nhc
northwest hydraulic consultants

NHC Ref. No. 2002962

02 August 2017

Lewis County Public Works
2025 NE Kresky Ave
Chehalis, WA
98532

Attention: **Ann Weckback**
Environmental Planner

Via email: Ann.Weckback@lewiscountywa.gov

Re: **South Fork Chehalis River at Bank Protection Design Boistfort Road**
60% Plan Basis of Design Report

Dear Ms. Weckback:

Under a task assignment dated May 16, 2017, NHC is submitting the 60% bank protection design for South Fork Chehalis River upstream of the confluence with Stillman Creek, where erosion has occurred to the west of Boistfort Road. This letter report summarizes the existing condition at the project site, hydrologic and hydraulic analysis conducted as part of this project, and design constraints and presents the 60% design plan to prevent further erosion and protect the road. The 60% plan is provided in Attachment A.

1 BASIN AND REACH CONDITIONS

1.1 Basin Conditions

The project location on the South Fork Chehalis River (SF Chehalis) is approximately 5.3 miles upstream of the confluence of the with the Chehalis River. The project site's hydraulics are expected to be affected by backwater from the Stillman Creek confluence approximately 250 feet downstream.. Figure 1 shows the drainage areas delineated for the project site and for Stillman Creek. Watershed characteristics for both drainage areas are summarized in Table 1.

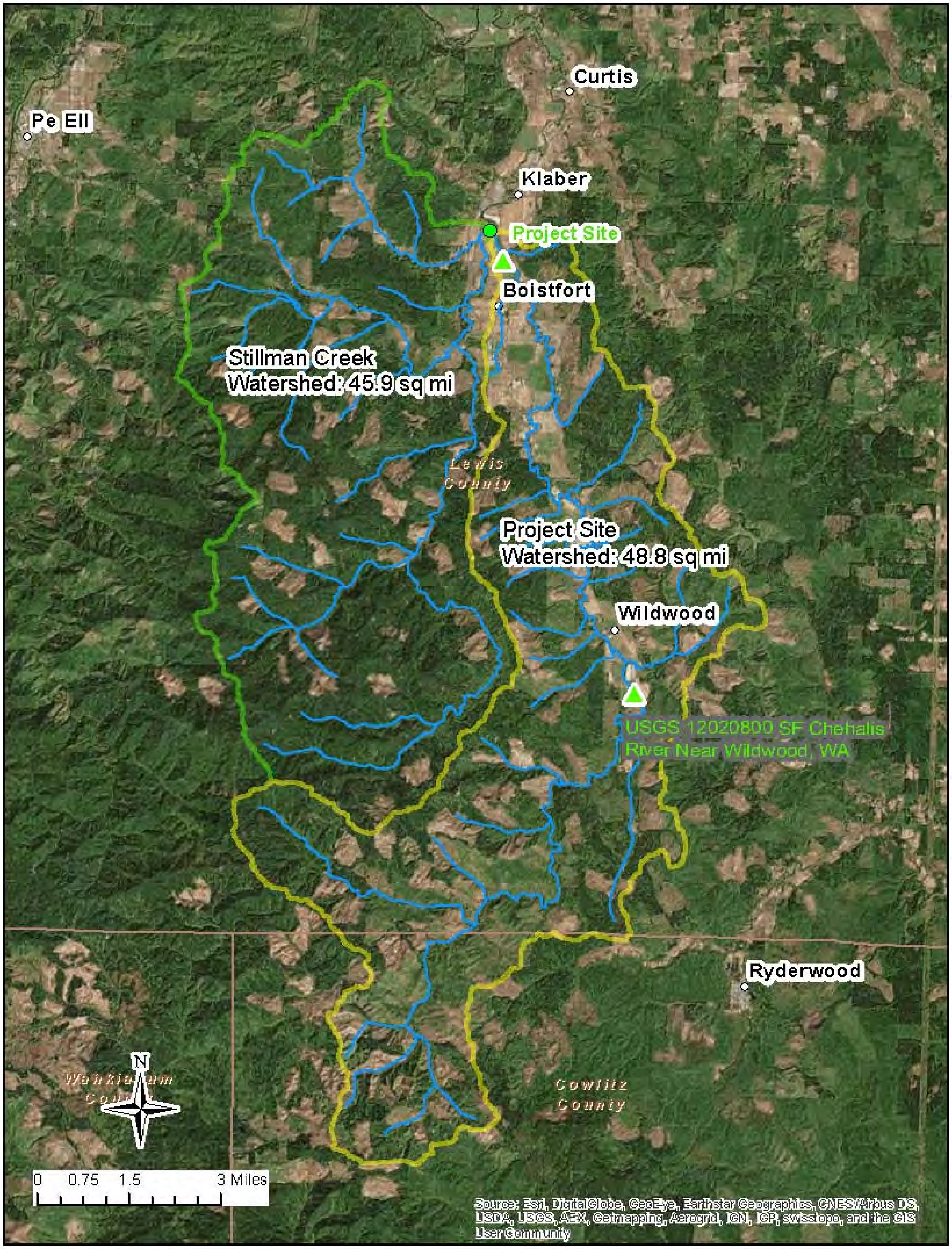


Figure 1. Project site and Stillman Creek drainage areas.

Table 1. Drainage Area Characteristics

Watershed	Drainage Area (sq mi)	Elevation Range (ft NAVD88)	Mean Annual Precipitation (in)	Forest Canopy Cover (%)
Project Site	48.8	244 - 2660	78.5	47.1
Stillman Creek	45.9	1190 - 3140	86.8	67.4
Watershed statistics from StreamStats Washington.				

Based on the discharge summarized in the *Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species* study (WSE 2013), Stillman Creek contributes about 36% of the SF Chehalis River at confluence with Stillman Creek's peak flows.

1.2 Reach Conditions

The segment of the SF Chehalis upstream of the Stillman Creek confluence has a typical bankfull width of 50 to 60 feet, widening to 140 to 180 feet downstream of the confluence. LiDAR data show the slope of the river thalweg is approximately 0.36% upstream and 0. 28% downstream of the confluence. This segment of the SF Chehalis River exhibits pool-riffle morphology with sand to cobble size bed material. Stillman Creek's bankful width ranges from 50 to 60 feet, and the gradient is about 0. 17 percent. For reference in this report and on the construction plan set, NHC established stream channel station lines along the SF Chehalis River and Stillman Creek. Figure 2 shows these two stationing lines overlaid on a recent aerial image.

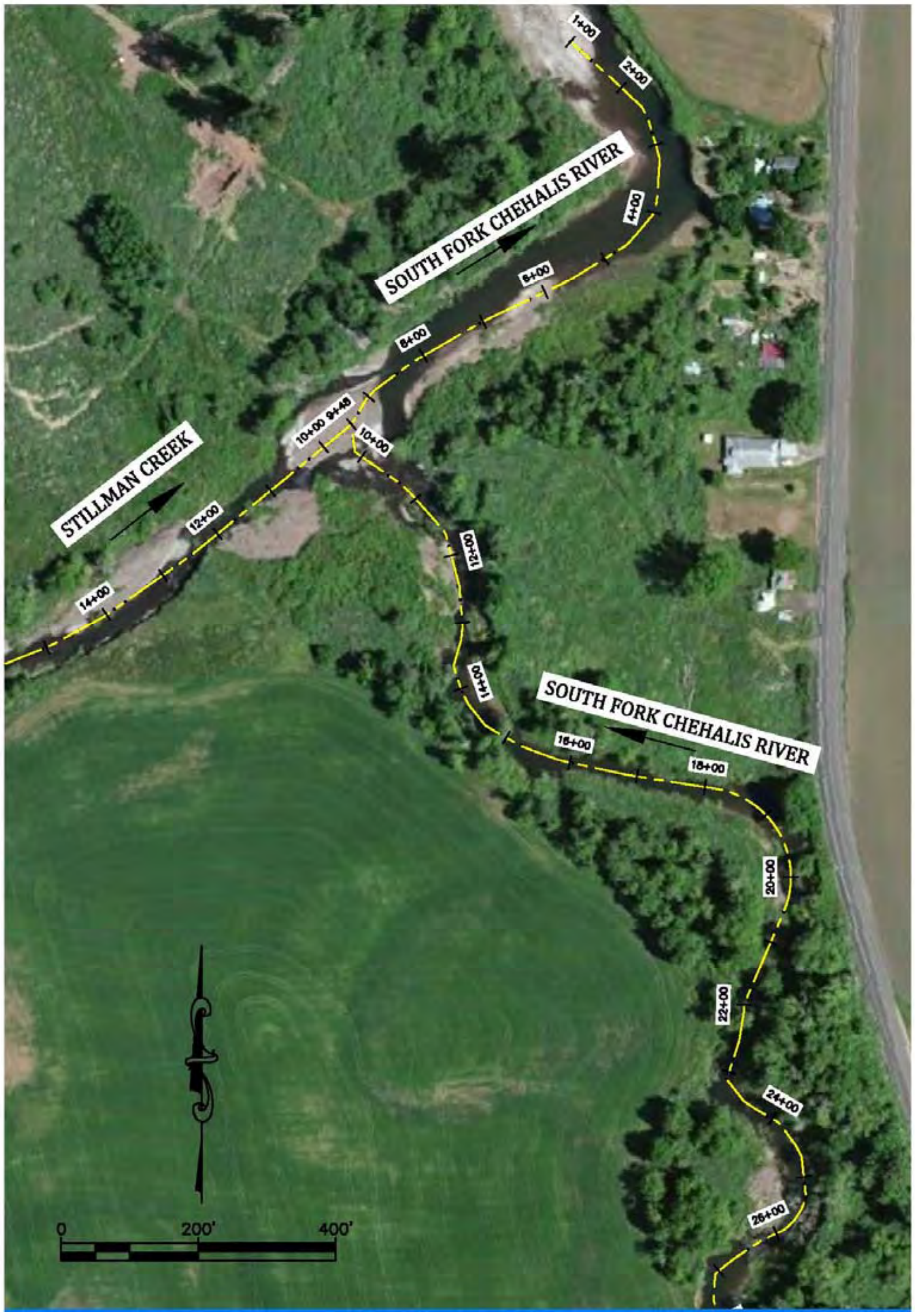


Figure 2. Aerial view of project reach and stream channel centerline.

