Stream, Wetland, and Mountain Meadow Services
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December 5, 2018

Michele Prestowitz
Program Manager
Truckee River Watershed Council

RE: Donner Creek Sites 2-4 Admin. Draft 100% Design Submittal

The enclosed Admin. 100% Design Plans for Donner Creek Site 2-4 Restoration are a result of 2017 field investigations, engineering analysis, client and stakeholder input, discussions during Conceptual Design and Draft Advanced Conceptual Design meetings held in August and September 2017, and discussions during the 90% Design meeting held in October 2018. The designs as presented are intended to restore and stabilize riparian and wetland areas, provide water quality treatment and protection, and enhance geomorphic function and riparian habitat.

The following summarizes the design approach and methodology. A summary of the hydraulic modeling results is included.

Next steps are to reach out to Site 4 private property owners to confirm permission to perform Site 4 work on private property can be obtained, and to review and revise these 100% Admin. Design Plans to generate final stamped plans for inclusion in the project construction bid.

Please don't hesitate to contact me if you have any questions.

Regards,

Carol Beahan, PE
Project Manager

CC: Lisa Wallace, TRWC

Design Approach

Site 2:
Shear forces on the west bank resulted in accelerated bank erosion and undermining during relatively high flow events. We incorporated bendway weirs as part of the bank stabilization repairs in order to redirect high-velocity flows towards the channel centerline and away from the bank, resulting in reduced shear stresses and reduced likelihood of bank erosion. To aid in this "pressure relief" of the west bank, the plans direct removal of a large debris pile and reworking/lowering the east gravel/sand bar to reactivate existing secondary overflow channels and increase the available floodplain. All existing utilities, particularly the sanitary sewer line to the east of the channel have been identified and the designs provide sufficient clearance to avoid them.

Site 3:
The primary goal of the wetland restoration design portion of the project is to improve Donner Creek and Truckee River water quality by improving stormwater treatment functional values and habitat diversity. The approach for Site 3 is to work with existing stormwater infrastructure and access shallow groundwater through a general decrease in bottom elevation of the existing habitat. This will increase storage/pollutant trapping and improve water quality and diversity of habitat. Willow and riparian plantings will be incorporated for habitat and highway screening to support use of this area by wildlife.

Additional water quality improvements will be achieved through construction of a grass-lined swale with rock check dams for pretreatment of freeway runoff prior to discharge to the wetland complex.

Site 4:
Working within the confines of the highway and privately-owned parcels, we continued the approach introduced in the prior watershed assessment (cbec 2016), proposing log bendway weir installations on alternating banks to encourage natural development of a "meander" in this straightened section of Donner Creek. The result will be improved aquatic habitat during low flow conditions. Revegetation with willow stakes and native grass species will stabilize disturbed banks in the areas surrounding the rootwad installations.

Similar to Site 3, water quality improvements will be achieved through construction of a grass-lined swale with rock check dams for pretreatment of freeway runoff prior to discharge to the creek.

Design Methodology

Site 2:
Rock slope protection is proposed to armor the west bank and protect vulnerable infrastructure. Riparian plantings, willow poles, and native grasses are to be integrated with the rock to enhance habitat and provide shade. The existing biotechnical stabilization was insufficient to provide reliable protection against the erosive forces on the bank. Performance of existing stabilization features was likely worsened by the hydraulic "ping-pong" effect from when flows emanate from the 90-degree bend immediately downstream of the Highway 89 crossing and upstream from the project area.

To mitigate hydraulic shear forces on the west bank, five boulder bendway weirs are proposed at a spacing of 40 to 50 feet along the proposed repair. The proposed spacing, angle, and geometry of the bendway weirs were selected based on existing channel geometry, results from hydraulic modeling, and technical guidance documents (CDOT 2004, NRCS 2007).

Adjustment of the east gravel bar by excavating one to two feet at strategic locations is intended to restore the east floodplain and reactivate secondary overflow channels during 1- to 2-year storm events. This should provide additional mitigation of hydraulic shear forces on the west bank.

A maximum restored bank slope of 1.5:1 is specified for the west bank to ensure long-term bank stability while minimizing cut/fill and working within the spatial limitations posed by existing infrastructure. Survey data confirms that site conditions (bank height, horizontal distance between top of bank and toe of bank) allow restoration of the bank to 1.5:1 slope and more mild (2:1) in most locations.

Based on design review discussion, the design was revised to withstand predicted velocities and associated shear forces during a 100-year design event. The revised design incorporates:

- 1-ton anchor boulders along the center of the Site 2 boulder bendway weirs transitioning to 1/2-ton then 1/4-ton boulders to build up the bendway weirs,
- 1/2-ton keyed boulders at the toe of the restored Site 2 bank with 1/2-ton boulder continuing up bank and transitioning to 1/4-ton boulders extending to just above the predicted 100-year water-surface elevation, and
- 1/2-ton to 1-ton boulder ballasts to secure the Site 4 rootwad bendway weirs.

