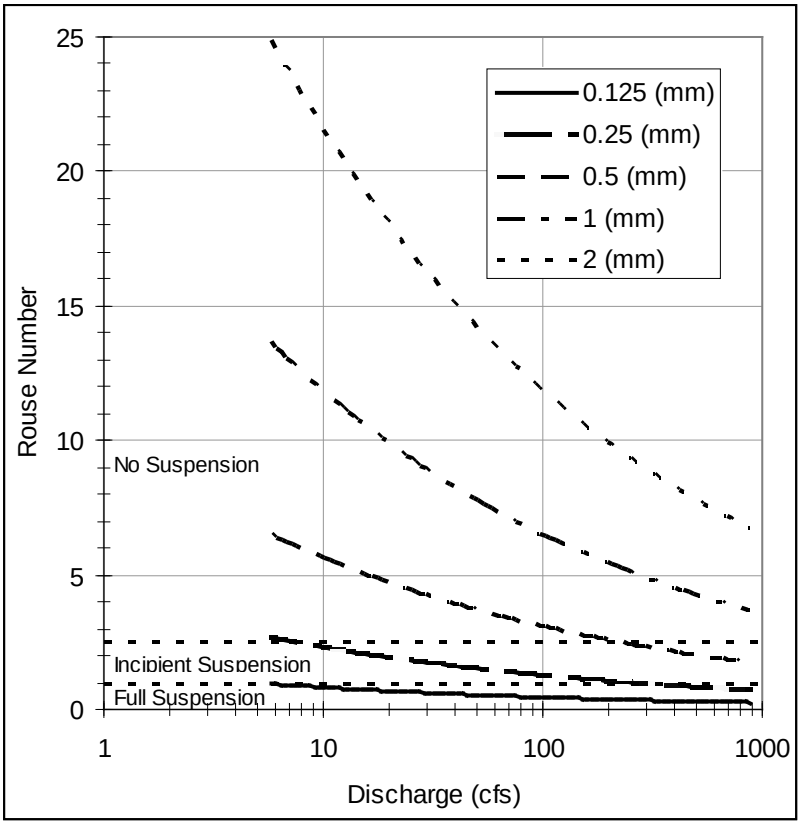


Figure 8. Rouse number calculations at the Sunset Stables bedload measurement cross-section (RS 25+670)

Sediment Transport Modeling

Sediment transport modeling was performed in 2006 for this project to predict bedload transport rates at RS 25+670 for discharges higher than what could be safely and accurately measured by wading in the field. The Wilcock (2001) two-fraction bedload transport formula was used in the sediment modeling because it is an approach designed for calibrating modeled bedload with transport rates measured at lower discharges in order to predict transport rates at higher discharges. Furthermore, the model is designed for predicting sediment transport in rivers like the UT that are composed of mixed sand and gravel sediment in which partial transport of sand over a coarser gravel bed is the dominant mode of transport. The Wilcock (2001) method requires: 1) collection of bedload samples under relatively low flow conditions that enable better precision and accuracy in sampling technique, 2) information on the percentages of sand and gravel on the bed surface, and 3) detailed knowledge of channel hydraulics at the bedload sampling cross-section.

Channel hydraulic data was obtained from a combination of field topographic measurements of the channel topography and water surface slopes, and from hydraulic modeling of the sediment transport modeling reach. Analysis of the sediment samples taken at RS 25+670 shows that sand composes about 65% of the bed surface sediment.

Calibration of the Wilcock (2001) transport function with the measured data requires specification of the reference shear stress (τ_r) that produces the best fit to the measured data. The reference shear stress τ_r is the shear stress (τ) value at which sediment transport is negligible, and τ_r is considered a measurable surrogate for the critical shear stress (τ_c) required for incipient motion of a given grain diameter (Wilcock 2001). The advantage of the Wilcock (2001) approach is that the critical dimensionless shear stress (Shields number, τ^*_c) for incipient motion is calculated from the measured data, thereby reducing much of the uncertainty that is typically involved for selecting an appropriate τ^*_c number to use in sediment transport equations.

A gravel reference shear stress (τ_{rg}) of 0.022 lb/ft^2 provided the best fit to the gravel fraction of the measured data. This results in a calculated dimensionless reference Shields number for gravel ($\tau*_{rg}$) of 0.01. A reference Shields number of 0.01 is low compared to commonly reported values for uniform gravel sediment (Buffington and Montgomery, 1997). However, as shown by Wilcock (1998), a high percentage of sand on the bed surface has the effect of lowering the critical shear stress necessary to mobilize gravel. The $\tau*_{rg}$ of 0.01 calculated from the measured bedload is nearly the same $\tau*_{rg}$ value Wilcock (1998) reported for sediment mixtures with approximately 65% sand.

A sand reference shear stress (τ_{rs}) of 0.007 lb/ft^2 provided the best fit to the sand fraction of the measured data. This results in a calculated reference sand Shields number ($\tau*_{rs}$) of 0.03. The $\tau*_{rs}$ of 0.03 calculated from the measured bedload is slightly lower than the trendline $\tau*_{rs}$ value of 0.04 Wilcock (1998) reported for sediment mixtures with approximately 65% sand, yet it is within the scatter of the data points.

These results indicate that application of the Wilcock (2001) calibrated formula provides a useful prediction of sediment transport at higher discharges since the modeled transport

curve fit to the 2006 measured data is based on quite reasonable reference Shields number values.

In addition to the Wilcock (2001) formula, the Ackers and White (1973) fractional bed-material load equation modified with Proffitt and Sutherland's (1983) hiding function (hereby abbreviated as AWPS) was also used to model sediment transport at RS 25+670. The AWPS equation has been found to provide good predictive results for mixed sand and gravel beds with grain sizes and bed slopes similar to the UT River at the modeling site (Batalla, 1997; Reid and Dunne, 1996). The AWPS model is not a calibrated approach, and thus the modeled results are independent of the measured bedload rates. The same grain size gradations and channel hydraulics used in the Wilcock (2001) calculations were used to calculate transport with the AWPS equation. Results in Figure 9 show that the sediment transport rating curve predicted by the AWPS equation is in strong agreement with both the measured bedload data and the Wilcock (2001) modeled transport.

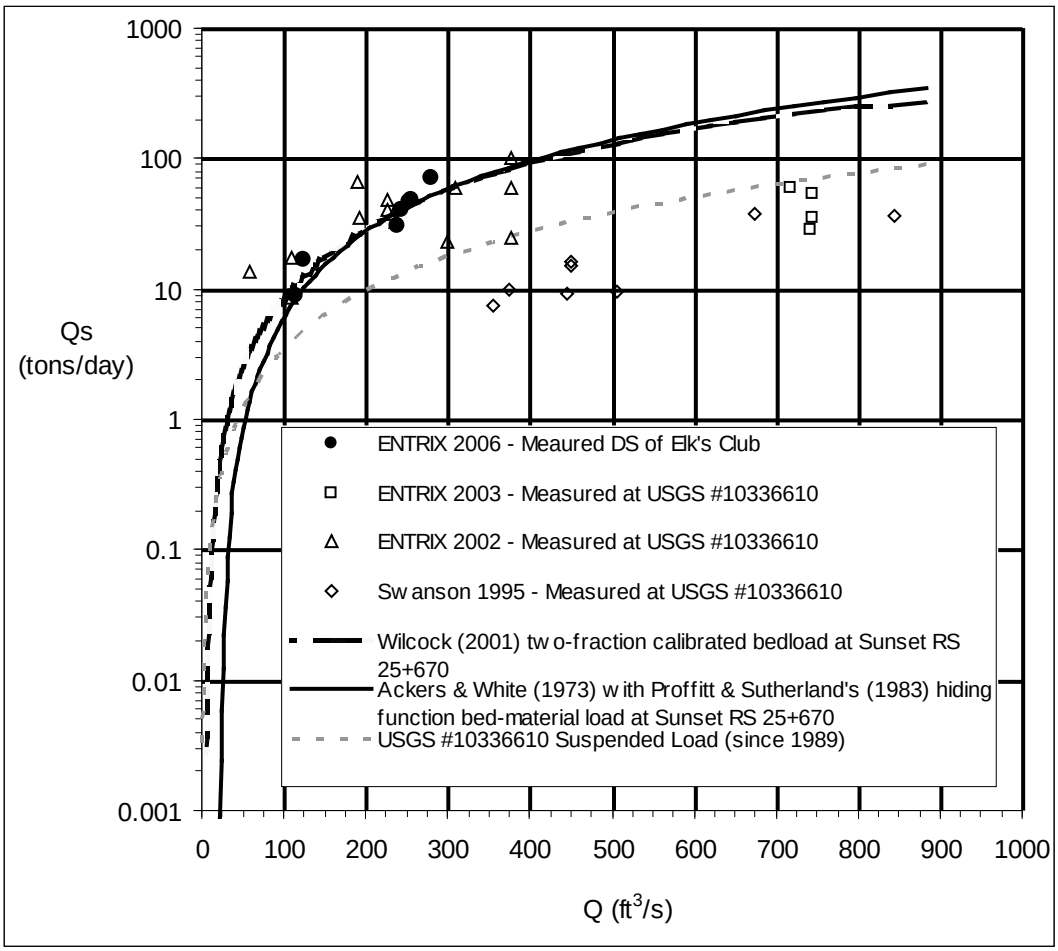


Figure 9. Comparison of UT River measured and modeled bedload transport rates at the Sunset Stables bedload measurement cross-section downstream of the Elk's Club (RS 25+670)

Sediment transport predicted by both the Wilcock (2001) and AWPS equations correlate well with the 2006 measured bedload at Sunset Stables, and is within the range of the

ENTRIX 2002 bedload sampled downstream of Lake Tahoe Blvd. For a 450 cfs flow, the transport function predicts a sediment discharge of about 115 tons/day.