2 SITE CONDITIONS AND BANK FAILURE

The project site is at Mile Post 4.152 of Boistfort road, where a bank failure has developed and erosion is being exacerbated by flow deflection due to the snags in the stream channel. Photo 1 shows the condition of the project site on 8 June 2017 looking downstream. The site is currently characterized by a coarse cobble deposit on the inside left bank of the channel that pushes flow towards the eroding right bank of the channel. Two large fallen trees located in the center of the channel are exacerbating the right bank erosion by directing additional flow to the right side of the channel. The existing right bank condition consists of exposed bare soil with limited vegetation due to the recent treefall and erosion events. The edge of pavement along Boistfort Road is about six feet away from the top of bank at its closest point. Photo 2 shows the steepness of the eroded bank. Photo 3 shows the project site looking upstream. From this perspective, the left bank deposit directing flow toward the right bank road grade is visible, as is the vegetated right bank (left side of photo) upstream of the over-steepened section. A tie-in with the existing bank can be planned at that location. Photo 3

Photo 1. Eroded bank slope, looking downstream from top of bank. Photo taken on 6/8/2017.

Photo 2. Eroding bank showing erosion from flow redirected by fallen tree.

Photo 3. Eroded bank slope, looking upstream from top of bank. Photo taken on 6/8/2017.

3 DESIGN CONSIDERATIONS AND CONSTRAINTS

NHC identified the following considerations and constraints on the solution to protect and rehabilitate the SF Chehalis River bank at Boistfort Road:

- There is a minimum shoulder width requirement between the bank and edge of road. A Lewis County representative communicated to NHC that a guardrail is not required if there is at least 10 feet of flat terrain (bank slope of 4:1 or flatter) from the edge of road. Having less than this minimum shoulder width may require construction of guardrail.
- Examination of the survey topography and field reconnaissance shows that there are two 4-foot deep scour holes upstream of the fallen trees. This suggests a minimum of 4-foot scour depth. Calculation based on the 2D hydraulic model results shows a maximum anticipated scour depth of up to 8 feet below the existing channel bed. The design scour elevation is 236.0 feet NAVD88. All logjams will be keyed in below the existing bed and intended to adapt and maintain function if a scour hole develops. Section 6.3 provides additional details.
- The construction process can be expedited if the project features fits within the requirements of Nationwide Permit 13. Nationwide Permit 13 requires, among other things, a project length of no more than 500 feet and no more than one cubic yard per foot (on average) of fill below the Ordinary High-Water Mark (OHW). Lewis County surveyed the OHW to be 246.3 ft NAVD88 at the project site. This means that the lower five to six feet of the riverbank are below OHW and can only be filled with native material or woody debris.
- The SF Chehalis River is identified as a Zone AE special flood hazard area on the FEMA Flood Insurance Rate Map (FEMA 2006). The west bank of the channel is identified as Zone X. The proposed designs are optimized to stay within the FEMA no-rise requirements.
- Equipment access is limited by the overhead powerline at the top of the bank. Equipment may need to construct a 12-inch thick by 15-foot wide access pad at the toe of site to construct proposed structures.

4 HYDROLOGY

NHC opted to use the flows published in the *Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species* study’s calibrated HEC-RAS model for this project (WSE 2013). These flows were compared with peak flow frequency analysis from nearby USGS Chehalis River at Doty gage (12020000) and the published FEMA Flood Insurance Study (FIS) flows, both scaled to the project site. Flow comparisons are summarized in Table 2. The WSE flows are 9 to 11% lower than the Doty gage-based peaks. This difference corresponds to the reduction adjustment WSE applied to the recurrence interval peak flows after calibration and validation to the February 1996 and January 2009 events. The FEMA FIS analysis was updated in 2010 to include gage data up to from 1999 to 2009. The following sections provide detail on each flood flow estimate method.

Table 2. Event peaks at the project site from 2013 WSE study, scaled flood frequency analysis at Chehalis River at Doty Gage (USGS 12020000), and scaled FEMA FIS flows.

Recurrence Interval, year	WSE HEC-RAS Model Inflows	Chehalis @ Doty Gage Flood Frequency Scaled to Project Site	FEMA FIS Flows Scaled to Project Site
100	14,960	16,820	6,260
50	-	14,300	5,440
20	9,900	11,280	-
10	8,070	9,190	3,940
2	4,270	4,650	-

4.1 Flood Event Flow Estimates

Peak flow data from the Chehalis River at Doty USGS for 1939 through 2016 were used to compute flood frequency quantiles using the Bulletin 17B method (Hydrology Committee of the US Water Resources Council, 1981). There are four USGS gages in the vicinity of the project site: Chehalis River near Adna (USGS 12021800) is 6.4 miles downstream of the site and includes flow from Stillman Creek; SF Chehalis River at Boistfort (USGS 12020900) is 2 miles upstream of the site; SF Chehalis River at Wildwood (USGS 12020900) is 8.5 miles upstream of the site; and Chehalis River at Doty (USGS 12020000) is in an adjacent watershed west of the SF Chehalis that has similar meteorological and physiographic characteristics as demonstrated by the range of precipitation and elevations. The Doty watershed is about 113 mi^2. The project site drainage area is 48.8 mi^2, 43% of the Doty gage. The considerations for selecting an appropriate gage for flood frequency analysis to scale to the project site are as follows:

- Gage record must be sufficiently long (greater than 20 years) and contain the recent high flow events recorded in 1996 and 2009.
- The drainage area of the gage site should be between 50% and 150% of the project site drainage area to be within recommended application limits of the USGS basin area scaling relation.

The project site drainage area of 48.8 square miles is 180% of the Wildwood gage area and 43% of the Doty gage drainage area. The Doty gage is the best fit in terms of drainage area and has the longest record of data up to 2016. The flood frequency computation was performed in HEC-SSP, a software program developed by USACE Hydrologic Engineering Center (HEC 2016). Figure 3 shows the flood frequency plot with computed exceedance curve and confidence limits. One high outlier (December 2007, 65,100 cfs event) was identified in the Doty gage record. The computed recurrence event flows were then transposed to the project site using the USGS basin area scaling relation (Sumioka et al, 1998) shown in Equation 1. The 100-, 50-, 20-, 10-, and 2- years flows transposed to the site are 16,820 cfs, 14,300 cfs, 11,280 cfs, 9,190 cfs, and 4,650 cfs, respectively.

$$Q_{site} = \frac{A_{site}}{A_{gage}}^{R} \qquad \text{Equation 1}$$

Where, A_{site} is the drainage area of the project site (48.8 square miles), A_{gage} is the drainage area of the Doty gage (113 mi^2). R is the regional regression equation area exponent (0.93 for southwestern Washington (Region 3)).

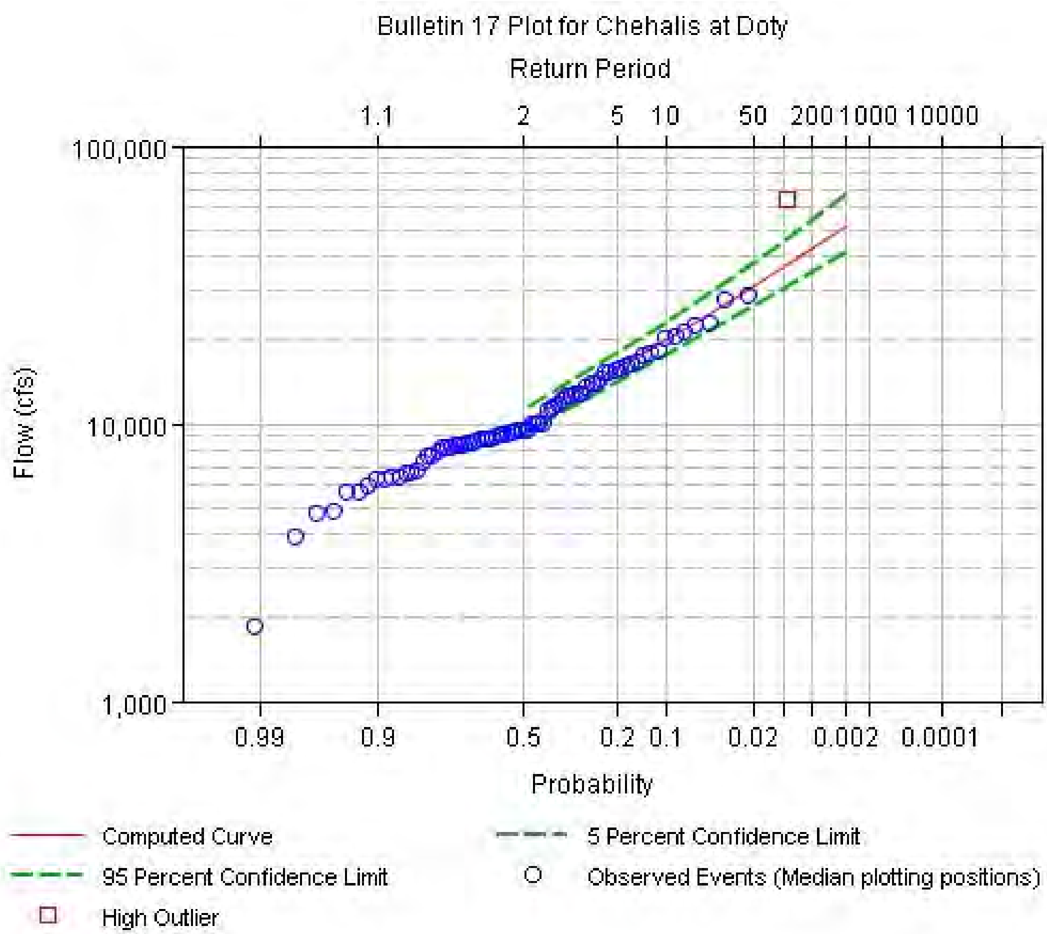


Figure 3. Bulletin 17B Frequency Plot for the Chehalis at Doty Gage (USGS 12020000).

The most recent FEMA FIS was published in November 2010 based on updated hydrology data. The FIS uses the 1965-1980 data recorded at the gage near Boistfort (USGS 12020900) and 1999-2009 data recorded near Wildwood (USGS 12020900) to conduct a flood frequency analysis. Table 13 of the FIS summarizes the peak discharges at the SF Chehalis River confluence with the Chehalis River. The SF Chehalis drainage area at this location is 123 square miles. Equation 1 was used to transpose recurrence event flows to the project site: the scaled 100-, 50-, and 10-year event peak discharges are 6260, 5440, and 3940 cfs.