These boulders will be keyed into the bank and filled in with native fill material (cut from the east gravel bar) and layered with willow, wood's rose, honeysuckle, alder, and dogwood to enhance habitat and provide added stability (prevent soil erosion in pore space between rocks and possibility of bank sloughing). The top of bank will be seeded with native and fast-growing grass species.

The revised boulder sizing is based on the predicted velocities and associated shear forces during a 100-year design event (Table 1), Caltrans RSP equations (Caltrans 2010), and the permissible shear and velocity values for various RSP boulder sizes shown in Table 2. As shown in Table 1, the maximum predicted 100-year design velocity through Sites 2 and 4 is approximately 10.3 feet per second (fps) and the maximum shear stress is approximately 1.6 pounds per square foot (psf). Per Table 2, the minimum required RSP boulder size to ensure these maximum predicted values are within the permissible limits is 18-inch d_{50}, which most nearly equates to 1/4-ton boulders per Table 3 below. Accordingly, the minimum boulder size to be placed below the predicted 100-year water surface elevation is 1/4-ton class RSP. The inclusion of larger boulder sizes serves to support the upper bank 1/4-ton boulders, optimize buildability, and integrate an added factor of safety to ensure longevity of the proposed improvements.

Construction will require a strategic approach to minimize temporary water quality impacts and avoid equipment operations within an active section of the creek. Potential strategies include (1) work from Caltrans easement above the west bank, with movement of east gravel bar material to the west bank via a long-reach excavator swinging from the west edge of the gravel bar and turbidity curtains and/or silt fence installed along the west edge of the east work area and east edge of the west work area, (2) construction of a temporary crossing to enable equipment travel from the east work area to the west work area with similar temporary erosion control features as described for option 1, or (3) pipeline diversion through the project reach to reduce access restrictions and associated challenges and reduce need for temporary erosion control features. The pipeline diversion is the proposed method and is described in greater detail in the "Diversion Design" section below.

Table 1. HEC-RAS Hydraulic Modeling Summary Data

Site No.	River Station	Existing Conditions Water Surface Elevation[1] (NAVD88)	Proposed Conditions Water Surface Elevation[1] (NAVD88)	Change in Water Surface Elevation (ft)	Existing Conditions Velocity[2] (fps)	Proposed Conditions Velocity[2] (fps)	Existing Conditions Shear Stress[3] (psf)	Proposed Conditions Shear Stress[3] (psf)
2	2232.4 (upper boundary)	5876.8	5877.3	0.5	9.05	8.36	1.37	1.15
	2091.1 (middle)	5876.9	5876.9	0	6.74	6.91	0.73	0.77
	1957.9 (lower boundary)	5876.8	5876.7	-0.1	6.21	6.33	0.56	0.59
4	6244.7 (upper boundary)	5899.6	5899.8	0.2	9.14	10.28	1.35	1.58
	5076.5 (middle)	5893.4	5893.5	0.1	9.36	9.73	1.46	1.60
	4020.2 (lower boundary)	5890.2	5890.3	-0.1	7.11	7.05	0.80	0.79

1. Values for Water Surface Elevation shown from HEC-RAS Model outputs for 100-year flowrate (3,385 cfs)
2. Values for Velocity shown from HEC-RAS Model outputs for 100-year flowrate (3,385 cfs)
3. Values for Shear Stress shown from HEC-RAS Model outputs for 100-year flowrate (3,385 cfs).

Table 2. Caltrans Rip Rap Size

RSP Class	D_{50} Size[1]	D_{50} Weight
	inches	pounds
8 Ton	71	17600
4 Ton	56	8800
2 Ton	45	4400
1 Ton	36	2200
1/2 Ton	28	1100
1/4 Ton	23	550
Light	16	200
Facing	12	75
Backing No 1	12	75
Backing No 2	8	25
Backing No 3	4 2/3	5
Small RSP (7-inch)	3	1 1/3
Small RSP (5-inch)	2	2/5
Small RSP (4-inch)	1	1/20

[1]Assumes rock density = 165 lb/ft^3

(Source: Caltrans Standard Specifications 2015)