Bedload has been measured in three different years at the pedestrian bridge (USGS gage #10336610) just downstream of Lake Tahoe Blvd. over discharges ranging from 58 cfs to 845 cfs (Figure 9) (Swanson 1995, ENTRIX 2002 and 2003, as presented in CTC, 2004c). The range in measured bedload rates varies widely, particularly between the Swanson 1995/ENTRIX 2003 data and the ENTRIX 2002 data. In part because of the variability in the measured bedload, at the pedestrian bridge, ENTRIX modeled bedload transport at this site in the UT Marsh to see how it would compare with the measured values. The Wilcock and Crowe (2003) surface-based equation for mixed-size sediment was used for the bedload modeling. Similar to the Wilcock (2001) calibrated formula, the Wilcock and Crowe (2003) model is useful for modeling bedload transport in mixed sand and gravel streams in which partial transport of sand over a coarser gravel bed is the dominant mode of transport. Unlike the Wilcock (2001) formula, the Wilcock and Crowe (2003) equation was not designed for calibration to measured bedload data, and thus the modeled values are determined independently of the measured bedload.

A sediment sample taken of the bed at the pedestrian bridge has a median grain diameter of 5.0 mm and 27% surface sand content. As shown in Figure 10, the Wilcock and Crowe (2003) modeled bedload transport plots near the Swanson 1995 and ENTRIX 2003 values, but well below the ENTRIX 2002 and 2006 values.

Cross-section 26 (RS 24+540) in Sunset Stables is located in a plane bed reach with a bed sediment composition (median grain diameter of 4.3 mm and 37% surface sand content) quite similar to the bedload sampling site at the pedestrian bridge in the UT Marsh, yet appreciably coarser than the sandy bed sediment at the Sunset Stables bedload measurement site. Additional bedload modeling was performed at this location to compare with bedload measurements and modeling at both the UT Marsh and in Sunset Stables in an effort to gain further insight into the observed variability in bedload transport rates[4]. The Wilcock and Crowe (2003) model predicts bedload transport rates quite similar to the rates predicted at the UT Marsh site, which are on the low end of all the measured values. The fractional cumulative grain size distributions for discharges ranging from 15 cfs to 610 cfs predicted by the Wilcock and Crowe (2003) model at Sunset Stables cross-section 26 are displayed in Figure 11. The median transported grain size typically varies from medium to coarse sand, which demonstrates the dominant transport of sand sized material over a coarser gravel substrate.

Similar to the Wilcock and Crowe (2003) model, application of the AWPS equation at Sunset Stables cross-section 26 also predicts sediment transport rates near the low end of all measured values and the two Wilcock and Crowe (2003) estimates. For a 450 cfs flow, the Wilcock and Crowe (2003) and AWPS transport equations predicts a sediment discharge of about 15 tons/day at Sunset Stables cross-section 26.

[4] The Wilcock and Crowe (2003) sediment transport model was not applied at the Sunset Stables bedload measurement site (RS 25+670) because the high sand content of the bed surface (65%) is beyond the limit for which the transport model was developed.

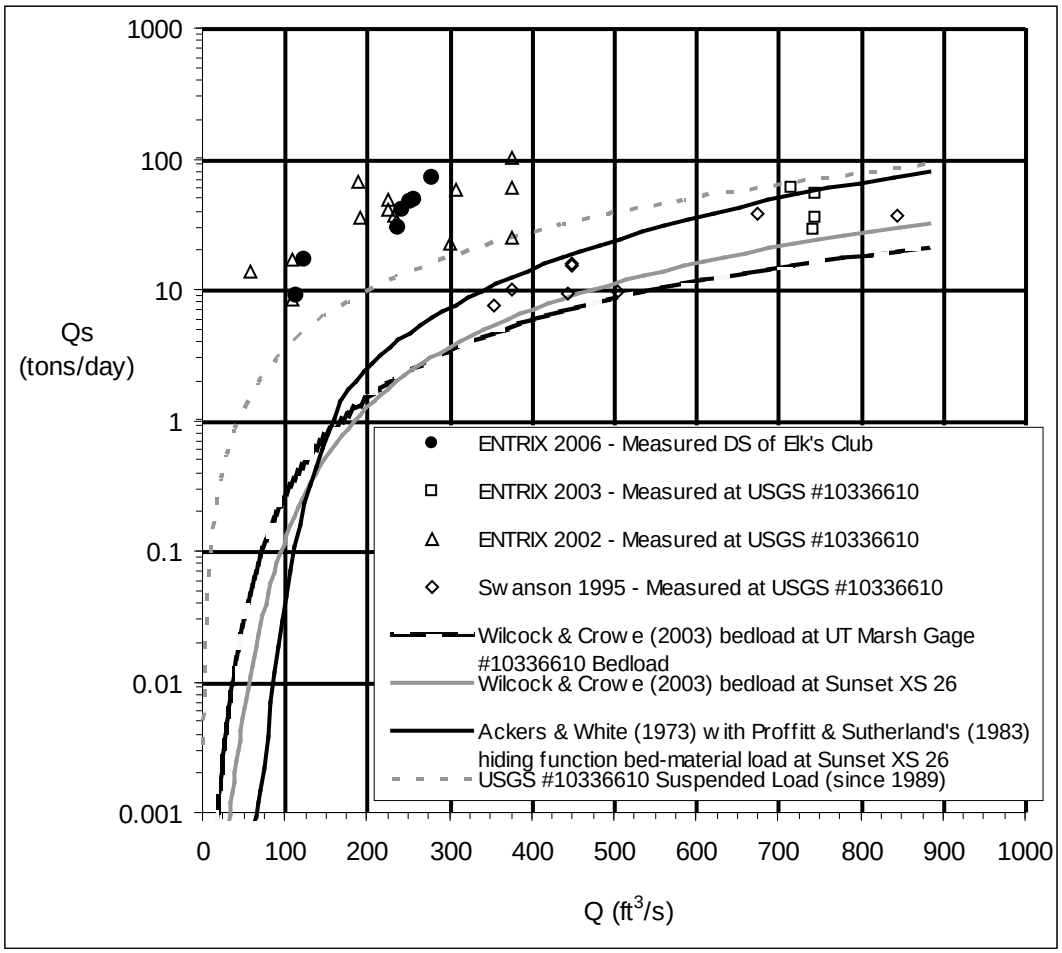


Figure 10. Comparison of UT River measured and modeled bedload transport rates at the UT Marsh pedestrian bridge and at Sunset Stables cross-section 26 (RS 24+540)

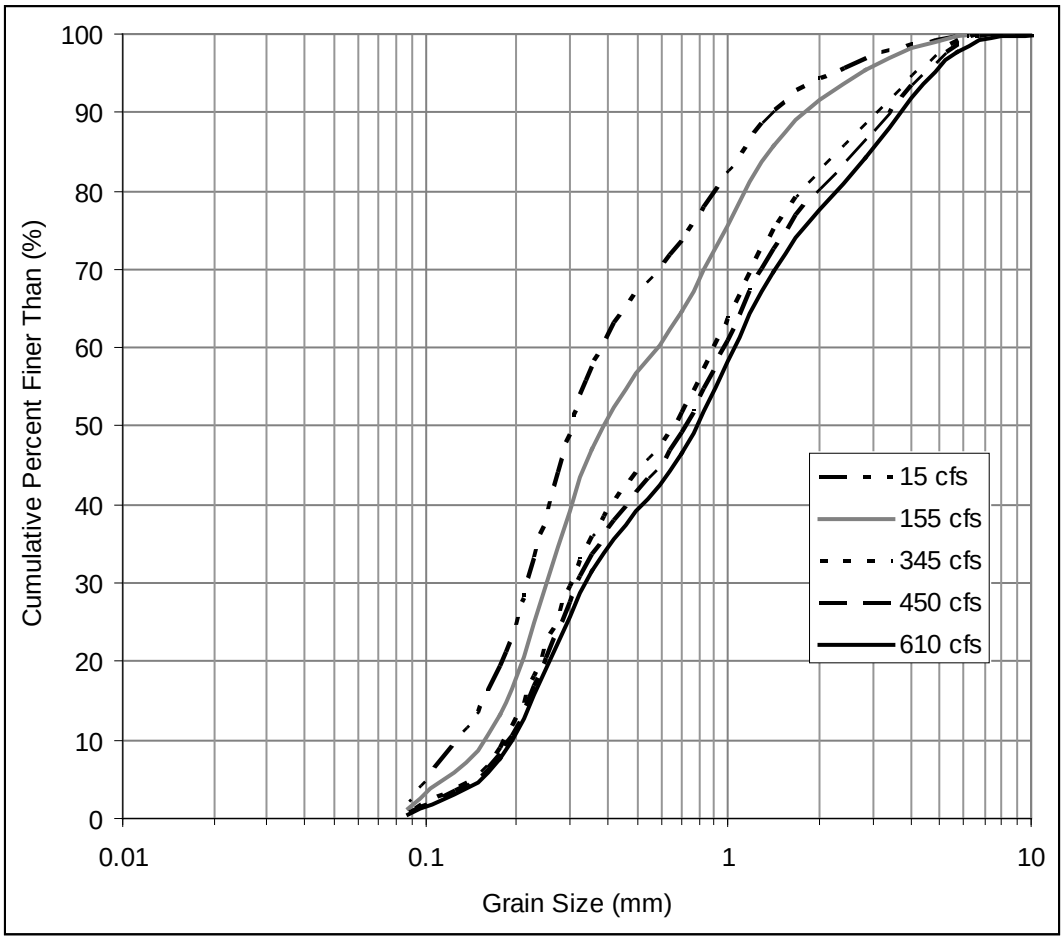


Figure 11. Bedload cumulative grain size distributions predicted by the Wilcock and Crowe (2003) transport model at Sunset Stables cross-section 26 (RS 24+540)

The high variability in sediment transport evident in both the measured and modeled data does not mean that one set of measurements is a more accurate indication of UT River sediment transport than the other. Rather, both the high and low sediment transport estimates likely reflect the true variability in bedload transport that naturally occurs throughout the lower UT River.