Detailed hydrologic and hydraulic analysis was conducted as part of a previous study to develop and calibrate a HEC-RAS model for the February 1996, January 2009, and December 2007 flood events (WSE & WEST 2012). The calibrated model was then used to simulate synthetic storm events for 500-, 100-, 20-, 10-, and 2- year events, as documented in the *Chehalis Basin Strategy: Reducing Flood Damage and Enhancing Aquatic Species* study (WSE 2013). The HEC-RAS model was provided to NHC by WSE for the development of this project's 2D hydraulic model. The inflow to the SF Chehalis River reach was applied at river mile (RM) 5.84, which is 0.4 miles upstream of the project site. These flows are directly applicable to the site.

4.2 Monthly Average Flows

The mean daily flow data at the SF Chehalis gage at Wildwood from the period of 1998 to 2017 was analyzed to establish monthly average flows. The record was transposed to the site with the USGS drainage area relation (Equation 1). The Wildwood gage shows very low flows during the months of June through September, with no flow recorded in some years. A zero-flow assumption is unreasonable for the larger project watershed, so NHC supplemented the data based on the SF Chehalis River at Boistfort record, which extended from 1965 through 1980. The monthly average flows computed for the site from the combined gage records are shown in Figure 4. The average monthly flow from May through September ranges from 7 cfs to 65 cfs.

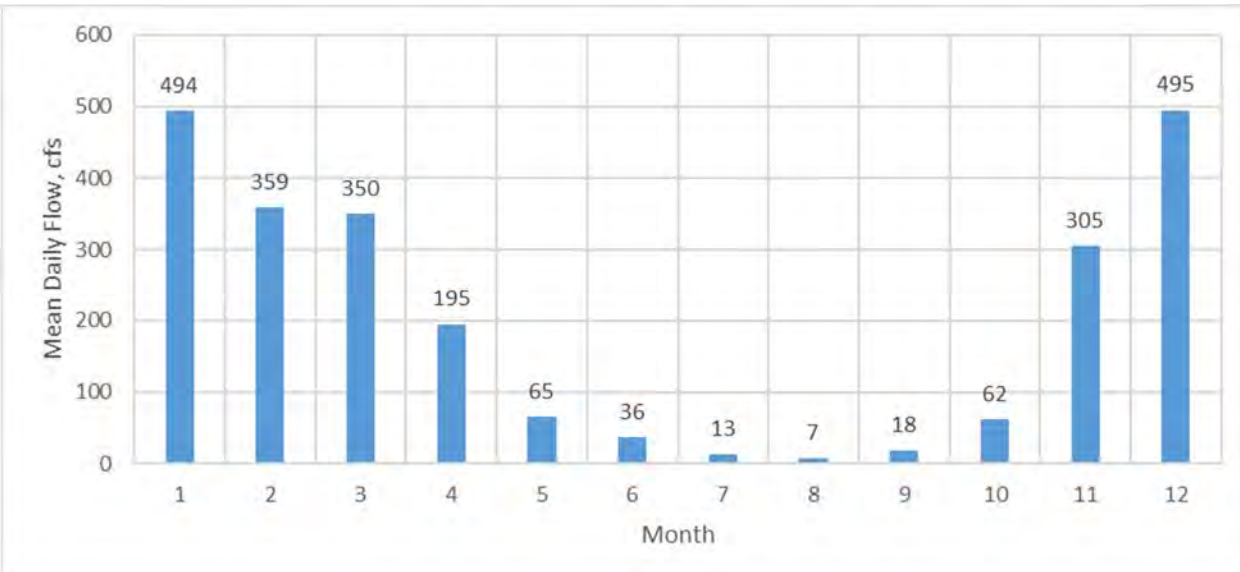


Figure 4. Monthly average flow for project site.

5 EXISTING CONDITION HYDRAULIC ANALYSIS

5.1 Introduction

NHC developed a two-dimensional hydraulic model to compute the hydraulic impacts and design forces for the bank protection design. The US Bureau of Reclamation's SRH2D computation routine was selected for this task. SRH2D solves the two-dimensional depth-averaged dynamic wave equations using flexible mesh for robust and stable numerical schemes with a seamless wetting-drying algorithm (USBR 2008). NHC has applied this computation routine in various bank stabilization and habitat feature design projects in California and Washington with good results.

5.2 Model Setup

The model was set up to include 3,180 feet of the SF Chehalis River and 1,200 feet of Stillman Creek, which joins the SF Chehalis River at Station 9+48. The outflow boundary condition was set near the WSE (2013) HEC-RAS cross section at River Mile (RM) 5.29, the SF Chehalis River inflow was applied near RM 5.8, and the Stillman Creek inflow was applied at the model boundary 1,200 feet above the confluence. Figure 5 shows the model extent, WSE (2013) HEC-RAS cross section locations, inflow and outflow boundary locations, and topographic data sources for the existing condition model geometry.

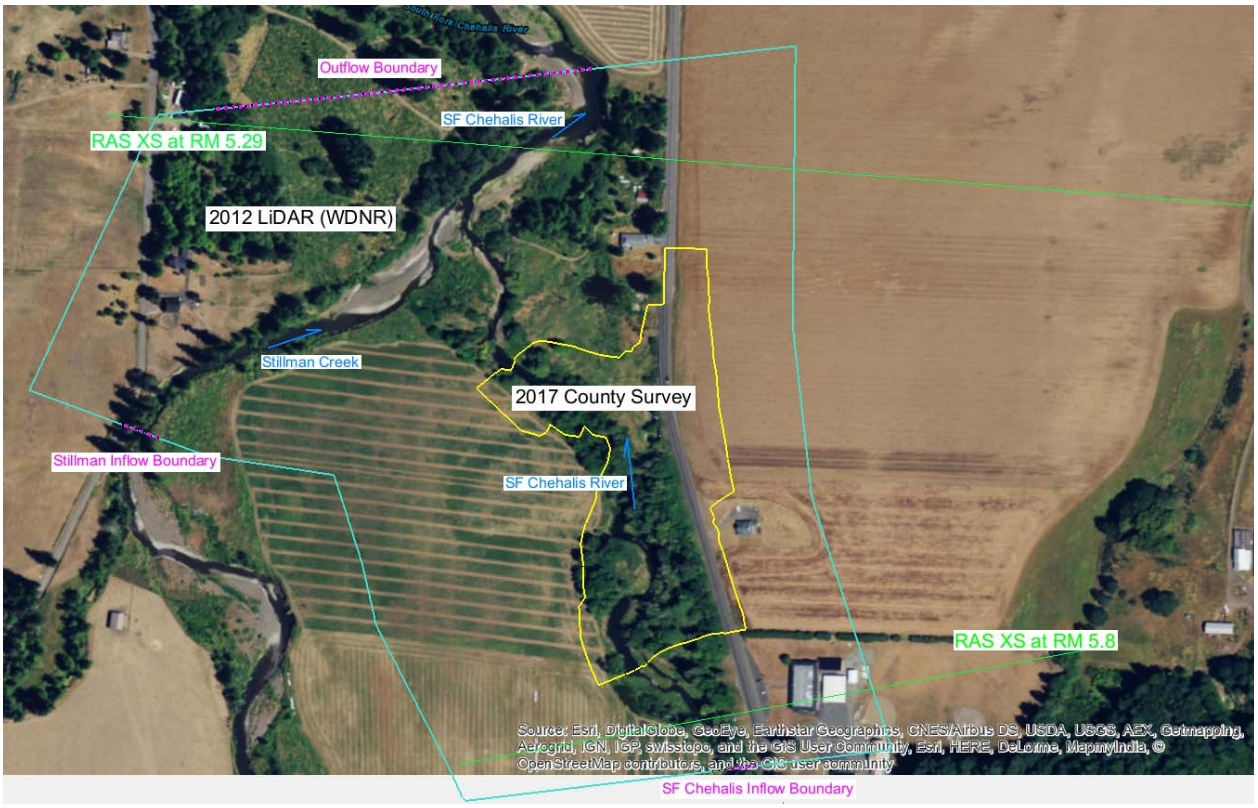


Figure 5. SRH-2D model domain and project features.

The geometry data sources for the model extent includes 2017 topographic data collected by the County and 2012 LiDAR from the Washington State Department of Natural Resources LiDAR portal. The 2017 topographic data covers the SF Chehalis channel and part of the left bank floodplain from 800 feet upstream to 450 feet downstream of the bank protection site. Outside of the survey topo area, the 2012 LiDAR was used.

It is important to note that LiDAR typically does not capture data below water surface. Figure 6 shows the SRH2D and HEC-RAS geometry data at RAS cross section 5.29 and station 4+88. Figure 7 shows RAS cross section 5.8 and station 31+69. Comparison of the LiDAR, HEC-RAS cross sections, and survey data shows that the LiDAR data is approximately four feet higher than the actual channel bottom. Therefore, channel toe breaklines were incorporated into the LiDAR data.