Table 3. Permissible Velocities and Shears for Various Materials

Boundary Category	Boundary Type	Permissible Shear Stress (lb/sq ft)	Permissible Velocity (ft/sec)	Citation(s)
Soils	Fine colloidal sand	0.02 - 0.03	1.5	A
	Sandy loam (noncolloidal)	0.03 - 0.04	1.75	A
	Alluvial silt (noncolloidal)	0.045 - 0.05	2	A
	Silty loam (noncolloidal)	0.045 - 0.05	1.75 – 2.25	A
	Firm loam	0.075	2.5	A
	Fine gravels	0.075	2.5	A
	Stiff clay	0.26	3 – 4.5	A, F
	Alluvial silt (colloidal)	0.26	3.75	A
	Graded loam to cobbles	0.38	3.75	A
	Graded silts to cobbles	0.43	4	A
	Shales and hardpan	0.67	6	A
Gravel/Cobble	1-in.	0.33	2.5 – 5	A
	2-in.	0.67	3 – 6	A
	6-in.	2.0	4 – 7.5	A
	12-in.	4.0	5.5 – 12	A
Vegetation	Class A turf	3.7	6 – 8	E, N
	Class B turf	2.1	4 - 7	E, N
	Class C turf	1.0	3.5	E, N
	Long native grasses	1.2 – 1.7	4 – 6	G, H, L, N
	Short native and bunch grass	0.7 - 0.95	3 – 4	G, H, L, N
	Reed plantings	0.1-0.6	N/A	E, N
	Hardwood tree plantings	0.41-2.5	N/A	E, N
Temporary Degradable RECPs	Jute net	0.45	1 – 2.5	E, H, M
	Straw with net	1.5 – 1.65	1 – 3	E, H, M
	Coconut fiber with net	2.25	3 – 4	E, M
	Fiberglass roving	2.00	2.5 – 7	E, H, M
Non-Degradable RECPs	Unvegetated	3.00	5 – 7	E, G, M
	Partially established	4.0-6.0	7.5 – 15	E, G, M
	Fully vegetated	8.00	8 – 21	F, L, M
Riprap	6 – in. d_{50}	2.5	5 – 10	H
	9 – in. d_{50}	3.8	7 – 11	H
	12 – in. d_{50}	5.1	10 – 13	H
	18 – in. d_{50}	7.6	12 – 16	H
	24 – in. d_{50}	10.1	14 – 18	E
Soil Bioengineering	Wattles	0.2 – 1.0	3	C, I, J, N
	Reed fascine	0.6-1.25	5	E
	Coir roll	3 - 5	8	E, M, N
	Vegetated coir mat	4 - 8	9.5	E, M, N
	Live brush mattress (initial)	0.4 – 4.1	4	B, E, I
	Live brush mattress (grown)	3.90-8.2	12	B, C, E, I, N
	Brush layering (initial/grown)	0.4 – 6.25	12	E, I, N
	Live fascine	1.25-3.10	6 – 8	C, E, I, J
	Live willow stakes	2.10-3.10	3 – 10	E, N, O
Hard Surfacing	Gabions	10	14 – 19	D
	Concrete	12.5	>18	H

[1] Ranges of values generally reflect multiple sources of data or different testing conditions.

A. Chang, H.H. (1988).
B. Florineth. (1982)
C. Gerstgraser, C. (1998).
D. Goff, K. (1999).
E. Gray, D.H., and Sotir, R.B. (1996).
F. Julien, P.Y. (1995).
G. Kouwen, N.; Li, R. M.; and Simons, D.B., (1980).
H. Norman, J. N. (1975).
I. Schiechtl, H. M. and R. Stern. (1996).
J. Schoklitsch, A. (1937).
K. Sprague, C.J. (1999).
L. Temple, D.M. (1980).
M. TXDOT (1999)
N. Data from Author (2001)
O. USACE (1997).

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(Source: Fischenich 2001)

Site 3:
The proposed design includes site-specific excavation at a depth ranging from 2 to 4 feet to expand and enhance wet meadow, emergent marsh, and riparian habitat while also improving water quality of the system by intercepting the shallow groundwater table that is connected hydraulically to Donner Creek. Historical creek alignment, knowledge of fill placement for highway construction/construction in the vicinity of the high school, and elevation data from field surveys informed the proposed excavation depths.

Vegetation specifications included native riparian tree and shrub planting, salvage and replacement of wet meadow and emergent marsh plants, and broadcast seeding of native grasses to revegetate access roads and other disturbed areas.

Design information on the proposed grass-lined swale is described in the "Swale Design" section below.

The combination of increased storage capacity with no proposed changes to inlets, outlets, or upstream conditions will improve attenuation of peak discharges downstream from the site. An extensive hydrologic modeling effort will be required to quantify the effect and it is likely that the effect is insignificant for low-frequency flood events (i.e. 100-year to 500-year floods). It is recommended that bore hole dilution tests in the vicinity of the excavated wetland feature be done prior to construction in order to more accurately define the groundwater interactions in this area and corroborate functionality and sustainability of the design as proposed.

To address any potential for haul trucks to degrade the school access road we suggest the bid package and plans include a note requiring pre-construction and post-construction conditions of TTUSD's pavement along the haul route be video documented by a third party and restoration to pre-construction conditions shall be the responsibility of the Contractor.

Site 4:
Nine rootwad bendway weirs are proposed on alternating banks at a longitudinal spacing of 260 to 320 feet. The proposed spacing, angle, and installation specifications were selected based on general guidance (NRCS 2007, FISRWG 2001, WSAH, 2012) and prior project experience with similar installations to encourage the natural development of the meander process in this highly straightened reach under low flows. A hydraulic model was used to confirm that these installations will exert little influence on creek hydraulics under relatively rare flows, showing little to no increase in water-surface elevations as described in the Hydraulic Modeling section below.