In a channel with unchanging hydraulics, in general the size and volume of sediment transported depends on the discharge magnitude and the amount and size of sediment supply available for transport. The median grain size of sediment transported as bedload through Sunset Stables is predominantly medium to coarse sand (approximately 0.5 to 2 mm diameter), yet at higher channel forming flows grain sizes typically up to fine to medium gravel (about 8 to 12 mm) are mobilized and deposited downstream. As is discussed below, larger grain sizes are transported, but only at infrequently occurring discharges.

Through many meander bends the low-flow channel bed surface exhibits a distinct lateral sediment sorting in which one side of the channel is mostly composed of fine to coarse gravel and the other side is mostly sand sized sediment. Patches of sand deposited on coarser gravel substrate are also common in straight reaches. The partial transport of the

sand throughput portion of the bedload likely explains much of the variability in bedload transport exhibited in Figures 9 and 10 (Paola and Seal, 1995). Consider a situation where bedload measurements are collected at a cross-section at the same time a sheet of sand or larger bedform is being transported over the underlying gravel substrate at the sampling site. Then, after some time a second set of bedload measurements are collected at the same cross-section, except this time the wave of sand has completely passed through the sampling site and the bed surface is primarily composed of gravel with interstitial sand. One would expect a higher transport rate to be measured during the first set of measurements with the active sand transport since sand is more easily transported at high volumes than coarser gravel. This is the case with the 2006 bedload sampling in which a large volume of sand was being transported through the sampling site for the duration of the sampling season. The same situation probably resulted in the relatively high transport rates measured by ENTRIX in 2002 at the pedestrian bridge in South Lake Tahoe.

Since the sediment transport equations require input of the fractional grain size distribution of the bed, it should be expected that the equations will predict higher transport rates when applied at cross-sections where the bed surface is composed of relatively fine sediment (e.g. sand) compared to cross-sections where the bed surface is relatively coarse (e.g., fine gravel). This is the case with the transport equations applied at the two different Sunset Stables cross-sections. At the bedload measurement cross-section (RS 25+670), the transport equations predicted high transport because over 65% of the bed material was composed of readily transported sand sized material. At Sunset Stables cross-section 26 (RS 24+540), the bed sediment is coarser and composed of only 37% sand. As the amount of fine material on the bed surface decreases, the ease at which it can be mobilized and transported also decreases since the sand grains become protected from fluid shear by the wakes of the surrounding larger gravel, thereby reducing the overall bedload transport rate.

It is not uncommon for the volume of measured bedload to vary by several orders of magnitude for the same given discharge (Moog and Whiting, 1998). Like suspended sediment, bedload transport rates can also exhibit temporal hysteresis. The supply of sediment from hillslopes, streambanks, and tributaries varies seasonally, and breakup of the coarse gravel pavement on the hydrograph's rising limb can generate a new supply of sediment that persists until the gravel pavement reforms and again restricts the sand supply (Moog and Whiting, 1998). Additionally, sand transported over the underlying gravel during relatively low discharges can collect along streambanks, pools, and other storage areas of relatively low velocities. As this stored sediment is flushed during a rising hydrograph, a river will have a higher bedload transport rate until the stored sediment supply is exhausted (Moog and Whiting, 1998). All of these factors likely contribute to the observed variability in bedload transport observed throughout the lower UT River.

In terms of channel morphology, the channel must be sized to accommodate the relatively higher transport rates resulting from passage of sediment pulses through the system. The channel would likely undergo form adjustments of channel aggradation and widening if it was initially sized according to the lower transport rates and then suddenly had to pass a sediment pulse.

Section 5 - New Channel Design

The first step taken in the channel and floodplain design was to search for suitable reference reaches to the UT River at Sunset Stables. As described in Section 2, the channel in Sunset Stables is degraded throughout the entire project reach, thus limiting its usefulness as an analog for determining new channel dimensions. Likewise, reaches of the UT River up and downstream of Sunset Stables have undergone extensive channel deepening and widening in the past, thus also limiting their usefulness as reference reaches. In addition to the major land use changes that date back to the Comstock Era of the late 1800s, direct channel disturbances have dramatically altered the UT River. Between Sunset Stables and Lake Tahoe Boulevard downstream in South Lake Tahoe, nearly the entire length of the channel was straightened to accommodate airport construction or grazing. In the California State Parks reach upstream of Sunset Stables, construction of Highway 50 and of the golf course resulted in a relocated and straightened channel. Any channel planform observed from historic air photos in 1940 is the product of highly altered hydrologic and sediment regimes, and direct channel modifications that date back to the Comstock Era. Just as the existing channel continues to adjust its geomorphic form, the channel observed in the historic air photos was also adjusting to the major watershed land use changes. ENTRIX GIS analysis of a relatively long and continuous remnant channel reach in the west Mosher meadow upstream of Lake Tahoe Boulevard that possibly pre-dates the Comstock Era showed the channel had a sinuosity of about 2.71 and mean slope of 0.0007. This single thread remnant channel reach has a substantially lower slope and higher sinuosity than existing or historic active channels dating back to 1940, which suggests that it functioned under a different prevailing hydrologic and sediment condition.

Channel Pattern

The most basic step in designing the new channel is to determine whether the channel should have a straight, meandering, wandering, braided, or anastomosing channel pattern. The 1940 photo of the UT River in the project area shows a sinuous, single-thread channel with irregular meander bends (Figure 1). The presence of well-defined and unvegetated point bars, as well as numerous channel remnants on the floodplain, suggests an actively meandering channel. Likewise, the existing channel planform also exhibits an actively meandering channel planform. Given a 450 cfs bankfull discharge and a 0.001 channel slope, the UT River plots well within the “meandering” channel data on Leopold and Wolman’s (1957) channel pattern threshold relationships.

Past research has related a channel’s pattern to the quantity and dominant mechanism (bedload vs. suspended load) of sediment transport. *Bedload* channels are commonly defined as channels that transport more than 11% of their total sediment as bedload, whereas *Mixed Load* channels transport between 3-11% of their total load as bedload (Figure 12) (Schumm, 1977). During the passage of sand sheets through Sunset Stables, sediment transported as bedload accounts for approximately 76% of the total annual sediment load (sum of bedload and suspended load). More often, sand sheets are not passing through the system, thus bedload transport rates are lower and account for

approximately 14% of the total annual sediment load[5]. The lower value of 14% for bedload transport as a percentage of total transport is more representative of the prevailing sediment transport regime on the UT River in the project area since the higher value of 76% when sand sheets are passing though is less common. Based on a 14% value, the UT River in the project reach is near the boundary of *Mixed Load* and *Bedload* type systems, and is best depicted by schematic 3b in Figure 12, which is an actively meandering channel with point-bars. The existing channel sinuosity of 1.56 and 1940 channel sinuosity of 1.55, which are between the reported sinuosities for *Mixed Load* (<2.0) and *Bedload* (<1.3) channels, also supports the conclusion that an actively meandering channel with point-bars is the appropriate channel planform for the new channel based on the quantity and dominant mechanism of sediment transport.

Multiple lines of evidence, including historic photographs, slope-discharge relationships, and the nature of sediment transport, indicate that a meandering channel pattern is most appropriate for the UT River in the project area. The morphologic characteristics of a *Mixed Load* system described in Figure 12 have design implications for the new channel, and are discussed further below.

[5] Bedload transport as a percentage of total sediment transport calculations are based on bedload measurements and modeling performed by ENTRIX and described in detail in Section 4 and in CTC 2007a. Suspended sediment volumes are based on the best-fit line for data collected by the USGS at their gage #10336610 in S. Lake Tahoe (Figure 6). Total annual sediment loads are calculated by applying the sediment discharge rating curves over an annual hydrograph of median mean daily discharges.

Type of channel	Bed load (% of total load)	Single-channel systems	Multiple-channel systems
Suspended load	< 3	Suspended-load channel W/D ratio < 10, sinuosity > 2.0, gradient relatively gentle	Anastomosing system
Mixed load	3–11	Mixed-load channel W/D ratio 10–40, sinuosity < 2.0, gradient moderate, may be braided	Delta distributaries Alluvial plain distributaries
Bed load	> 11	Bed-load channel W/D ratio > 40, sinuosity < 1.3, gradient relatively steep, may be braided	Alluvial fan distributaries

W/D=width/depth ratio.