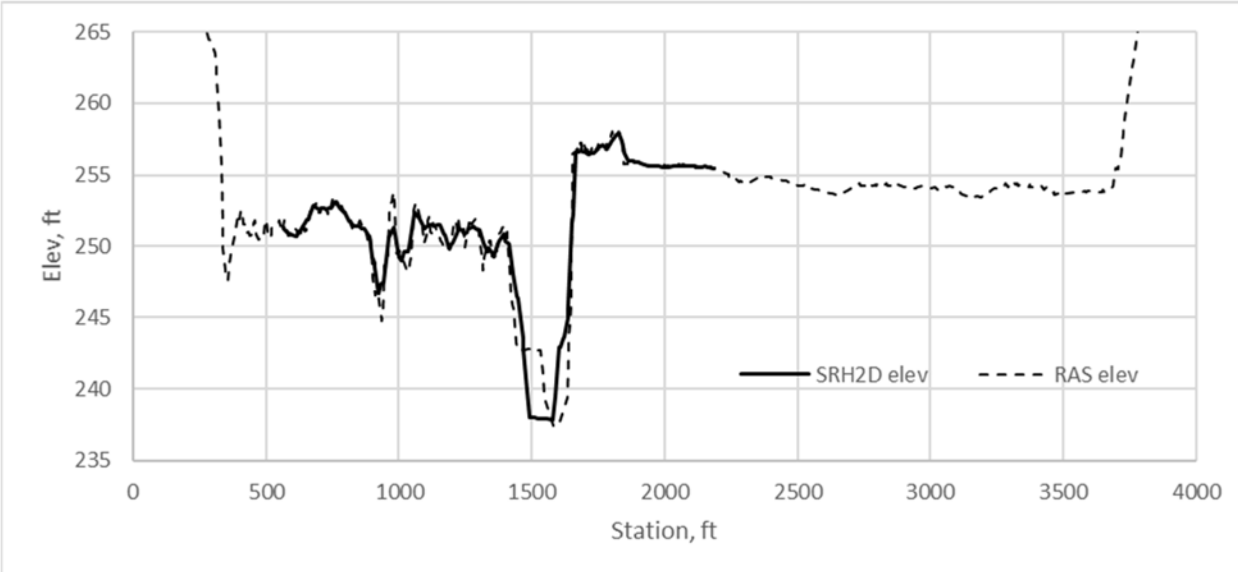


Figure 6. Comparison of SRH2D and RAS model cross section data. (RAS XS at RM 5.29 and Station 4+88)

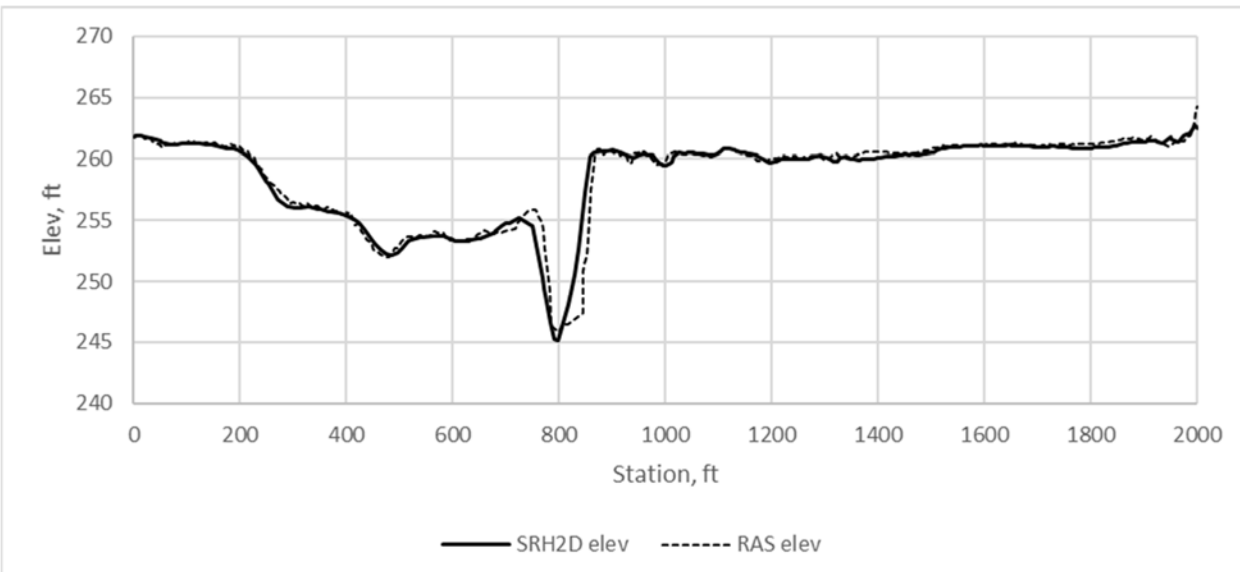


Figure 7. Comparison of SRH2D and RAS model cross section data. (RAS XS at RM 5.8 and Station 31+69)

Figure 8 shows the land cover types and n-values assigned in the SRH2D model based on aerial imagery and field observation. The n-value for each land cover was assigned by the engineer and adjusted to calibrate the model results with WSE's HEC-RAS model results. Figure 9 shows the computed 2-year and 100-year water surface profiles from the SRH2D and HEC-RAS 1D models. Note there are only two HEC-RAS cross sections within the model extent, and the downstream RAS cross section is used as the outflow boundary of the SRH2D model. Therefore, the n-value calibration was mainly used to check for similar magnitude of friction loss computed in both models, such that the water level at the upstream RAS cross section matches reasonably well.

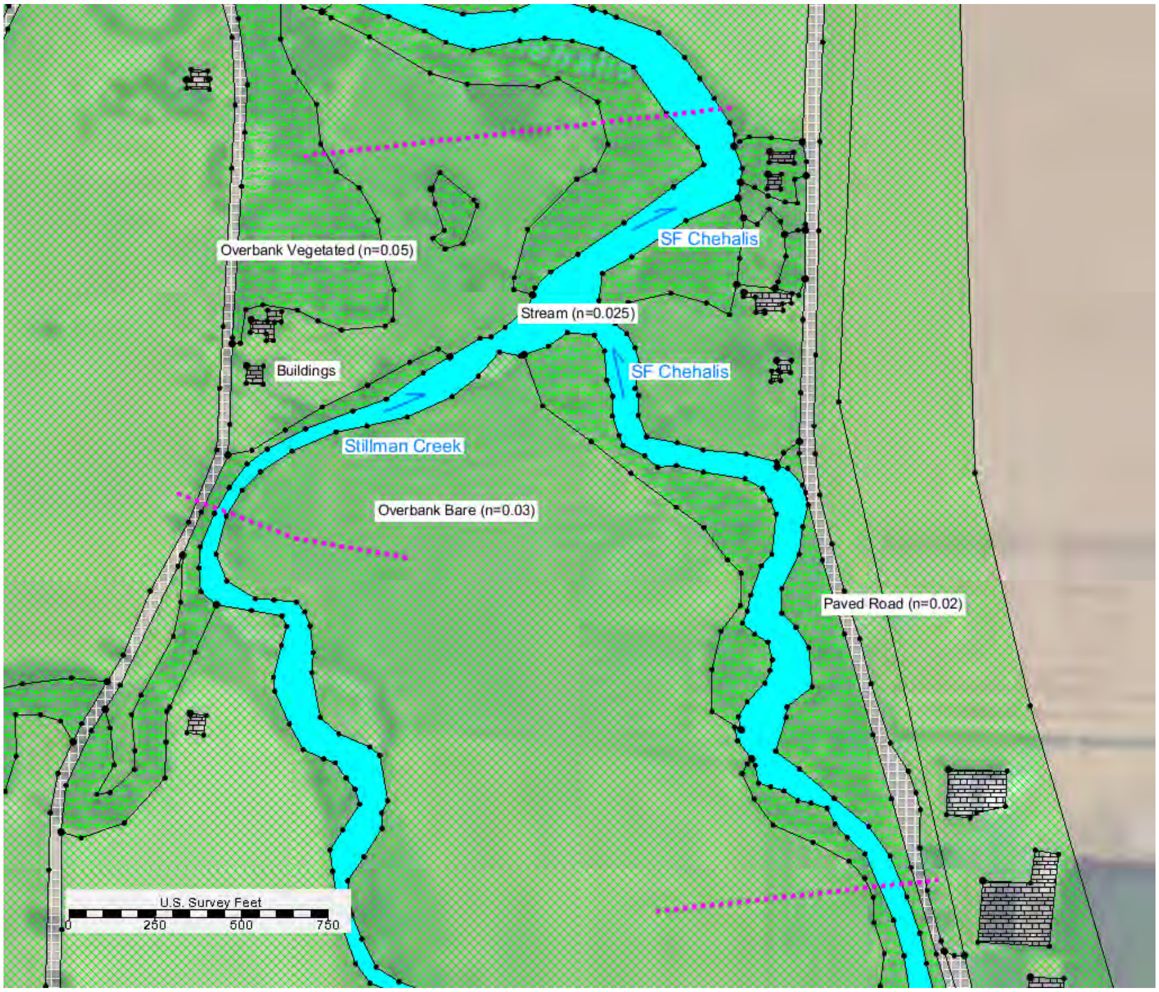


Figure 8. Land cover and n-values for SRH2D model.

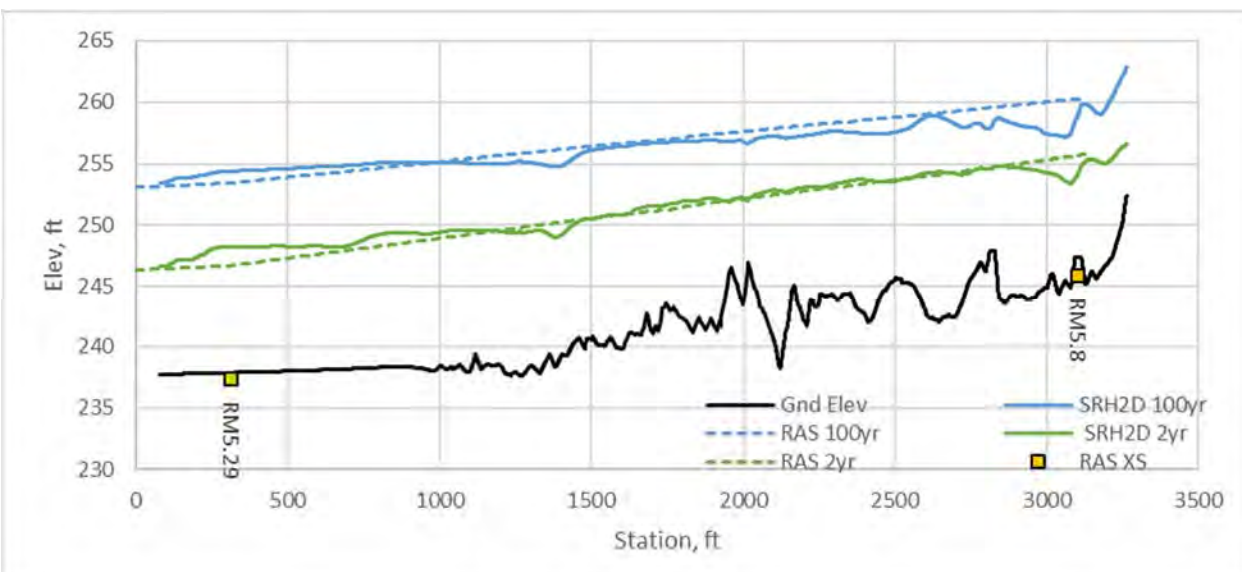


Figure 9. 100yr and 2yr water surface profiles from SRH2D and HEC-RAS 1D models.