Two temporary staging areas and access ramps are proposed from the north side of the creek (freeway side), one at the east end of the project and one at the west end. The proposed locations have adequate space for safe access from the freeway and will reduce equipment travel distance/time for various installations. The west staging area is the primary staging area, with sufficient space for equipment and material staging.

Diversion Design:
Site 2 requires equipment operations along the west bank, including filling the scoured toe, rebuilding the bank out over the scoured toe, and installing boulder bendway weirs out into the active Donner Creek channel.

Accordingly, a pipeline creek diversion is proposed for Site 2. Similarly, Site 4 requires equipment work along the toe of the north and south banks through the Site 4 project reach. A pipeline diversion through the project reach is proposed to simplify the logistics of construction and ensure water quality protection.

The proposed diversions for Sites 2 and 4 include three 36-inch diameter diversion pipes, sand bag/visqueen coffer dams at the upstream project boundary to pond water at the pipeline inlets and prevent water from flowing through the work areas, fish screens upstream of the inlets to prevent fish passage through the diversions, and rock energy dissipation at the pipeline outlets. The diversion pipe alignment for both sites is approximately along the creek thalweg, such that the diversions can flow by gravity. Both diversions require one or more crossing locations for equipment travel over the diversion pipes. Native onsite gravel supplemented with clean gravel purchased from a local quarry is proposed for use in constructing the temporary crossings. The gravel is to be built up around and over the pipes in order to construct crossings of sufficient width, length (per slope requirements), and depth (minimum 12" cover over the pipes). If the pipe material type used cannot withstand the load of the equipment, the diversion pipes will be run through heavy duty CMP culverts for the length of the crossing to protect the pipes.

The diversion pipe capacities were developed to meet the 100 cfs maximum allowable flowrate in Donner Creek prior to start of construction as required by the design plans (see General Notes on Sheet G-1 of the Project Plans). Flowrates in Donner Creek typically drop below 100 cfs by mid-July and continue to decrease through the summer until Donner Lake dam releases begin in early September (see Figure 1 below). For the maximum expected flowrate of 100 cfs through the diversion pipes (will be significantly lower through the majority of construction, but may increase to near 100 cfs in the event of a large rain storm), the proposed pipe diversion configuration would yield flow velocities of 4.7 fps and friction head losses of 0.5 feet and 3.9 feet for Sites 2 and 4, respectively (per Hazen-Williams equation calculations assuming pipe-full conditions, HDPE pipe roughness coefficient of 140, Site 2 diversion length of 300 feet, and Site 4 diversion length of 2400 feet).

The calculated pipe-full velocity of 4.72 fps is an acceptable velocity (within the range of existing conditions velocities through the project reaches). The friction head loss is compared to the elevation head (elevation difference between inlets and outlets of the proposed diversions) to confirm gravity flow is feasible. For Site 2, the friction head loss of 0.5 feet is sufficiently less than the elevation head of 2.2 feet (5868.6 approximate elevation at inlet and 5866.4 approximate elevation at outlet). For Site 4, the friction head loss of 3.9 feet is sufficiently less than the elevation head of 10.0 feet (5894 approximate elevation at inlet and 5884 approximate elevation at outlet).

Additional calculations were performed using Manning's equation, Sites 2 and 4 average channel slopes (0.73% and 0.41%, respectively) and Manning's roughness coefficient value of 0.012 (HDPE pipe), and it was determined that the maximum flow conveyance for the proposed Sites 2 and 4 diversions will be approximately 140 cfs and 185 cfs, respectively.