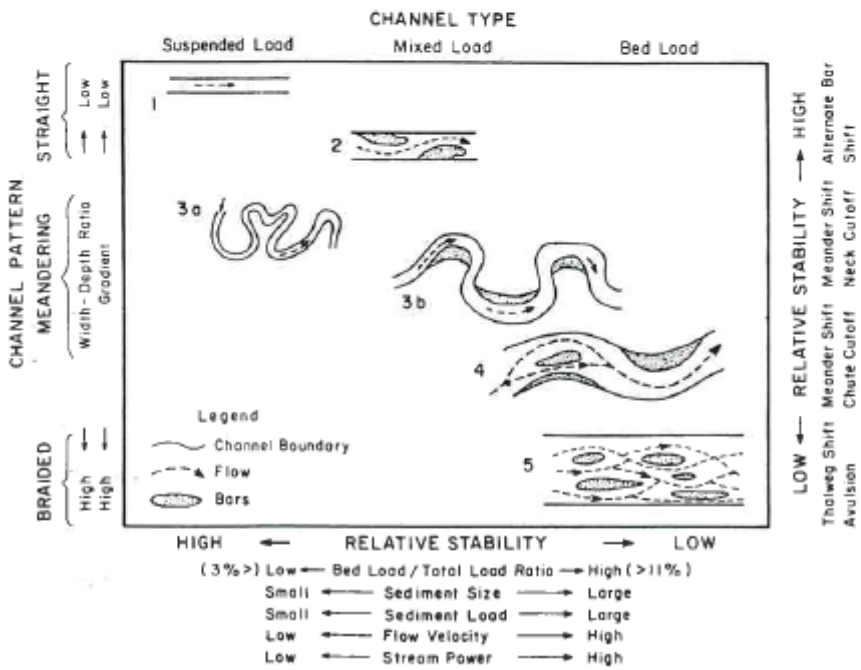


Figure 12. Alluvial channel patterns (Source: Miall, 2006, from Schumm, 1985)

The *Gravel-Sand Meandering River* model described by Miall (Miall, 2006) closely depicts the type of channel this project aims to create (Figure 13):

> They are bed load streams, in which the channel and macroform sediments consist of sand and pebbly sand, or sand with a lag of gravel. Abandoned channels and meander scars are common on the floodplain, and are preserved as clay-silt plugs (p. 215).

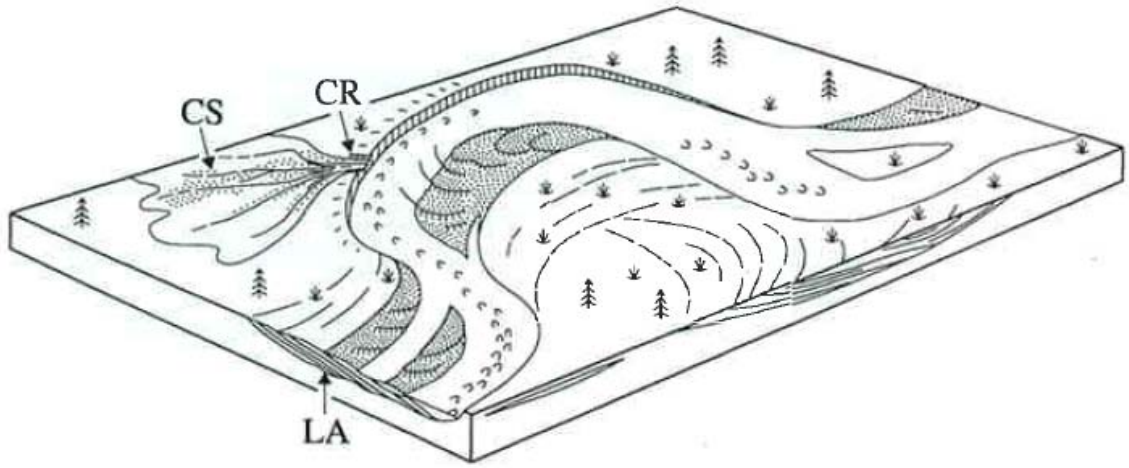


Figure 13. Gravel-sand meandering river architectural model. Source: Miall, 2006, p. 222

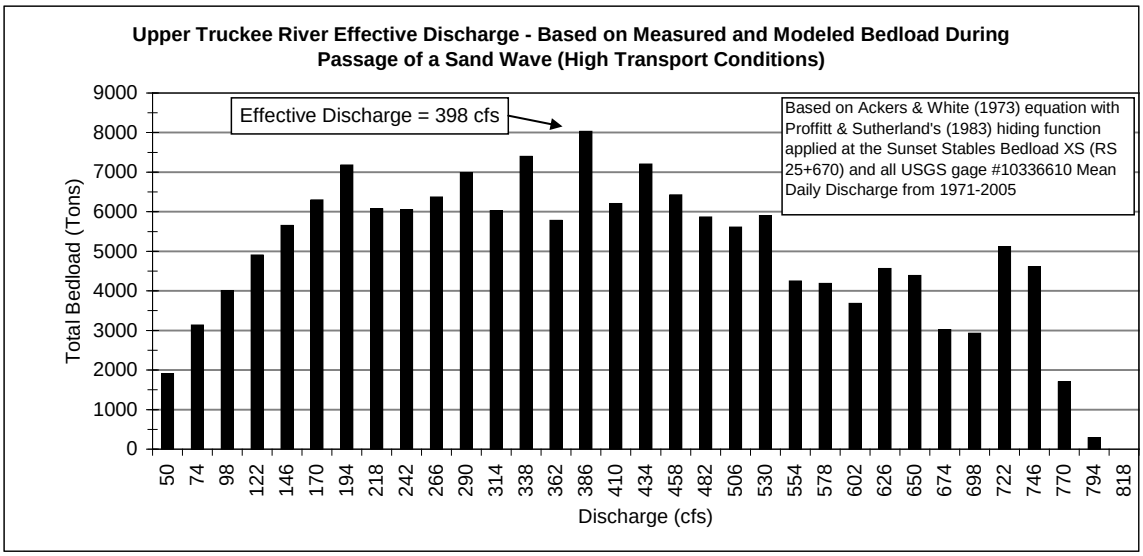
Design Discharge

The flow conveyance capacity of the new channel, or design discharge, has to be specified prior to determining the new channel's dimensions. For this project, the design discharge is the bankfull flow, which is the maximum amount of water the channel can convey without overtopping its banks and inundating the floodplain. ENTRIX conducted a channel-forming flow assessment (CTC, 2003) to determine the design discharge for a restoration project on the UT River downstream of the project reach in the UT Marsh. Through analyses of the effective discharge, flood frequency analysis, and bankfull geomorphic indicators, ENTRIX determined to use 450 cfs as the channel-forming flow. Since that report was written, 4 more years of flow data and the analysis of the sediment load coming into the Sunset Stables are available. With the additional flow data, a 450 cfs flow still has a peak annual recurrence interval of approximately 1.4 years, and occurs about 15 days per year, on average, based on mean daily discharge records at USGS gage #10336610 in S. Lake Tahoe.

Research has shown that channel form and size is often adjusted to the effective discharge[6]. With the new sediment transport data measured and modeled in Sunset Stables, effective discharge calculations were updated, and are presented below (Figure 14). Effective discharges were calculated for both bedload transport conditions described on the UT River: 1) high transport conditions during passage of sand sheets, and 2) low transport conditions when sand sheets are not passing through. In either case, the effective discharge is approximately 400 cfs.

[6] The effective discharge is often defined as the discharge that if constantly maintained in an alluvial stream would produce the same channel morphology as is produced by the entire hydrograph. It is considered to be the flow that performs the most geomorphic work in terms of sediment transport (Wolman and Miller, 1960). While larger flood flows may transport more sediment for a single runoff event, their cumulative affect on channel morphology is not as significant as more frequently occurring moderate high-flow events. Similarly, frequently occurring small magnitude discharges do not transport enough sediment to control channel morphology (Knighton, 1998).

Based on the updated analyses, a 450 cfs discharge is still appropriate to use as the design discharge since it is near both the commonly used 1.5-year return flow and the effective discharge.



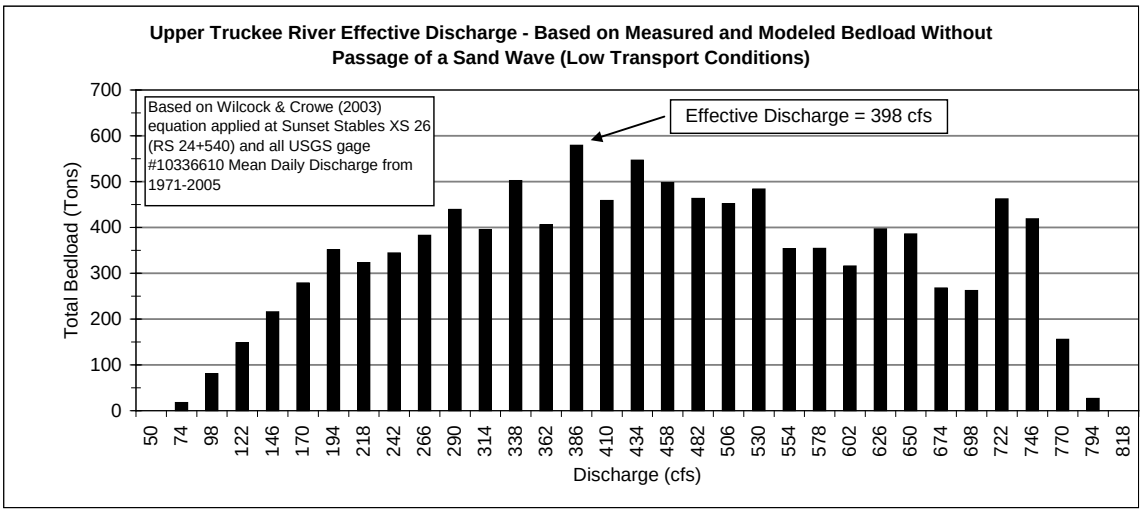


Figure 14. UT River effective discharge calculations for variable sediment loads

New Channel Dimensions

Countless combinations of channel width, depth, and slope could be used to construct a new channel sized to reach bankfull at 450 cfs. The objective of this design is to use a combination that provides a channel geometric form and sinuosity appropriate for a meandering channel in a gently sloping valley that will be in equilibrium with the prevailing hydrologic and sediment regimes. Determination of how wide, deep, and steep the channel should be required a combination of empirical and analytic analyses described below.