Inflows from the SF Chehalis River and Stillman Creek were applied at the locations indicated in Figure 5. The downstream boundary was set to the computed water level at HEC-RAS cross section at River Mile 5.29 for each event. Table 3 summarizes the boundary conditions for the SRH2D model. Steady state simulations were conducted for summer flow, and 2-, 10-, 20-, and 100-year events. Each simulation was computed at a 0.5-second time step for one hour, at which point the steady state equilibrium was achieved throughout the project reach.

Table 3. Inflow and outflow conditions for SRH2D model.

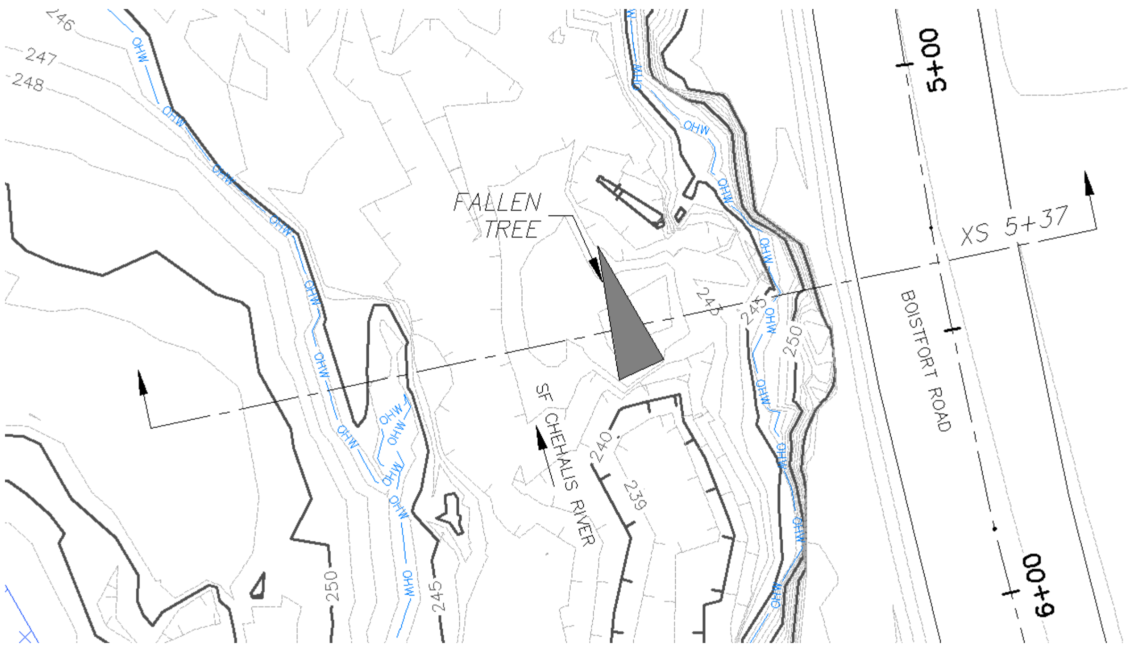
Recurrence Interval, year	Inflow at SF Chehalis US of Boistfort (HEC-RAS Inflow at RM 5.8	Inflow at Stillman Creek (HEC-RAS Lateral Inflow at RM 5.8)	DS Boundary (WSE at RM 5.29)
100	14,960	5,470	253.4
20	9,900	3,600	251.1
10	8,070	2,950	249.7
2	4,270	1,540	246.6
Summer Flow	65	24	238.0

5.3 Model Results

Figure 10 shows the 100-, 20-, 10-, and 2-year flow, as well as summer low flow (65 cfs), water surface elevation, velocity, flow depth, and shear stress at Station 19+51 (RM 5.37). The 2-year event shows the highest velocity and shear stress at 6.8 feet per second (fps) and 0.46 pounds per square foot (psf). This is expected as the 2-year bankfull event conveys most flood flows through the channel and has the steepest hydraulic gradient. Other flood events have deeper flow depth and spill flows onto the floodplain, which results in relatively lower velocity and shear stress. Table 4 summarizes the flow depth, velocity, and shear stress, also at Station 19+51, in the channel and on the right bank. This table will serve as the baseline for comparison with the project condition hydraulic model results. Figure 11 shows velocity contours for the 2-year event through the project site. The results indicate that velocities in excess of 5 feet per second occur along the right bank, which validate the observe erosion is caused by high flow velocity on the bank slope. Project feature design will focus on reducing the flow velocity along bank slope to less than 4 fps and directing higher flow within the stream channel.

Table 4. Existing condition model results: water surface elevation, flow depth, velocity, and shear stress for existing condition (Station 19+51, RM 5.37).

		Right Bank Toe			Stream Channel		
Event	WSE (ft)	Flow Depth (ft)	Velocity (fps)	Shear Stress (psf)	Flow Depth (ft)	Velocity (fps)	Shear Stress (psf)
100yr	256.8	13.5	3.5	0.09	10.3	4.9	0.20
20yr	255.4	12.1	3.5	0.09	8.9	4.9	0.21
10yr	254.7	11.4	3.7	0.10	8.2	5.1	0.23
2yr	251.8	8.6	5.0	0.21	5.3	6.8	0.46
65 cfs	243.4	No flow			1.7	1.0	0.05



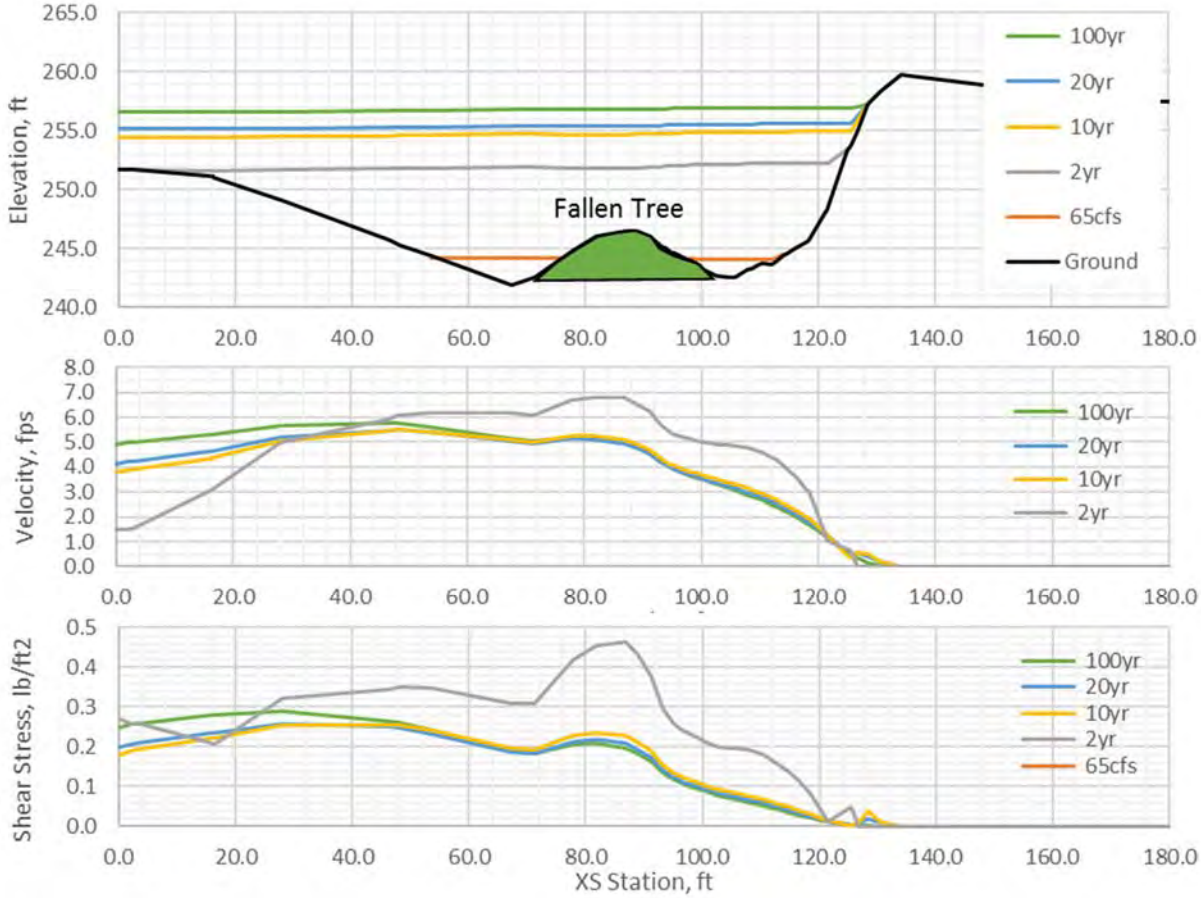


Figure 10. Water surface elevation, velocity, shear stress for cross section (Station 19+51, RM 5.37).