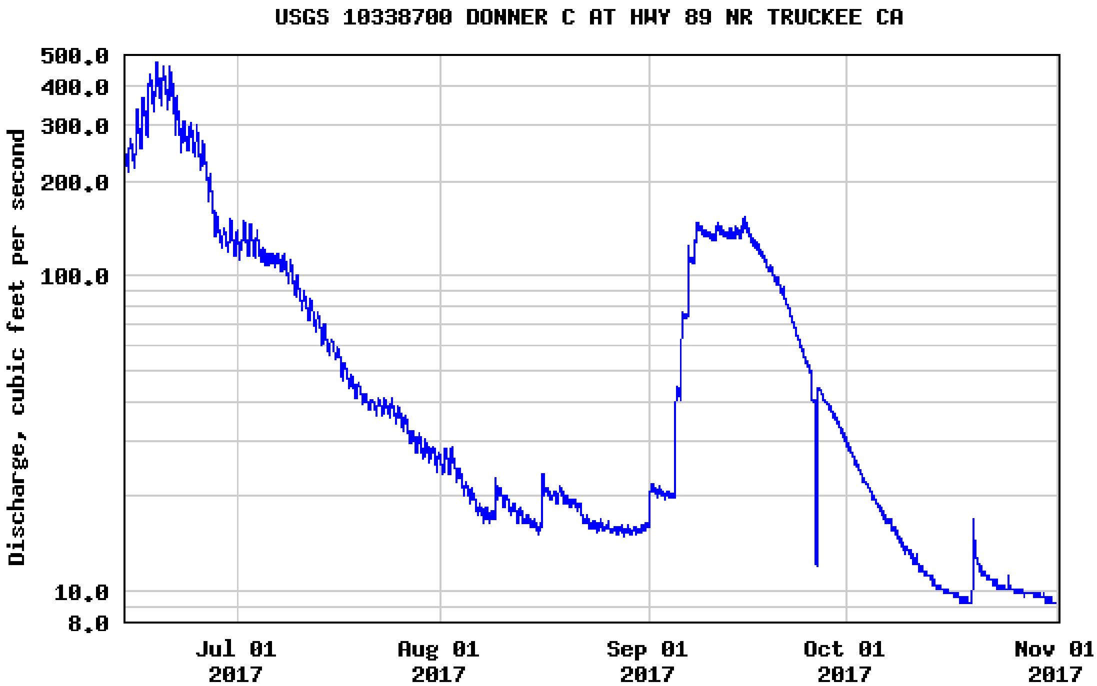


Figure 1. USGS Gage Station Flowrate Data Summer 2017
(Source: https://waterdata.usgs.gov/nwis/uv/?site_no=10338700)

Swale Design:
Interstate 80 eastbound adjacent to Site 4 has a cross slope toward Donner Creek. Opposite Site 4, Interstate 80 westbound has a cross slope toward the Site 3 stormwater complex. There is a vegetated filter strip on both sides. However, the presence of rilling and gullying and accumulated trash and sediment adjacent to the freeway shoulders and along the banks (observed during field reconnaissance and surveying) prompted the design team to explore solutions for better controlling runoff and reducing pollutant transport from the freeway in the project area. Rock-lined infiltration ditches were initially proposed, but were rejected due to Caltrans maintenance concerns. Grass-lined swales with rock check dams were proposed as a low-maintenance alternative. Figures 2 through 5 below highlight the existing conditions along the Caltrans ROW immediately north of Site 4 and south of Site 3.

Figure 2. Site 4 rilling from freeway runoff, looking west; grass-lined swale with check dams proposed along south side (left) of fence.

Figure 3. Site 4 location of proposed grass-lined swale with check dams (along south/right side of fence), looking east from approximate west edge of proposed swale.

Figure 4. Site 4, existing drainage inlets along freeway near location of proposed swale outlet (low point of freeway/shoulder grade in project vicinity); drainage outlets to remain in place with grass-lined swale as secondary/redundant filtration system; notice erosion from runoff bypassing drainage inlets.

Figures 5 and 6. Site 3 drainage looking westward. Gullies forming along shoulder drainage.