ENTRIX used the "equilibrium slope" method (Wilcock, 2004) for this project to determine the combinations of channel depth and slope required to pass the estimated bedload of a 450 cfs channel-forming flow while maintaining channel stability (i.e., neither aggrading or degrading, nor undergoing channel widening or narrowing). This type of analysis is supported by researchers in the field of stream restoration as a process-based method of designing new channels (Shields et al., 2003; Soar and Thorne, 2001). The equilibrium slope method requires specification of channel width and a bedload supply before channel depth and channel slope can be solved iteratively.

Channel Width

Hydraulic geometry equations were analyzed to select the mean channel design width. The hydraulic geometry relationships outlined in Philip J. Soar and Colin R. Thorne's USACE publication, "Channel Restoration Design for Meandering Rivers," (2001) and the equations of Huang and Nanson (1998) were utilized for this task. Soar and Thorne (2001) compiled several data sets of channel width hydraulic geometry relationships for sand and gravel beds with banks of varying cohesiveness. For a 450 cfs design flow (i.e., the bankfull flow), the average channel width predicted by the relationships for gravel beds with cohesive banks is 35 ft. For sand bed channels with cohesive banks, the predicted channel width is 39 ft.

Huang and Nanson's (1998) equation for channel width requires specification of the channel slope and Manning's roughness n value. A mean slope of 0.001 and an n value of 0.035 were specified (typical for the UT River in the meadow reaches). For a 450 cfs flow with moderately cohesive sand banks, the predicted channel width is 45 ft. If the moderately cohesive sand banks are vegetated, thus increasing erosion resistance, the predicted channel width is 37 ft. For highly vegetated and highly cohesive sand the predicted channel width is 27 ft. For highly cohesive sand with no modifier for vegetative cover, the predicted channel width is 31 ft.

Based on the above analysis, a final channel width of 38 ft was chosen as the design standard. This is within the range of Soar and Thorne's (2001) values for gravel and sand bed channels with cohesive banks, and Huang and Nanson's (1998) width prediction for channels with moderately vegetated and moderately cohesive sand banks. Note that the 38 ft width is based on a rectangular channel to simplify the calculations to determine the equilibrium slope and depth. While 38 ft was selected as the channel width at a typical riffle (meander bend cross-over) location, the constructed width will vary continuously to account for changes in channel planform (e.g., at meander inflection points and apexes) and soil bank texture and cohesion.

Channel Depth and Slope

Specifying a flow of 450 cfs and a channel width of 38 ft, calculation iterations were performed to solve for the channel depth and channel slope combination needed for equilibrium transport of the bedload supplied to the new channel from upstream in Sunset Stables. Two bedload supply scenarios were analyzed: 1) a low sediment supply of 15 tons/day at 450 cfs representing the prevailing sediment conditions, and 2) a high sediment supply of 122 tons/day representing passage of sand sheets. Additional specified input included the Manning's n roughness value, the median grain size (D_{50}), and the critical Shields number (τ^*_c) for sediment mobility. A Manning's n of 0.035 was

specified since it is the most common value used in the roughness calibrated HEC-RAS model for the project and is appropriate for a meandering sand and gravel channel. Median grain size values of 4.5 mm during low transport conditions, and 1.4 mm during high transport conditions when sand sheets are passing through, correspond to the bed surface sediment sampled at the sediment transport cross-sections. Specification of critical Shields values of 0.039 for 4.5 mm sediment and 0.031 for 1.4 mm sediment are calculated from Wilcock's (2004) analytical fit of the original Shields curve.

Two separate sediment transport equations that fit the measured sediment transport data well were used to solve for depth and slope to compare the sensitivity of the calculations to the transport equations. The Meyer-Peter Müller (Meyer-Peter and Müller, 1948) equation modified with Wiberg and Smith's (1989) calibrated transport parameter (α) was used to calculate depth and slope for both low and high sediment supplies. In addition, ENTRIX performed equilibrium slope calculations based on the Ackers and White (1973) equation with Proffitt and Sutherland's (1983) hiding function for the case of a high sediment supply since it correlates well with the measured high sediment transport rates (Figure 9).

Results from the equilibrium slope analyses are presented in Table 5. The calculated depths, velocities, and slopes do not vary substantially between the high and low sediment supplies or between the sediment transport equation used. From the output, ENTRIX established the preliminary mean values for the new channel, which are displayed in Table 6. These are the mean values for a rectangular channel representative of the symmetrical bed and bank geometry typically located at riffle cross-sections where the channel transitions from one meander bend into the next bend downstream. The next step was to convert the rectangular channel geometries into a more realistic trapezoidal geometry with non-vertical banks. The meadow soils throughout Sunset Stables are rather cohesive with high percentages of silt and clay content (ENTRIX sediment sampling results show the existing channel banks are composed of 46% silt/clay, on average) (CTC, 2007a). The preliminary soils data was used to develop the final mean channel dimensions representative of a typical riffle crest listed in Table 7. These are the new channel dimensions used in the Alternatives HEC-RAS models. The banks for the prototype riffle are rather steep at 67.5 degrees. Additional research, including detailed geotechnical data studies throughout the project area, will be required to refine the steepness of the final design channel banks for pool, riffle, and transition cross-sections so they are constructible and sustainable with the existing soils.

The final channel dimensions result in a channel width to channel depth ratio of 10. Prior research has related the percent of silt and clay in the channel perimeter to the width to depth ratio of the channel (Schumm, 1960). Based on the amount of silt and clay present in ENTRIX's bank sediment sampling, Schumm's equation predicts that the width to depth ratio should be approximately 12. Although this equation is by no means exact in its predictive capability, it does give an indication that the final widths and depths predicted by the equilibrium slope analysis and listed in Table 7 are reasonable values for a mixed sand and gravel channel meandering through cohesive meadow soils in a gently sloping valley.

Channel Sinuosity

The sinuosity of the new channel is the channel length divided by the valley length[7]. Since the valley length is fixed as the straight line distance between where the new channel starts and ends, selection of the channel slope and starting and ending channel elevations determines the channel sinuosity. As described above, the target reach average slope for the new channel is 0.00105. Since channel slope is already determined, channel length and sinuosity is ultimately dependent upon the starting and ending channel elevations. The difference in elevation between the start and end of the channel determines how long the channel needs to be to maintain the target slope. A relatively high difference in elevation between the start and end will require a longer channel to maintain slope, while a relatively low difference will require a shorter channel.

A major objective of this restoration project is to reconnect the channel with the existing floodplain surface. The bed elevations in the new channel should be constructed at an elevation that will enable a 3.9 ft deep channel (on average) to achieve bankfull capacity around 450 cfs and begin inundating the existing floodplain surface with increased flow. Two perceivable scenarios arise if the bed elevations of the new channel are set too low. If the existing ground (i.e., floodplain) surface is used as the new channel top-of-bank, then the distance between the existing floodplain surface and the new bed will be greater than 3.9 ft, resulting in a channel that is possibly unstable and too deep to inundate the floodplain as frequently as desired since more flow will be required to overbank. Or, some of the existing floodplain could be excavated to lower the floodplain surface so that the new top-of-bank elevations maintain of a 3.9 ft deep channel. Conversely, if the new bed elevations are set too high, two additional scenarios could arise. If the existing ground surface is used as the new channel top-of-bank, then the distance between the existing floodplain surface and the new bed will be less than 3.9 ft, resulting in a channel that is too shallow and possibly unstable. Or, filling of the floodplain would be required to build up the height of the channel so that the new bank elevation is high enough to maintain a 3.9 ft deep channel. Creating sections of deep or shallow channel, or excavating or filling the floodplain to create the proper size channel should be limited as much as possible.

The desire to create a new channel with bed elevations set 3.9 ft deeper than the existing ground surface means there is little flexibility in determining the new channels vertical elevations. Therefore, the new channel's length and sinuosity is not an independent design variable that can be adjusted substantially higher or lower. Channel length and sinuosity are dependent upon the pre-determined channel slope and starting and ending elevations. Although the planforms of the channels in Alternatives 2-4 vary, the starting and ending elevations are quite similar. Alternative 2 has a new channel sinuosity of 1.56 and Alternative 3 has a new sinuosity of 1.60. Since Alternative 4 uses the same channel planform as Alternative 3 in the new channel reach, it also has a new channel sinuosity of 1.60. These new channel sinuosity values are essentially the same as the existing channel's reach average sinuosity of 1.56, and the 1940 channel sinuosity of 1.55.

[7] The valley slope of Sunset Stables is 0.0016. This calculation is based on a valley floor alignment that follows the valley topography and the general axis of the UT River.

Channel Planform

The channel planforms depicted on the Alternatives maps (Appendix A) were created by digitizing for each Alternative a line of the required length that provides the correct channel slope of 0.00105. The line was digitized to have irregular shaped meander bends similar to existing conditions and those shown in the 1940 air photograph. The alignments of the new planforms are based on several conditions, such as the need to maintain top-of-bank elevations close to the existing ground surface, the need to avoid existing pipelines, and the need to limit the number of crossings of the existing channel. Although the alignments on Appendix A depict channels with the correct target slope and sinuosity, they are still conceptual level alignments. Additional design work will be conducted to refine the final alignment of the preferred Alternative.

Table 5. Specified input and calculated output for the equilibrium slope analysis to calculate new channel dimensions.