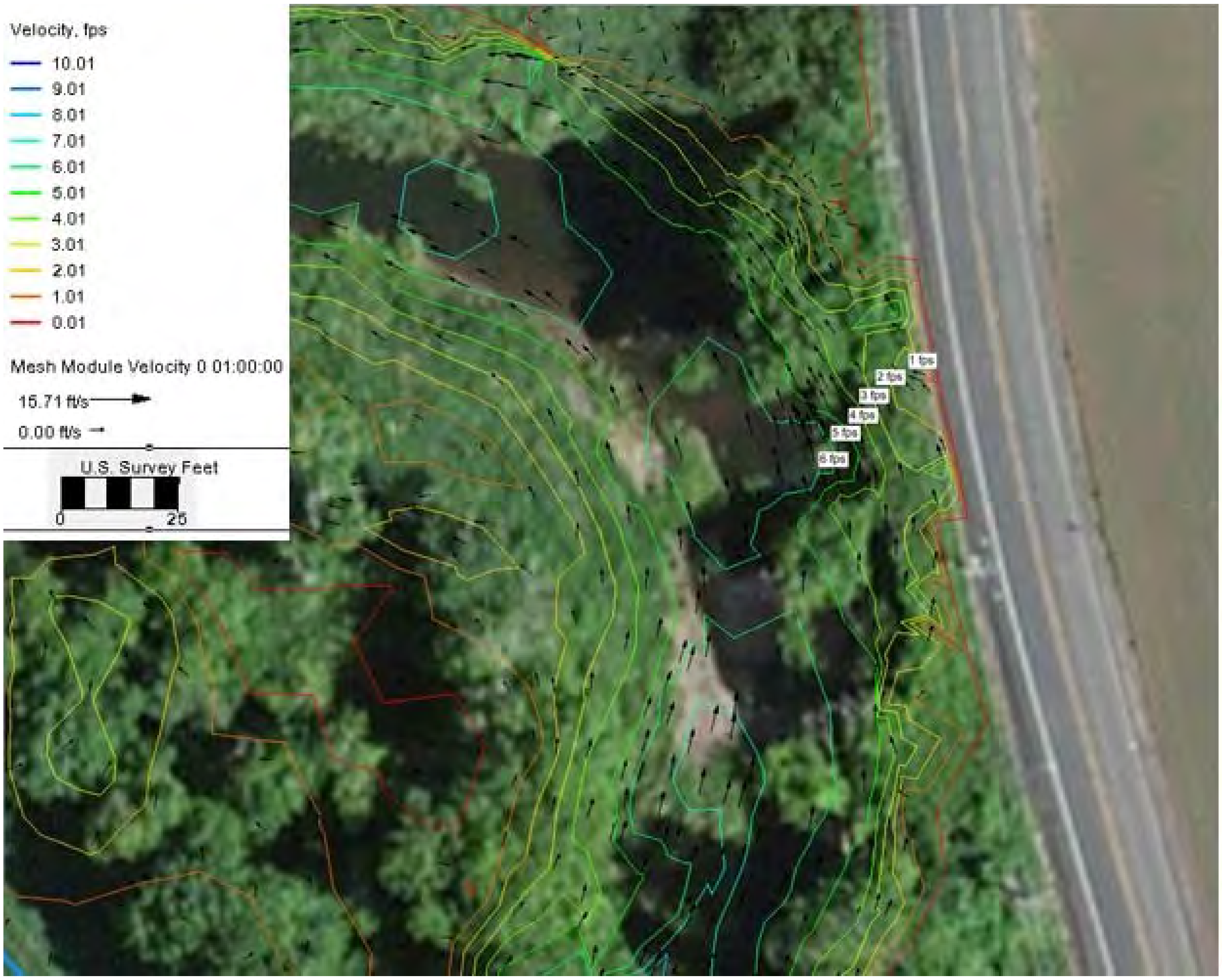


Figure 11. Velocity contour and vectors for 2-year event flow.

6 BASIS OF DESIGN FOR 60% PLAN SET

6.1 Description and Layout

The 60% design plan is in Attachment A. The design proposes to:

- Remove the fallen trees from the channel;
- Place seven engineered log jams at approximately 30’ spacing along the proposed toe line from station 18+40 to 20+00;and
- Install vegetated reinforced soil slope with log toe (VRSSLT) bank protection between log jams 2, 3, and 4.

Sheet 3 of the plan set shows the site topography and project feature layout in plan view. Sheet 5 shows typical details of the small and large log jams and VRSSLT. Log jams 2 through 6 extrude approximately 25 feet from the top of bank and into the channel. Logjams 1 and 7 extrude approximately 15 feet from the bank into the channel. These two smaller log jams provide a transition back to the existing bank line. The overall length of bank stabilization work is 190 feet, and the log jams extend from the channel bed up to the 2-year water level.

6.2 Project Condition Hydraulic Analysis

The project features were added to the existing condition SRH2D model presented in Chapter 5. The project condition contours representing the channel with the fallen trees removed, log jams, and VRSSLT protection are shown in Figure 12. The n-value is selected based on the engineer's judgement of flow depth, the rootwad's projection into flow area in proximity of the Engineered Log Jams. Simulations of the 2-year flood event were conducted to analyze the design flow hydraulics. Simulation of the 100-year flood event was conducted to verify that the design meets the no-rise condition of the FEMA flood zone requirement. The removal of the fallen trees in the stream channel is sufficient to offset the restoration of the bank toe line with the proposed logjams. Section 6.2 provides additional details. Figure 13 compares the water surface elevation, velocity, and shear stress at cross section at RM 5.37. With the log jams in place for the project condition, the velocity at the right bank was reduced from 4.7 fps to 2.6 fps. The shear stress was reduced from 0.19 psf to 0.06 psf. The main flow conveyance was moved away from the right bank by 10 feet or more. This helps encourage sediment transport through the reach and creates a flood conveyance channel away from the eroding bank. Figure 13 compared the 100-year and 2-year water surface profiles for existing and project conditions. There is about 0.1 to 0.2 feet of rise for the 2-year event caused by the addition of the log jams. For the 100-year event, there is negligible difference in the water level. Therefore, the FEMA no-rise criteria is satisfied.

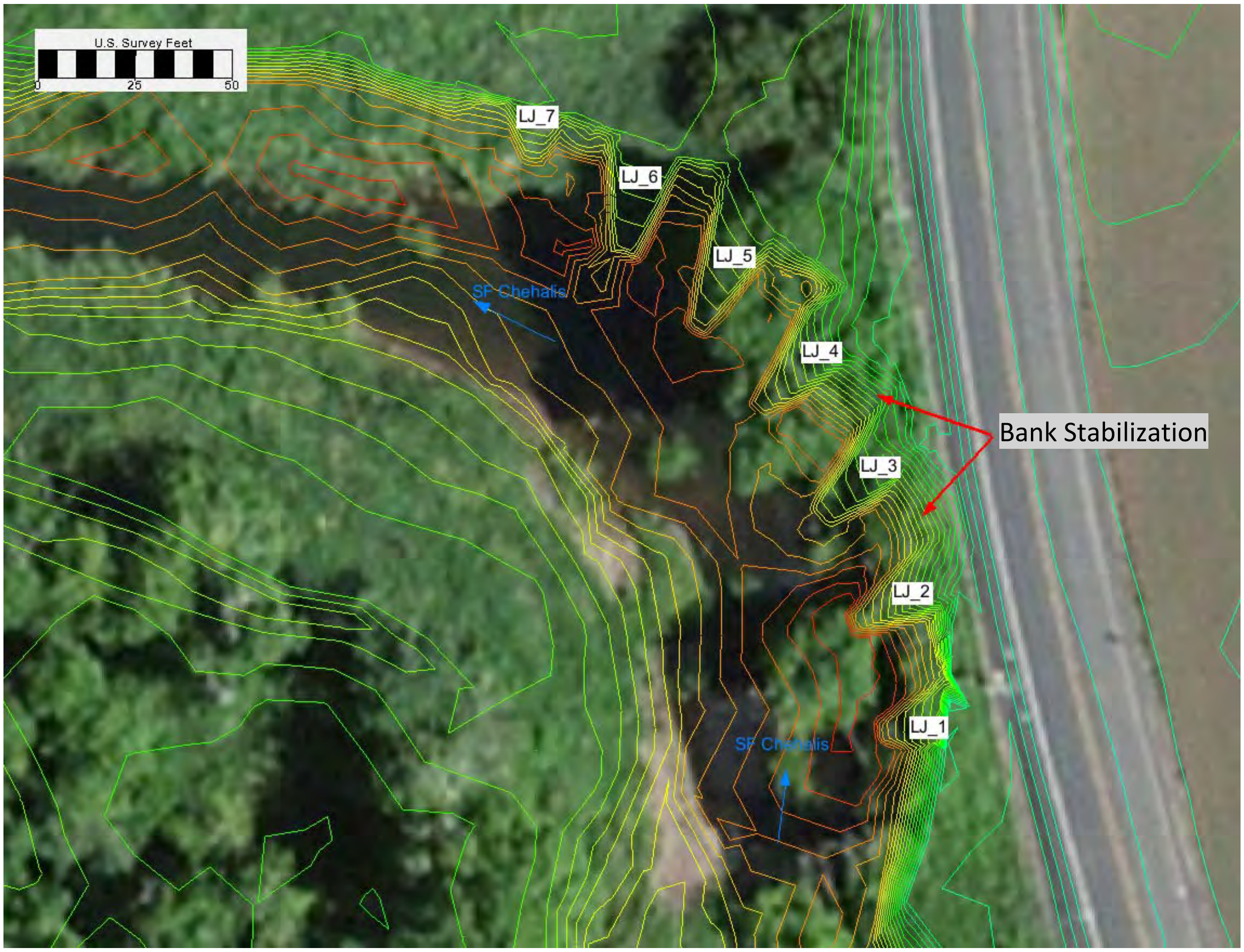


Figure 12. Project condition model contours.

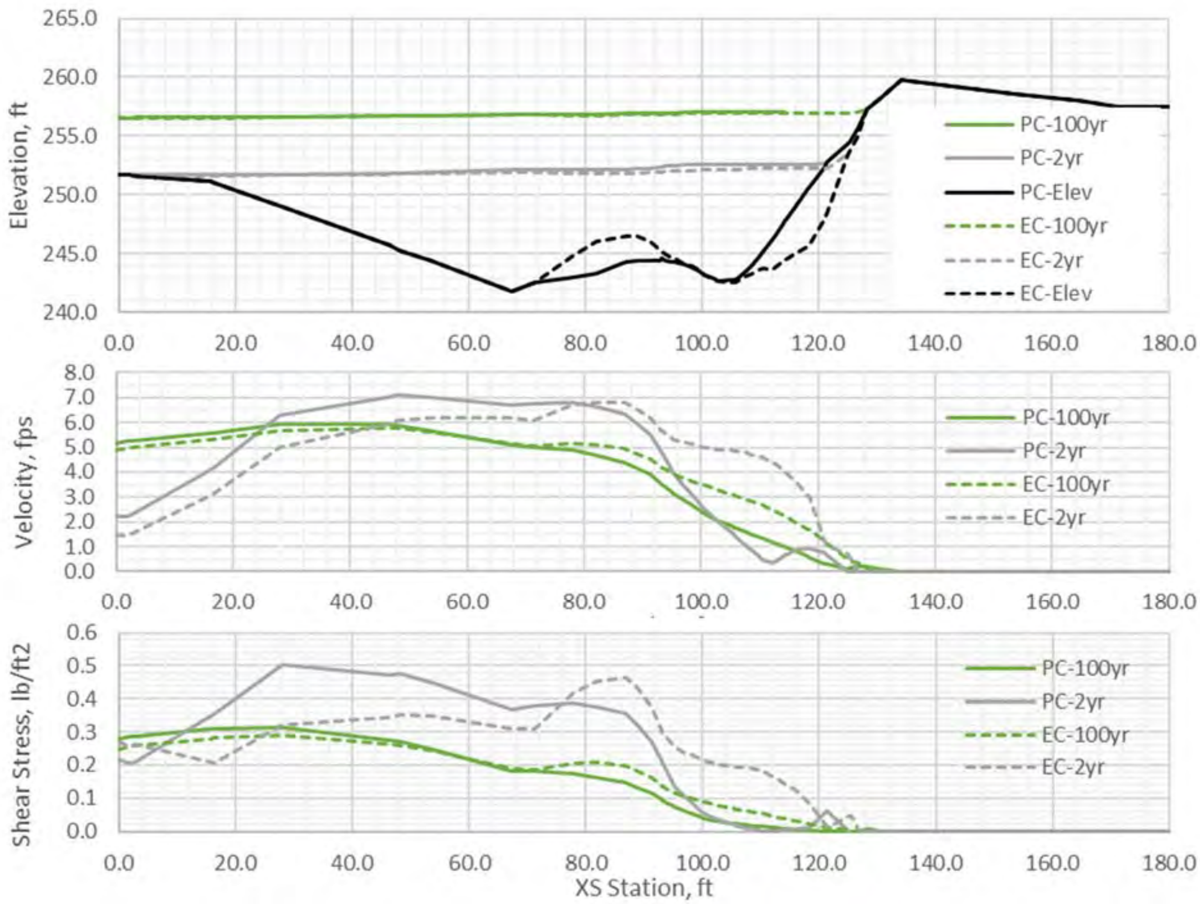


Figure 13. Project condition and existing condition water surface elevation, velocity, shear stress for cross section 5+37.