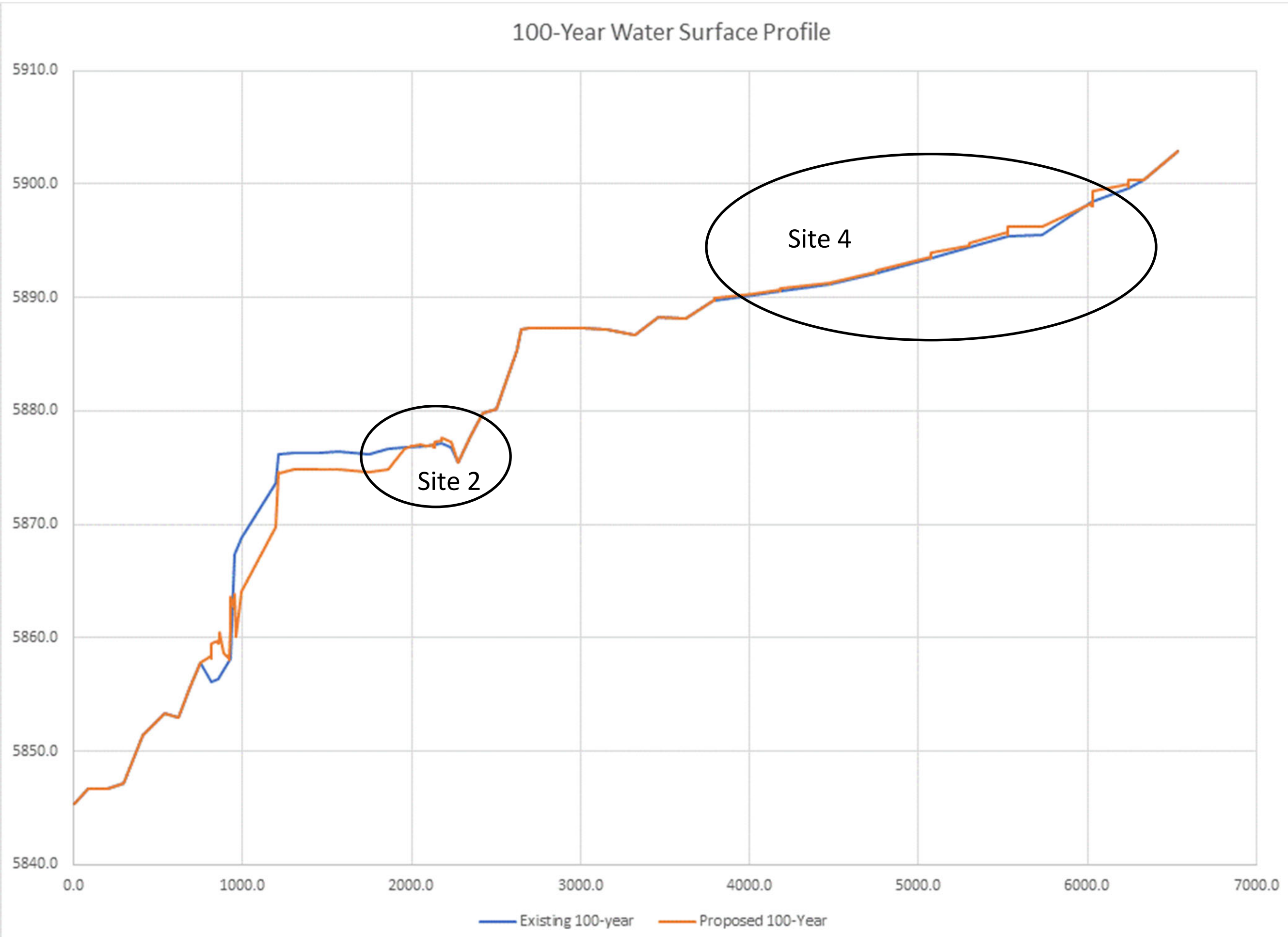
The proposed swales are a hybrid swale/infiltration trench/check dam design which will act as multi-functional stormwater treatment systems. They will have approximately 0.6 to 0.8% longitudinal slope (equal to the longitudinal freeway grade) toward a single outlet and will have rock check dams spaced at approximately 100 feet. The low slope will yield low velocity flow, causing suspended solids to settle and encouraging infiltration during small rain events. The rock check dams will further slow flows and capture sediment and trash. The swales will be small and low-profile (3 feet wide for Site 3, and 4 feet wide for Site 4 by 6 to 9 inches tall), such that they fit within the available space adjacent to the existing freeway shoulder and don't pose added danger to drivers or maintenance workers or create any challenges related to snow removal. They are designed such that any runoff ponding that may occur as a result of sediment/trash accumulation on the structures will overflow to the existing vegetated filter strip (biofiltration strip) and will not cause backwater onto the freeway. They will require minimal maintenance (annual or biannual removal of trash and sediment, possibly occasional reseeding).

Proposing grass-lined swales for this application is consistent with the Caltrans Project Planning and Design Guide), which states that swales should be considered wherever site conditions and climate allow vegetation to be established and where flow velocities will not cause scour. Although these swales are a hybrid design, the maximum potential velocity and associated shear are checked against permissible values to ensure erosion would not occur. Attachment 1 provides a summary of the calculations done for Sites 3 and 4 vegetated swales per the Caltrans Design Manual.

The proposed treatment/conveyance system is not a certified trash full capture system (per https://www.waterboards.ca.gov/water_issues/programs/stormwater/docs/trash_implementation/fcs_list_of_mbts_04aug17.pdf), but will serve as a trash capture system (trash caught up in swales and on rock check dams). Additional trash capture can be achieved at Site 3 through re-installing the existing grate covers at the culvert outlet from the stormwater complex. For additional trash capture capacity, a certified trash treatment control device could be installed at the swale outlets (per https://www.waterboards.ca.gov/water_issues/programs/stormwater/docs/trash_implementation/a1_certified_fcd_rev04aug17.pdf).

Hydraulic Modeling
The USACE's Hydraulic Engineering Center River Analysis System (HEC-RAS) Version 5.0.3 (USACE, 2016) was used to develop Existing Conditions and Proposed Conditions hydraulic models. The models were developed using field survey data, field measurements, and 2014 Tahoe National Forest LiDAR data. Results from model operation were used to inform boulder size and rootwad anchoring requirements, to assess the hydraulic effects of the proposed Site 2 bank repair and reworked gravel bar, and Site 4 rootwad installations and confirm that there would be no impacts to surrounding infrastructure due to any potential rise in water surface elevations post construction (Figure 7).

Figure 7. 100-Year Water-Surface Profiles of Donner Creek per HEC RAS 1-Dimensional Model



The design discharges for Donner Creek were derived by fitting a log-normal distribution to the annual peak series data collected by USGS at the SR89 stream gaging station. Quantiles from the fitted distribution were compared to published FEMA discharges and compared reasonably well. All modeling data and digital files are included on the submitted flash drive.