Specified Input						Calculated Output				
Flow Discharge (Q in cfs)	Bedload Discharge (Qs in tons/day)	Median Grain Size (D_{50} in mm)	Channel Width (B in ft)	Critical Shields Number (τ^*_c)	Mannings n	Grain Stress (τ' in lb/ft^2)	Mean Velocity (U in ft/s)	Mean Depth (H in ft)	Hydraulic Radius (R in ft)	Slope (S)
Meyer-Peter Müller (1948) Equation with Wiberg & Smith (1989) Transport Parameter (α)										
450	15	4.5	38	0.039	0.035	0.072	2.9	4	3.3	0.0010
450	122	1.4	38	0.031	0.035	0.058	3.1	3.8	3.2	0.0011
Ackers & White (1973) Equation with Proffitt & Sutherland's (1983) Hiding Function										
450	122	1.4	38	0.031	0.035	NA	3.0	3.9	3.3	0.0010

Table 6. Final rectangular channel design parameters determined from equilibrium slope analysis. Note numbers are mean values.

Flow Discharge (Q in cfs)	Channel Width (B in ft)	Mean Depth (H in ft)	Slope (S)	Mannings n	Boundary Stress (τ in lb/ft^2)	Mean Velocity (U in ft/s)	Hydraulic Radius (R in ft)
450	38	3.9	0.00105	0.035	0.21	3.0	3.3

Table 7. Final riffle-crest channel design parameters used to represent the new channel in the HEC-RAS model.

Flow Discharge (Q in cfs)	Channel Top-Width (H in ft)	Channel Bottom Width (B in ft)	Bank Angle (Degrees)	Mean Depth (H in ft)	Slope (S)	Mannings n	Boundary Stress (τ in lb/ft^2)	Mean Velocity (U in ft/s)	Hydraulic Radius (R in ft)	Width to Depth Ratio
450	39.1	35.8	67.5	3.93	0.00105	0.035	0.21	3.1	3.3	10

Section 6 - Hydraulic Modeling of the Alternatives

Alternatives 2-4 were modeled in HEC-RAS software to evaluate their hydraulic performance and compare between the Alternatives and the existing condition[8]. The output from the hydraulic modeling provides much of the quantitative data used in the Alternatives evaluation ranking process to select the preferred Alternative. The following is a presentation of the methods and assumptions made in the hydraulic analysis. This is followed by presentation and discussion of the model's results.

HEC-RAS Model Input

Each Alternative's HEC-RAS geometry file includes cross-sections in the CSLT airport reach downstream and the California State Parks golf course reach upstream. The Alternative's models use the new channel geometry proposed for the restored CSLT airport reach channel. This restoration project is currently at the 95% design level. No major changes are expected to the channel geometry or elevations prior to its scheduled construction starting in summer 2008. It was decided to use the proposed new airport reach channel in the model because it presents the most likely downstream hydraulic conditions for the future Sunset Stables channel that must be accounted for in the Sunset design. In the golf course upstream of Sunset Stables, the existing channel geometry was included in the model. Even though a restoration project is proposed for the golf course reach, it is at a much earlier design stage than the airport reach and no detailed designs have been created yet. The assumed geometry of the golf course reach does not impact the modeled hydraulics in Sunset Stables since HEC-RAS is a fixed-bed backwater model. Cross-sections from the golf course reach were included in the model to analyze the hydraulic impact each Sunset Stables Alternative has on existing channel conditions in the golf course reach.

New floodplain and channel cross-section geometries were created for each of the Alternatives. Maps of each Alternatives 2-4 cross-section locations are presented in Appendix A. Approximately 32 cross-sections are included in each Alternative model for the Sunset Stables reaches. Many additional cross-sections are also included for the bounding golf course and CSLT airport reaches. Cross-sections were drawn across the entire Sunset Stables floodplain and crossing perpendicular to the new channel planforms. The majority of the cross-sections cross the new channel planforms at the riffle crossover location on the meander bends. This was done for two purposes: 1) cross-sections are needed at riffles in the hydraulic model because riffle cross-sections control water surface profile elevations, and 2) the model uses the new channel geometry dimensions listed in Table 7, which are representative of a near-symmetrical geometry typically located at riffle crossover locations. Additional cross-sections are located at important hydraulic transitions, such as the Highway 50 bridge, transitions into and out of new channel reaches, and at proposed in-channel structures (e.g., the grade control structures in Alternative 4).

[8] See CTC 2007b "Results of Sunset Stables Sub-Task 4C2: Topographic Longitudinal Profile and Cross-Section Surveys and Hydraulic Modeling" for a detailed description of the data input, modeling assumptions, and hydraulic output of Sunset Stable's existing conditions.

The cross-section topography is based on a DTM surface constructed from the 2003 1-ft contour interval LiDAR elevations. Field surveyed elevations are used to supplement the DTM surface wherever possible. All elevations are in NGVD29 feet.

The same Manning's n floodplain and channel roughness values assumed in the existing condition HEC-RAS model (CTC 2007b) are used in the Alternatives modeling. Cross-sections located in new channel reaches are assigned a 0.035 n value for the channel. This value is similar to the n values used in the channel of the existing condition model, and is appropriate for a meandering, low-gradient, mixed sand and gravel channel (Chow, 1959).

The existing channel is partially filled at all modeled cross-sections where a new channel is proposed. The existing channel is filled at least 2-3 ft from the bottom, and at many locations the fill extends nearly to the flat floodplain surface. Existing channel fill is represented as an obstruction in HEC-RAS.

Five different flow events were modeled for each Alternative in HEC-RAS. These include: 1) 15 cfs, approximate base flow, 2) 450 cfs, 1.4-yr return interval, 3) 760 cfs, 2-yr return interval, 4) 1,600 cfs, 5-yr return interval, and 5) 7,650 cfs, 100-yr return interval. To account for variability in channel and floodplain flow paths at varying flow magnitudes, a different set of channel and floodplain reach lengths were used in the modeling for the relatively low 15 cfs, 450 cfs, and 760 cfs flows versus the relatively high flood flows of 1,600 cfs and 7,650 cfs.

HEC-RAS Model Results

The results from the Alternatives hydraulic modeling are presented in several formats. These include: 1) multiple longitudinal profile plots showing ground elevations, water surface elevations, and various hydraulic parameters, 2) channel and floodplain cross-section plots, and 3) floodplain inundation maps created in HEC-GeoRAS software.

Cross-Section Plots

Cross-section plots for Alternatives 2-4 are located in Appendices B to D, respectively. Each cross-section for all the Alternatives has two plots. One shows the water surfaces elevations for 15 cfs, 450 cfs, and 760 cfs, while the other shows the elevations of the 1,600 cfs and 7,650 cfs flows. Areas where the existing channel would be filled are displayed with black fill (i.e., a HEC-RAS channel obstruction). Ineffective flow areas are also shown in green to delimit locations on the floodplain where water would never flow freely (i.e., a HEC-RAS permanent ineffective flow area, used in areas such as floodplain depressions where water would pond rather than flow freely), or would only flow freely once the water surface elevation reaches a certain elevation (i.e., a HEC-RAS non-permanent ineffective flow area). Manning's n values are displayed along the top of each cross-section.

Left and right top-of-bank elevations are shown as red circles. At cross-sections with new channel, the bank stations were placed 3.9 ft up in elevation from the new channel's bed regardless of whether the elevations of the banks are exactly level with the floodplain surface. This was done to illustrate on each cross-section how much higher or lower the ideal bank elevations using a consistent 3.9 ft deep channel are compared to the floodplain surface. Locations where the bank stations are higher than the floodplain

indicate that local fill placement on the floodplain may be required to meet the elevation of the new channel bank, or the channel may be constructed shallower at that location, or the channel may have to be moved laterally to higher floodplain ground. Conversely, locations where the bank stations are lower than the floodplain indicate the channel is deeper than the target design, thereby resulting in either an increased capacity channel or the need for localized floodplain excavation to match the top-of-bank elevation. Note that since the LiDAR elevations are 1 foot contours, the floodplain elevations on the cross-section adjacent to the new channel could be 0.5 ft different from the true ground elevation since all LiDAR elevations were rounded up or down to the nearest foot. Thus, some of the modeled difference in top-of-bank elevations and the floodplain surface may not be real. The details of the top-of-bank elevations in relation to the existing floodplain surface will require additional attention as the design of the preferred Alternative is advanced. At this level of conceptual design, however, the top-of-bank elevations are realistically close to the existing ground surface. This indicates that the slope and sinuosity of the new channel is compatible with existing valley floor topography.

Longitudinal Profile Plots

Water Surface, Top-of-Bank, and Thalweg Profile Plots

Appendix E displays longitudinal profile plots of water surfaces, top-of-banks, and thalweg elevations for each of the five modeled flows for Alternatives 2-4 versus the existing condition. Note that the "HEC-RAS Cumulative Channel Distance (ft)" X-axis values on the plots relate to the channel distances of the Alternatives. These distances are not the same as the River Station values commonly used for the existing channel. These values represent the new channel lengths of the Alternatives, including the additional channel length that would be created by construction of the new channel in the CSLT airport reach. The existing condition data is plotted using the channel distances of the Alternatives (i.e., the existing condition cross-sections are plotted using the distance on the Alternatives centerline where it intersects the cross-section). This is done to facilitate direct location comparison of the Alternatives with the existing condition. The drawback to this approach is that distances between the existing condition cross-sections are not the true distances of the existing channel. Therefore, the trend line slopes of the existing profiles are inaccurate, yet the absolute values at each cross-section are correct.

The existing and proposed top-of-bank elevations are shown on the longitudinal plots. These lines are useful for examining the relation of the modeled flows to the floodplain surface. In addition, each plot has a "Centerline Existing Ground" profile plotted in black. This line represents the elevation profile of the existing ground surface along the proposed new centerline alignments of the Alternatives. It is included on the plots to illustrate the variability in ground elevations throughout the Sunset Stables valley floor and to show how the new proposed top-of-bank elevations compare to the adjacent existing ground elevations. It is evident on the plots that the proposed (green line) top-of-bank profile plots near the top of the centerline existing ground profile. As discussed above, this was done intentionally so that that the new channel's banks are as close as possible to the existing ground surface to maximize flooding without requiring large floodplain excavation. The tradeoff is that in some areas the new channel flows through depressions that would require placement of fill to construct the channel.