6.3 Scour Potential

The hydraulic model results were used to compute the minimum scour elevations along the project reach using the NCHRP 24-20 method published in HEC-18 (FHWA 2012). The method is based on contraction scour and applies a factor to account for the large-scale turbulence developed near the abutment (FHA 2012). The input data to this method are the 2D model hydraulic results summarized in Table 5. This table shows the flow depth, channel velocity, flow width and computed scour elevation in front of each log jam. Figure 14 shows the 2-year and 100-year water surface profiles, existing thalweg elevations, and the design scour elevations. Engineered log jams 1 and 7 are located near the bank and above the channel thalweg. Scour depth is minimal around these smaller structures, and should be lower than the bed elevations to below the channel thalweg. Worksheet A in the attachment shows the calculation details.

Table 5. Scour calculation summary.

Station	XS	WSE	Q, cfs	Width, ft	Y_0, ft	Y_{scour}, ft	Scour Elev, ft
20+22	1	252.51	4009	89.5	10.8		
19+97	2	252.49	3517	90.1	13.0	0	239.5
19+74	3	252.43	3633	93	14.1	1.5	236.8
19+45	4	251.98	3543	93.1	8.6	7.9	235.5
19+19	5	252.02	3759	91.2	10.1	4.4	237.6
18+95	6	252.01	3561	101.2	11.5	0	240.5
18+70	7	251.64	3492	96.2	6.7	10.2	234.8
18+39	8	251.61	3543	98.1	10.0	0	241.6

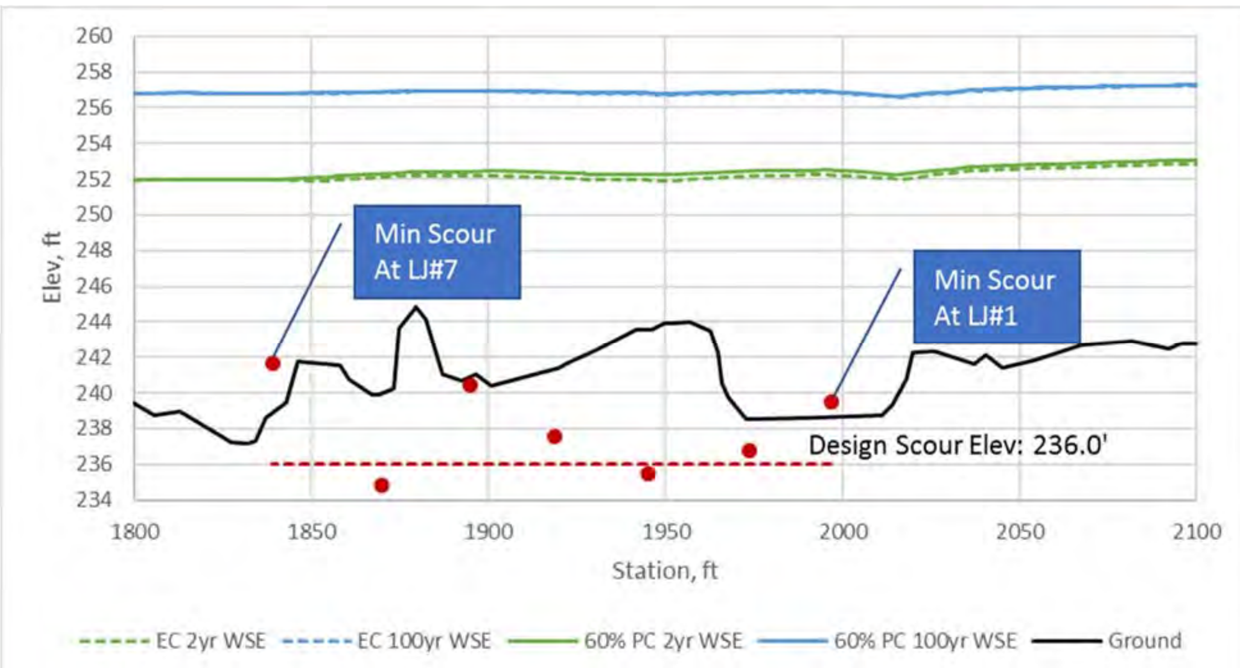


Figure 14. Water surface elevation, minimum scour elevation profile.

6.4 Log Jam Design

The engineered log jams were designed for stability with respect to vertical, horizontal, and rotational forces. Calculations followed the procedures outlined in the National Large Wood Manual (USBR 2016). Table 6 summarizes the resultant forces for small logjams (#1 & 7) and large logjams (#2-6). Worksheet B in the attachment shows the calculation details.

Table 6. Logjams ballast forces summary.

	Log Jams 1 & 7	Log Jams 2-6
Vertical Forces		
Buoyant Weight of Woody Material	7080 lbf	15310 lbf
Lift	700 lbf	2940 lbf
Ballast (Rock Size and Number)	13080 lbf (4x 4' dia)	22900 lbf (7x 4' dia)
2x Manta Ray MR-3 at 45° Angle	14140 lbf	14140 lbf
Safety Factor with Rock Ballast Only (SF with Soil Anchors)	1.68 (3.5)	1.25 (2.0)
Horizontal Forces		
Friction	3060	2680 lbf
Drag	560	2360 lbf
2x Manta Ray MR-3 at 45° Angle	14140 lbf	14140 lbf
Safety Factor with Rock Ballast Only (SF with Soil Anchors)	5.5 (30.8)	1.14 (7.1)
Rotational Moment		
	52920 lbf-ft clockwise into the bank	11020 lbf-ft clockwise into the bank

6.5 Streambed Gradation

The hydraulic parameters computed from the 2-year event project conditions model were used to determine stable rock size in the project reach. NHC used the Army Corp of Engineers' Engineering Manual (USACE 1994) and Abt and Johnson's riprap rock sizing methods (Abt and Johnson 1991) for this estimate. Both methods yield similar rock gradation: D_{30} = 0.41 feet, D_{50} = 0.5 feet, D_{100} = 1.0 feet. Based on these values, an appropriate gradation was selected from the Washington Department of Transportation 2016 Standard Specifications (WSDOT 2016 Section 9-03.11(2)) (Table 7).

Table 7. Native Streambed Gradation Adopted from WSDOT 2016 Standard Specification.

% Finer by Weight	Particle Size
99-100%	12 in
70-90%	10 in
30-60%	5 in
10%	0.75

6.6 Construction Sequence and Considerations

Sheet 6 of the plan set in Attachment A shows the construction sequence to build the log jams. In Step 1, the base log with rootwad is placed perpendicular to the bank toe line and secured with three ballast rocks. A footer log with no rootwad is then placed on top of the base log and angled down to the existing bank line. The logs are tied together with galvanized cable as shown in the log-to-log connection detail. In Step 2, two diagonal logs with rootwads are placed on top of the footer log and tied together with galvanized cable. One four-foot diameter ballast rock is tied to the diagonal logs for ballast. Two Manta-Ray mechanical earth anchors are placed at 45° angles from the ground to secure the whole structure from rotation and add additional ballast. In Step 3, the top log is placed over all the other logs with two ballast rocks. Log-to-log connections are also applied at each intersection with the top log. Engineered streambed material is then placed within the log jams to fill voids within the structure. Between log jams 2, 3, and 4, VRSSLT are to be constructed to buttress the exposed bank slope.

6.7 Guardrail

This proposed design includes installing 150 lineal feet of guardrail along the top of bank, as required for a site with top of bank less than ten feet from the edge of pavement. The guardrail design is in accordance with WSDOT's and Lewis County DPW's requirements. Typical details of the WSDOT Type 1 Wood Beam guardrail are shown on sheet 8 of the plan.

6.8 Quantity and Cost

The engineer's opinion of costs is summarized in Table 8. The cost includes 20% construction management cost and 25% contingency. The estimated total cost is $268,700 for the 190-lineal foot of logjam and VRSSLT bank rehabilitation work.

Table 8. Engineer's opinion of costs for proposed 60% design.