The most pertinent data (existing and proposed condition 100-year water-surface elevations and velocities and shear stresses at the approximate upstream boundary, center, and downstream boundary of the project locations) are shown in Table 1 above. As the data portrays, the proposed work would not worsen flooding, with water-surface elevations essentially the same or lower with the exception of a 0.5-foot increase at the upper boundary of Site 2 where there is still at least four feet of freeboard between the 100-year water-surface elevation and the toe of the existing concrete sidewalk. Velocities and shear stresses at Sites 2 and 4 stay relatively the same with a slight increase on the mid to lower portion of Site 2 reach and a slight increase on the upper to mid portion of Site 4 reach. The biggest increase in water-surface elevation under a 100-year event at Site 4 is only 0.2 feet which is still well below the top of bank. The constriction of the channel section by placing the root wads increases energy loss at those locations resulting in a very slight increase in stage upstream from each structure however there is adequate freeboard to absorb this slight increase without substantial impact on floodplain boundaries. The 100- flowrates and associated model-predicted data (i.e. shear stress) were the primary basis of the design.

Sediment Transport at Site 2

Given the observed aggradation of the gravel bar at Site 2, adequate sediment transport through this particular reach is a concern. It is recommended prior to construction that bed sediment samples be collected at several locations along the Site 2 reach and laboratory tests for particle-size distribution, particle specific gravity, and particle angularity are performed. These data would then be used in a numerical transport model to assess movement of bed sediments through the study reach. It would also be beneficial to collect bedload and suspended load transport during runoff events so that the sediment supply to this particular reach could be more accurately assessed.

For now, an incipient motion analysis was performed for a select cross-section of Site 2 to gain insight on what particles could be moved based on the return period of flows used in the HEC-RAS modeling analysis. A Julien adaptation of the Shields diagram was used to estimate the median particle size subject to incipient motion for the range of flowrates (RP 1.01 to 100 years) was used for our design. Results for RS 2177.2 and RS 2134 are presented in the following table. These are examples and do not constitute a complete sediment transport analysis, which would require data not currently collected and additional numerical modeling efforts.

Table 4. Site 2 Interim Incipient Motion Results

	River Station 2177.2			River Station 2134		
Recurrence Interval	**Discharge (cfs)**	**d50 (mm)**	**d50 (inches)**	**Discharge (cfs)**	**d50 (mm)**	**d50 (inches)**
100	3385	22.2	0.87	3385	64.1	2.52
50	2798	21.4	0.84	2798	54.3	2.14
20	2103	22.1	0.87	2103	53.3	2.10
10	1631	21.9	0.86	1631	57.4	2.26
5	1200	23.2	0.91	1200	55.6	2.19
2	666	21.1	0.83	666	45.4	1.79
1.25	370	14.7	0.58	370	25.4	1.00
1.11	272	11.0	0.43	272	17.5	0.69
1.05	211	9.1	0.36	211	13.4	0.53
1.01	131	6.5	0.26	131	8.0	0.31

In the table, d_{50} refers to the median particle size (in millimeters) that would be subject to incipient motion for the discharge and cross section hydraulic properties associated with that discharge. For example, for the 100-year discharge (3,385 cfs), particles less than or equal to 0.87 inches (22.2 mm) would be subject to transport by shear stresses from the flow at this cross section (RS 2177.2). Similarly, for the 1.01-year event (131 cfs), particles with a diameter of 0.26 inches (6.5 mm) or less would be subject to transport.

For RS 2134, particles up to 2.5 inches or 64.1 mm will be subject to transport during a 100-year (3,385 cfs) event. For a more common event (the 1.01-year event at 131 cfs), particles of diameter 0.31 inches or 8 mm or less would be subject to transport. Given the small size of particles being moved through this reach under existing conditions, a more technically-detailed analysis during the final design state is highly recommended.

Quantities

Tables 5 and 6 summarize the earthwork quantities and areas of disturbance and enhancement that will be key information for the permit applications. Material quantities, including boulders and rock will be incorporated into the Engineer's Estimate for bidding purposes.

Table 5. Habitat Enhancement and by Type

	PSSC (Riparian/Scrub Shrub Seasonally Flooded)	**PFO/SSJ** (Riparian Forest/Scrub Shrub Intermittently Flooded)	**PEMC** (Emergent Marsh/Wet Meadow Seasonally Flooded)	**PEMB** (Emergent Marsh/Wet Meadow Saturated)	**PUSCx** (Open Water /Emergent Seasonally Flooded, excavated)	**PSSCx** (Riparian /Scrub Shrub Seasonally Flooded, excavated)
Site #2 (0.22 ac Enhancement) Bank Stabilization and Channel Enhancement	0.07 ac (Restoration)				0.15 ac (Enhancement)	
Site #3 (0.95 ac Creation and Enhancement) Stormwater Treatment Wetland Complex	0.14 ac (Creation)	0.21 ac (Creation)	0.37 ac (Creation and Enhancement)	0.10 ac (Creation and Enhancement)	0.13 ac (Creation and Enhancement)	
Site #4 (0.21 ac) Large Wood Habitat Enhancement						0.21 ac (Enhancement)
Total Habitat Creation and Enhancement = 1.38 Acres	**0.21 ac**	**0.21 ac**	**0.37 ac**	**0.10 ac**	**0.28 ac**	**0.21 ac**