Several results are common to all the profile plots. Construction of the new channel in each Alternative raises the water surface elevations for the 15 cfs to 1,600 cfs modeled flows higher than the existing condition. At the largest modeled 100-yr flow event of 7,650 cfs, the Alternatives water surface elevations are similar to the existing condition. In the upper portion of the project area, the 100-yr flood water surface elevations are increased by a few tenths of feet for Alternatives 2 and 3, and nearly identical for Alternative 4 compared to the existing condition. The water surface elevations show a noticeable drop in elevation as the new Sunset channels transition into the existing channel immediately upstream of the proposed CSLT restored airport reach. This is due to lower base level elevations and an increased channel capacity in existing channel. However, the anticipated construction of a smaller capacity channel at a higher base elevation in the CSLT airport reach, thus creating higher water surface backwater elevations, has the beneficial effect of lessening the drop in Sunset Stables water surface elevations. At the upstream end of the Sunset project, construction of new channel in Alternatives 2 and 3 creates appreciable backwater effects that increase water surface elevations up to 2.5 ft upstream of the Highway 50 bridge in the Golf Course reach for flows from 15 cfs to 1,650 cfs. The backwater effects would extend for approximately 2,500 ft upstream of Highway 50 (beyond the extent of the modeling reach). The 100-yr flood (7,650 cfs) water surface elevation shows no change between the Alternatives and the existing condition upstream of the Highway 50 bridge.

As desired, the design discharge (450 cfs) water surface elevations in new channel reaches for each of the Alternatives is even with the new top-of-bank elevations. This indicates that each Alternative would be successful at increasing the frequency at which floodplain inundation occurs to about a 1.4-year event.

For Alternative 4, four grade control structures were included in the model in the upstream half of the project reach. The grade control structures are spaced relatively evenly throughout the reach and located at existing riffle crest locations to take advantage of the existing high bed elevations at these locations. The grade controls all extend 1.5 ft above the average bed elevation at the riffle crests. This height was selected because it is tall enough to slow down flow velocities, thus forcing expected sedimentation to meet the Alternative's objective of aggrading the existing channel bed. Grade control structures constructed much taller than 1.5 ft begin to cause greater backwater effects, and pose greater problems for fish passage and barriers to recreational users floating the channel. The backwater effects created by the grade control structures are particularly apparent at the base flow (15 cfs) and 450 cfs flows. At 15 cfs, the grade control structures back water up for hundreds of feet upstream, essentially creating long reaches of deep, ponded water. As flows increase, the backwater effect of the structures diminishes. At 450 cfs, the backwater effects of the grade control structures are still apparent, yet the water surface elevations still remain at least a foot below the top-of-bank for most of the upper reach. At 760 cfs, the effect of the grade control structures on raising water surface elevations above the existing condition further diminishes and becomes negligible at the 1,600 cfs and 7,650 cfs flows.

Main Channel Velocity Profile Plots

Longitudinal channel velocity profile plots of the Alternatives versus the existing condition for the modeled flows are shown in Appendix F. Each plot has an

accompanying water surface and thalweg plot at the bottom to facilitate analysis. The velocity plots show mean main channel velocity values.

The sections of new channel in all the Alternatives meet the target mean channel velocity of about 3 ft/s at the 450 cfs design discharge (see Table 7). For all modeled flows, the Alternatives new channel velocities are consistently higher than the existing condition. This is largely due to the greater flow conveyance efficiency of relatively deep and quick moving water in the new channels (low width to depth ratio) compared to relatively shallow and slower water in the existing channel (high width to depth ratio). As expected, mean channel velocities decrease for the Altneratives in backwater areas at the transition into the CSLT airport reach, and upstream of the start of the new Sunset Stables channel at Highway 50. Spikes in channel velocities are also observed where the water surface elevation drops and the flow accelerates from the new Sunset Stables channel into the beginning of the relatively lower elevations in the existing channel between the new CSLT airport reach and Sunset Stables reach.

The grade control structures in Alternative 4 have dual effects on mean channel velocities. At the grade control structures themselves, flow velocities are substantially increased as flow is accelerated over the top of the structures. However, velocities decrease compared to the existing condition in the backwater upstream of the structures for the 15 cfs, 450 cfs, and 760 cfs flows. Channel velocities upstream of the Highway 50 bridge are essentially unchanged in Alternative 4 compared to the existing condition.

The values from these plots are applied in the critical grain diameter plots described below.

Floodplain Velocity Profile Plots

Longitudinal floodplain velocity profile plots of the Alternatives versus the existing condition for the modeled flood flows of 760 cfs, 1,600 cfs and 7,650 cfs are shown in Appendix G. Mean velocity values for the floodplain to the left and right of the channel are shown.

Because all of the Alternatives would create additional floodplain flow compared to the existing condition, the floodplain velocities are generally higher for the Alternatives compared to the existing condition. Past research has related the maximum permissible velocity before the onset of erosion across different types of ground surfaces. For the type of grassy meadow vegetation, soil conditions, and valley slopes found at Sunset Stables, the maximum permissible velocities are in the range of 6 to 7 feet per second (Table 8). Locations on the floodplain velocity profile plots where the flood velocity exceeds 6 to 7 ft/s indicate areas susceptible to floodplain erosion (permissible floodplain velocities would be lower at locations of bare earth). Even at the highest modeled flow of 7,650 cfs (100-year event), floodplain velocities remain below 4 ft/s for all Alternatives. Floodplain velocity spikes are evident for all Alternatives where floodplain flow at the downstream end of Sunset Stables is funneled from the relatively broad meadow into the confined CSLT airport reach. The grade control structures in Alternative 4 also cause localized increases in floodplain velocities when they force more of the total flow out onto the floodplain.

Table 8. Maximum Permissible Velocities for Selected Ground Covers (Source: USACE, 1994)

Channel Material	Mean Channel Velocity, fps
Fine Sand	2.0
Coarse Sand	4.0
Fine Gravel[1]	6.0
Earth	
Sandy Silt	2.0
Silt Clay	3.5
Clay	6.0
Grass-lined Earth (slopes less than 5%)[2]	
Bermuda Grass	
Sandy Silt	6.0
Silt Clay	8.0
Kentucky Blue Grass	
Sandy Silt	5.0
Silt Clay	7.0
Poor Rock (usually sedimentary)	10.0
Soft Sandstone	8.0
Soft Shale	3.5
Good Rock (usually igneous or hard metamorphic)	20.0

Notes:
1. For particles larger than fine gravel (about 20 millimetres (mm) = 3/4 in.), see Plates 29 and 30.
2. Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.

Main Channel Boundary Shear Stress Plots

Longitudinal channel shear stress profile plots of the Alternatives versus the existing condition for the modeled flows are shown in Appendix H. The plots show mean main channel total boundary shear stress values based on the hydraulic radius and energy grade modeled at each cross-section. The values from these plots are applied in the critical grain diameter plots described below.

Critical Grain Diameter Plots

Longitudinal critical grain diameter profile plots of the Alternatives versus the existing condition for the modeled 450 cfs design discharge are shown in Appendix I. The plots

show the largest grain size predicted to be mobilized by a 450 cfs flow. Note that two critical grain size values are shown at each cross-section. One value is based on the total boundary shear stress values described above. This represents the mean total shear stress acting on the entire perimeter of the channel due to the depth and slope of the channel. Research has shown that much of the flow's total boundary shear stress is not available for mobilizing sediment since flow energy is often lost at other channel features such as meander bends, large woody debris, and bedforms (Chanson, 1999). Because of this, the total boundary shear can overestimate the calculated critical grain size. Therefore, a drag partition equation (Wilcock, 2001) was used to determine the grain shear stress available at the bed to mobilize sediment. The smaller critical grain diameters predicted by using only the grain stress are likely better estimates of the grain sizes that would be mobilized by the design discharge. The predicted values in the new channel in the 5 mm to 10 mm (fine to medium gravel) grain size range using the grain stress are closer to the existing channel substrate and bedload supply than the higher values around 15 mm predicted using the total boundary shear. Also note that the critical grain diameter calculations are based on mean channel hydraulics. In reality, shear stress and velocity can vary substantially across the channel, resulting in localized areas with higher and lower transport potential. Acknowledging this, the plots in Appendix I are still useful for examining trends in the data and for comparing the Alternatives against the existing condition.

The modeling predicts critical grain diameters approximately 3 to 4 mm greater for each of the new channel reaches in the Alternatives compared to the existing condition. This is because channel velocity and shear are greater in the new channel with width to depth ratios lower than the existing condition. Conversely, in the backwater areas created by the CSLT airport reach and upstream of the Highway 50 bridge, the critical grain diameters are reduced compared to existing conditions. The modeling predicts a 450 cfs discharge would only be able to mobilize 1 to 2 mm sand in these backwater areas. These results indicate that sedimentation and textural fining of the bed substrate can be expected in these backwater areas. Spikes in the critical grain size (up to 60 mm, very coarse gravel, size material) occur where flow velocities accelerate as the new Sunset Stables channel transitions into the existing channel at the downstream end of the project reach.

At the grade control structures in Alternative 4, the critical grain diameters at the grade control crests are high as flow accelerates over the top of the structures. However, in the backwater upstream of each structure, the critical grain sizes are reduced compared to the existing condition. These results suggest that the grade controls will be effective at forcing sedimentation, thus resulting in bed aggradation and textural fining.