1. Habitat Type Classification per Cowardin et. al 1979 and as described in the "Donner Creek Sties #1-4 Pre-Project Monitoring and General Habitat Study (Wildscape Engineering, Inc. 2017).
2. Disturbance areas by habitat type for Site 3 are as shown on Sheet C-4 of the Project Plans; Sites 2 and 4 disturbance areas shown on Sheets C-1 and C-7 are PSSC (Riparian/Scrub Shrub Seasonally Flooded).

Table 6. Cut/Fill Estimates

Location	**Description**	**Cut** (CY)	**Fill** (CY)	**Net** (CY)
Site 2				
	Restore eroded west bank		300+/-30	100
	Rework east channel bar	200+/-30		
Site 3				
	Wetland Enhancement/Creation	3200+/-300		2700
	Place salvaged topsoil/wet meadow Sod		500	
Site 4				
	Log Bendway Weir Complex		150	150
Total				**5080**

Works Cited

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Attachment 1
Grass-lined Swale Calculations

Per Table 865.2 of the Caltrans Highway Design Manual (HDM), the permissible velocity and shear stress for a long native grass-lined swale are 6 fps and 1.7 psf. The calculations below demonstrate that the proposed design meets these standards. Note, the parameters proposed for the swales, i.e. width and slopes, are consistent with the design guidelines portrayed in Table B-1 of the Caltrans Project Planning and Design Guide. Because of the low slope and shear stress (see calculation below), erosion control fabric is not proposed for the swales.

Design Flow
According to the Caltrans HDM, the design storm for freeway drainage design should be the 25-year storm. Using the Rational Method, the maximum 25-year flows rate is calculated as follows:

$Q_{max3} = CiA_{max3} = (0.9)(4.73)(0.64) = 2.72$ cfs
$Q_{max4} = CiA_{max4} = (0.9)(4.73)(1.24) = 5.28$ cfs

where:
Q_{max3} = Maximum design discharge for Site 3, cfs
Q_{max4} = Maximum design discharge for Site 4, cfs
C = Coefficient of runoff = 0.9 (0.7-0.95 for asphalt per Caltrans HDM Table 819.2B)
i = Average rainfall intensity in inches per hour for the selected frequency and for a duration equal to the time of concentration = 4.73 for 25-year storm and 5-minute time of concentration (from: https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca)
A_{max3} = Largest drainage area feeding the Site 3 proposed swale (east of outlet) = 0.64 acres
A_{max4} = Largest drainage area feeding the Site 4 proposed swale (west of outlet) = 1.24 acres

Velocity
The maximum velocity in the swales can be calculated from maximum design discharge above and approximate swale geometry as follows:

$V_{max3} = Q_{max3}/A_{x3} = 2.72/0.78 = 3.49 \text{ fps} < 6 \text{ fps}$
$V_{max4} = Q_{max4}/A_{x4} = 5.28/1.05 = 5.03 \text{ fps} < 6 \text{ fps}$

where:
V_{max3} = Maximum flow velocity in Site 3 swale, fps
V_{max4} = Maximum flow velocity in Site 4 swale, fps
A_{x3} = Cross-sectional flow area in Site 3 swale = 0.78 sf (measured in CAD for 3-foot width and 6-inch depth, specified as minimum allowable)
A_{x3} = Cross-sectional flow area in Site 3 swale = 1.05 sf (measured in CAD for 4-foot width and 8-inch depth, specified as minimum allowable)

Shear Stress

The maximum shear stress in the swales is calculated from the swale slope and maximum depth of flow as follows:

$\tau_{d3} = \gamma d_3 S$ = (62.4)(0.75)(0.007) = 0.33 lb/sf < 1.7 lb/sf
$\tau_{d4} = \gamma d_4 S$ = (62.4)(1.0)(0.007) = 0.44 lb/sf < 1.7 lb/sf

where:

τ_{d3} = Shear stress in Site 3 swale at maximum depth, lb/sf
τ_{d4} = Shear stress in Site 4 swale at maximum depth, lb/sf
γ = Specific weight of water = 62.4 lb/cf
d_3= Maximum depth of flow in Site 3 swale for the design discharge, 0.83 ft (max swale-full depth = 9", any overflow discharges to biofiltration strip)
d_4= Maximum depth of flow in Site 4 swale for the design discharge, 0.83 ft (max swale-full depth = 9", any overflow discharges to biofiltration strip)
S = Slope of channel = 0.007 ft/ft (0.6-0.8%)