Channel Width to Channel Depth Ratio Plots

Longitudinal width to depth ratio (W/D) profile plots of the Alternatives versus the existing condition for the 450 cfs design discharge are shown in Appendix J. The plots show the width to depth ratio based on the wetted channel width and flow depth at 450 cfs. Historic channel deepening and widening of the existing channel has resulted in an increased W/D ratio, averaging 23 for existing Sunset Stables modeled cross-sections. Examination of these plots illustrates how channel widths and depths for each Alternative would vary between new channel reaches and transitions into existing channel. All new

channel in the Altneratives would have a width to depth ration around 10 at a 450 cfs flow.

Flood Inundation Maps

The flood inundation areas for each of the Alternatives at 450 cfs, 760 cfs, 1,600 cfs, and 7,650 cfs were modeled using the HEC-RAS hydraulic output and HEC-GeoRAS software (Appendix K). The maps are generated by importing the water surface elevations modeled with HEC-RAS at each cross-section into GIS software, then using the HEC-GeoRAS software utility, continuous water surfaces elevations for the entire project area are determined by interpolating between the modeled cross-sections. If the modeled water surface elevation is greater than the elevation of the LiDAR elevation surface, then the area is classified as inundated. Only floodplain areas with surface water connections to the main channel are included in inundation mapping (i.e., isolated floodplain depressions, such as channel scars that have ground elevations lower than the water surface elevation, are not included since they have high ground disconnecting them with the channel).

Each map shows the predicted Alternative inundation area as a blue polygon. The existing condition flood inundation area is shown for comparison as a white hatched polygon. The maps also show the channel centerline alignments of the Alternatives (yellow polylines) and existing condition (white River Station circles). Area of inundation results are summarized in Table 9 for two flood flows of 760 cfs (2-yr event) and 1,600 cfs (5-yr event).

The data show that Alternatives 2 and 3, which contain almost all new channel constructed at grade with the existing floodplain, have the greatest gains in floodplain inundation compared to the existing channel. Flooding is enhanced throughout the project area wherever new channel is constructed. Much of the gain in floodplain inundation for Alternative 4 at a 760 cfs event is in the open meadow downstream of RS 21200, where the new channel begins. As is also evident in the water surface profiles (Appendix E), the four grade control structures in the upstream half of the existing channel have little affect on floodplain inundation. For all Alternatives, floodplain inundation largely ends at around RS 15+300. In the deep and wide existing channel reach that would remain between the new proposed channels in Sunset Stables and the CSLT airport reach, water surface elevations would remain relatively low. This has the effect of drawing down the water surface elevations at the downstream end of Sunset Stables, thus reducing floodplain inundation. It is also apparent that the increased backwater elevations caused by the new channels in Alternatives 2 and 3 increase floodplain inundation in the golf course reach upstream of the project.

The increases in floodplain inundation at the 1,600 cfs flow are comparatively less than the 760 cfs flow. The 5-yr event, 1,600 cfs flow is of high enough magnitude that even the existing channel is overbanking throughout the project area. Although the water surface elevations of the Alternatives are higher than the existing condition, the rate of increases in floodplain inundation decreases with increased flow since it takes a relatively large increase in water surface elevation to increase the extent of areal flooding on such a flat and broad floodplain surface. For all Alternatives, floodplain flow continues to be

constricted somewhat where the southern end of the airport cuts into the floodplain (RS 22+000).

At the 100-yr, 7,650 cfs flow, the inundation area of the Alternatives is nearly identical to the existing condition since the potential for increased inundation area is restricted by the steep valley walls.

Table 9. HEC-GeoRAS calculated flood inundation area results for 760 cfs and 1,600 cfs.

Alternative	Discharge (cfs)	Area (Acres)	Percent Increase Over Existing
Existing Condition	760	58	NA
Alternative 2	760	131	127%
Alternative 3	760	129	124%
Alternative 4	760	99	71%
Existing Condition	1,600	132	NA
Alternative 2	1,600	174	31%
Alternative 3	1,600	173	31%
Alternative 4	1,600	140	6%

Section 7 - Conclusions

The objectives of this report are to: 1) describe the process used to design a new channel for the Sunset Stables restoration project, and 2) present the approach and results of the Alternative channel designs hydraulic modeling. The geomorphically based design of the new channel is based primarily on principles of stream hydrology and sediment transport. The new channel is designed to have a geomorphic form appropriate for the environmental setting of the Sunset Stables project area, and to be sustainable by having a size and slope that is compatible with the prevailing hydrology and sediment load regimes.

Using the most recent hydrology and sediment transport data, flood recurrence and effective discharge calculations were performed. The results indicate that a 450 cfs discharge is an appropriate value to use as the design discharge for sizing the new channel since its 1.4-year return interval is near the commonly used 1.5-year bankfull return interval, and also near the calculated effective discharge of 400 cfs. After determining the new channel's design discharge, the equilibrium slope method was used to calculate the channel size needed to transport the bedload supply without causing channel degradation of aggradation. First, multiple hydraulic geometry relationships were analyzed to determine an average width for the new channel that is suitable with the cohesive properties of the existing Sunset Stables floodplain soils that will form the new channel banks. Then, based on the specified new channel width, a series of calculations were performed to determine the new channel's average slope and depth needed to transport the bedload supply at the 450 cfs design flow. From these calculations, the channel dimensions for a typical riffle cross-section were determined. These are the

channel dimensions that were used in the HEC-RAS modeling analysis of the restoration Alternatives. Once the preferred Alternative is selected, the preliminary symmetrical channel dimensions will be developed into asymmetrical dimensions to represent the channel geometries of non-riffle locations (e.g., pools).

The new Sunset Stables channel would have approximately the same channel slope and length as the existing channel, but it would be narrower and deeper. For average values at the 450 cfs design discharge, the new channel width would decrease from 65 ft to 39 ft and channel hydraulic depth would increase from 2.8 ft to 3.9 ft, resulting in a width to depth ratio decrease from 23 to 10. Compared to the existing channel, the narrower and deeper new channel would be more hydraulically efficient with an increase in mean channel velocity from 2.5 ft/s to 3.1 ft/s. Consequently, coarse sediment in the new channel bed would be mobilized more often. Furthermore, the new channel would start inundating the floodplain at 450 cfs (1.4 year event), instead of 700 cfs to 1,300 cfs in the existing channel (2 to 5 year event).

Three different HEC-RAS models were created to analyze the hydraulics of the new Alternatives channel designs. Streamflows ranging from a 15 cfs baseflow condition up to the 100-yr flood estimate (7,650 cfs) were modeled to evaluate the performance of both new channel reaches and adjacent existing channel reaches. The model results show that at this conceptual level, all three Alternatives achieve their intended objectives. The models for Alternatives 2 and 3 both have mostly new channel reaches with new top-of-bank elevations close to the existing floodplain surface. The new channel reaches are sized to reach bankfull at 450 cfs and provide substantial gains in floodplain inundation for a given flow compared to the existing condition. The modeled channel velocities for the new channel reaches match the target velocities of the design, and floodplain velocities are not altered substantially enough to cause floodplain erosion.

Two areas in Alternatives 2 and 3 will require additional attention if either Alternative is selected as the preferred Alternative and advanced in design. First, additional design detail will be required where the new channel flows into the existing channel at the downstream end of the project area. The hydraulic transition of in-channel flow going from a relatively higher elevation in the new channel into a lower elevation in the existing channel must be addressed. Additionally, the constriction of floodplain flow as it converges from the broad meadow adjacent to the new channel into the narrow downstream end of the project reach and into the CSLT airport reach must be analyzed in greater detail. At the other end of the project reach, the higher bed elevation of the new channel in Alternatives 2 and 3 creates backwater effects for hundreds of feet upstream of the Highway 50 bridge and into the CA State Parks golf course reach. This slow moving backwater will likely trap bedload and lead to aggradation of the existing channel upstream of the new channel. Aggradation of the existing channel upstream of the Highway 50 bridge may be beneficial for reducing channel capacity in the deeply incised reach. However, the affects of aggradation on flow conveyance capacity through the Highway 50 bridge must be analyzed. In addition, if much of the bedload supply for the new channel is trapped in the backwater upstream of the bridge, additional work must be done to develop a plan that ensures a source of bedload is available to the new channel to limit the potential for degradation of the new channel reach.

The downstream half of Alternative 4 performs similarly to the downstream half of Alternative 3. It is in the upstream half of Alternative 4, where the grade control structures would be constructed, that Alternative 4 differs. The model results show that the grade control structures are effective at creating backwater and slower channel velocities. It is expected that the grade controls will promote the intended channel aggradation. At this level of design, it is not known how long it may take for the bed to aggrade to the desired level in the grade control reach. Additional modeling, such as use of a sediment routing model, would be required to better understand the timeframe for the grade controls to trap enough sediment and aggrade the bed. The HEC-RAS model results do not assume any channel aggradation has occurred (i.e., the bed elevations in the model are the same as the existing condition upstream of the grade controls). Under this condition, the grade control structures create noticeable backwater effects, particularly at the 15 cfs flow. Yet, as flow increases, the ability of the grade controls to increase water surface elevations decreases. Therefore, the grade control structures alone are not enough to increase flooding in the upstream reach. Reduction of channel capacity through future channel aggradation will be required for this to occur. If Alternative 4 is selected as the preferred Alternative and advanced in design, additional work will be required, such as: 1) determining how long the grade control structures will take to aggrade, 2) analyzing how trapping of sediment in the upstream reach affects channel stability of the new channel downstream, and 3) assessing the impact of the grade controls on fish passage and aquatic habitat.

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APPENDIX A

APPENDIX B

APPENDIX C

APPENDIX D

APPENDIX E

APPENDIX F

APPENDIX G

APPENDIX H

APPENDIX I

APPENDIX J

APPENDIX K