Item Name	Unit	Unit Cost	Quan	Cost
General				
Traffic Control	LS	$6,000	1	$6,000
Construction Survey	LS	$2,000	1	$2,000
Water Isolation	LS	$2,500	1	$2,500
Engineered Log Jam				
Type A Logs - (22-26" dia, 20' long with 3-6' Rootwad)	EA	$1,200	6	$7,200
Type B Logs - (22-26" dia, 20' long with No Rootwad)	EA	$700	7	$4,900
Type C Logs - (22-26" dia, 35' long with 3-6' Rootwad)	EA	$1,800	20	$36,000
Type D Logs - (22-26" dia, 35' long with No Rootwad)	EA	$1,800	4	$7,200
Log Placement/Anchoring/Lashing	EA	$460	41	$17,020
Ballast Rocks (4 Man Rock), delivered and placed	EA	$400	51	$20,400
Mechanical Anchor and Chain	EA	$450	14	$6,300
Galvanized Wire Rope	LF	$2	400	$800
Streambed cobble, delivered and placed below OHW	TN	$80	88.5	$7,080
VRSSLT				
Imported Fill	CY	$30	160	$4,800
Streambed cobble, delivered and placed below OHW	TN	$80	84	$6,720
Geogrid Mesh	SY	$15	1244	$18,667
MIRAMESH GR facing wrap	SY	$20	498	$9,956
MIRAMESH 3XT	SY	$20	560	$11,200
Soil lift installation	LF	$38	80	$3,040
Seeding, native upland mix	AC	$3,000	0.15	$450
		Subtotal		**$172,200**
Mobilization (10% of total)	LS	10%		$17,220
Construction Management & Engineering (20% of total)	LS	20%		$34,440
Contingency (25%)	LS	25%		$43,050
Tax (7.8%)	LS	7.8%		$43,050
		Total		**$309,960**

7 CLOSURE

This letter presents the basis of design for the 60% plan to repair the eroded stream bank at Mile Post 4.152. The analysis is based on the existing condition 2-dimensional model developed using the 2017 survey data and 2012 LiDAR. The existing condition served as baseline for the 100yr no-rise condition and establishing design flow rate of 4270 cfs (2yr event). Section 4 summarizes the hydrologic analysis conducted to establish design flow. Section 5 presents the development of the existing condition model.

Seven engineered log jams are proposed to deflect flood flows away from the right bank (looking downstream). Vegetated reinforces soil slope with log toe (VRSSLT) are proposed between log jams 2, 3, and 4 to provide buttress for the bank slope. The proposed design was modeled in 2-dimensional model to predict flow pattern and hydraulic results for scour calculation. Section 6.2 presents the development of the project condition hydraulic model. Section 6.3 and worksheet A shows the calculation of the scour depth. The computed minimum scour elevation is 236.0’. The log jams are placed a minimum elevation of 240.0’, and are expected to adjust and arrest potential scour as localized erosion occur during a flood event. Ballast rocks on top of bank and tip of rootwad will help the log jam structure pivot without compromising the stability of the structure. Section 6.4 presents the ballast calculation for the log jams. Section 6.6 discuss the construction sequence to build the bank protection. The engineer’s opinion of cost for the structure is about 310,000 dollars with 25% contingency and 20% construction management.

Sincerely,

Northwest Hydraulic Consultants Inc.

Erik Rowland, PE
Engineer

ENCLOSURE:

Worksheet A. Project Condition Scour Depth Calculation
Worksheet B. Log Jam Ballast Calculation
SF Chehalis at Boistfort Road Bank Protection 60% Plan

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DISCLAIMER

Worksheet A: Scour Calculation

2yr event for project condition

Physical Parameters

NCHRP 24-20

Abutment Scour Approach, predicts total scour includes: general, local and abutment

Hydraulc Parameter from SRH2D

Upstream flow depth

$y_1 := 11.6\ ft$

$$q_1 := \frac{3603\ \frac{ft^3}{s}}{85\ ft} = 42.4\ \frac{ft^2}{s} \qquad q_{2c} := \frac{3557\ \frac{ft^3}{s}}{80\ ft} = 44.5\ \frac{ft^2}{s} \qquad \frac{q_{2c}}{q_1} = 1.0489$$

Flow depth prior to scour

$y_0 := 13.1\ ft$

$\alpha := 1.5$

Flow depth including live-bed or clear-water contraction scour

$$y_c := y_1 \cdot \left(\frac{q_{2c}}{q_1}\right)^{\frac{6}{7}} = 12.1\ ft$$

Maximum flow depth resulting from abutment scour

$y_{max} := \alpha \cdot y_c = 18.1\ ft$

Abutment scour depth

$y_s := y_{max} - y_0 = 5\ ft$

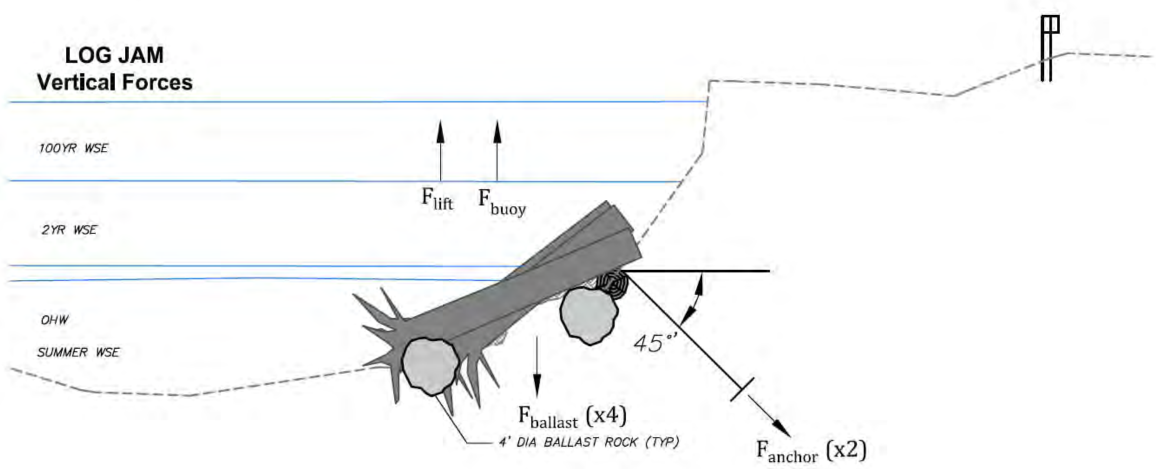
Worksheet B: Logjam Ballast

Physical Parameters

Specific Weight of Wood | Specific Weight of Water | Specific Weight of Boulder

$\gamma_{wood} := 25 \frac{lbf}{ft^3}$ $\quad$ $\gamma_{water} := 62.4 \frac{lbf}{ft^3}$ $\quad$ $\gamma_{rock} := 160 \frac{lbf}{ft^3} = 2.2 \frac{tonf}{yd^3}$ $\quad$ $g := 32.2 \frac{ft}{s^2}$

Type 1 (Logjam #1&7, Most US and DS)



Vertical Force

Area of Structure Project into Flow

$A := 80\ ft^2$

Depth Average Approach Vel (from SRH2D)

$U_0 := 3 \frac{ft}{s}$

Bouyancy Force of Woody Structure

$Fb(D, L) := (\gamma_{wood} - \gamma_{water}) \cdot \frac{D^2}{4} \cdot \pi \cdot L$

Rock Ballast Weight

$F_{rock}(D) := \frac{4}{3} \cdot \pi \cdot \left(\frac{D}{2}\right)^3 \cdot (\gamma_{rock} - \gamma_{water})$ $\quad$ $C_L := 1$

Lift Force of Woody Structure

$F_{lift} := (-1) \cdot \frac{C_L \cdot A \cdot \gamma_{water} \cdot U_0^2}{2 \cdot 32.2 \frac{ft}{s^2}} = -698\ lbf$

3x log each 24 inch Root BDH and 15 ft long, 1x logs each 24 inch BDH and 25 ft long

$Fb_{struct} := (Fb(24\ in, 15\ ft) + Fb(2\ in, 2\ ft) \cdot 6) \cdot 3 + Fb(24\ in, 15\ ft) \cdot 1 = -7079\ lbf$

Ballast Weight

$F_{ballast} := F_{rock}(4\ ft) \cdot 4 = 13082\ lbf$ $\quad$ $AnchorLoad := 10000 \cdot 2\ lbf$ $\quad$ $AnchorAngle := 45\ °$

Geotechnical Restraint (bury log, etc)

$F_{gv} := 0\ lbf$

Anchor Restraint (for romtational)

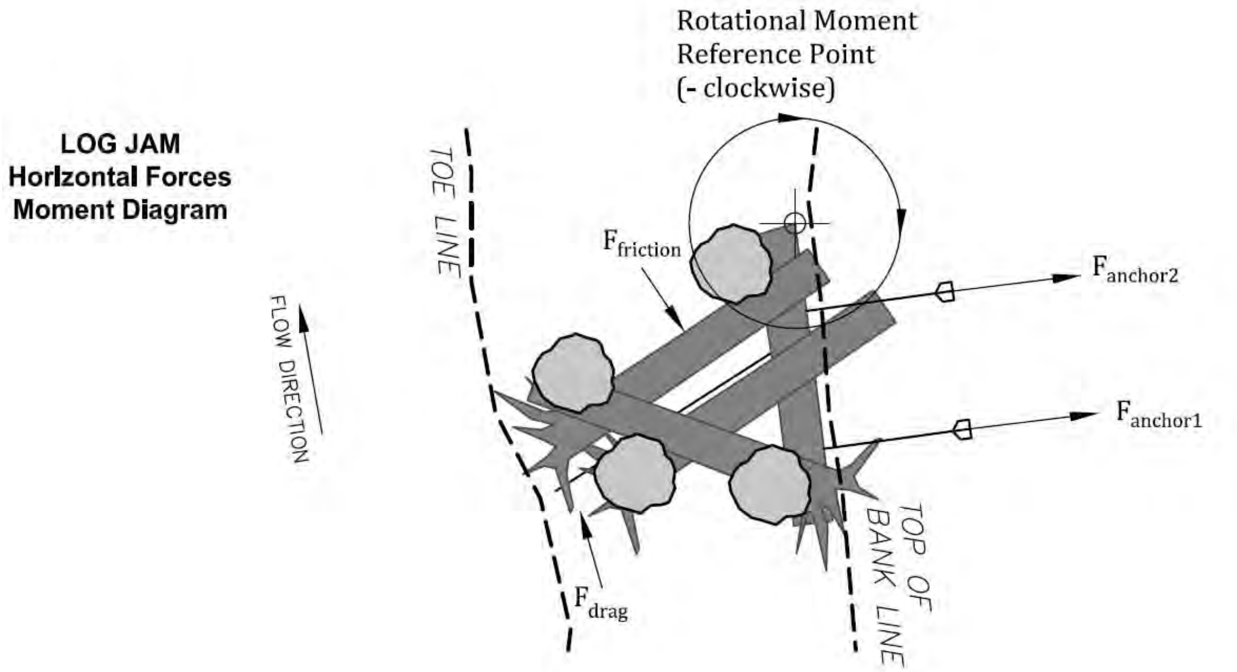
$F_{av} := AnchorLoad \cdot \sin(AnchorAngle) = 14142.1356\ lbf$

Safety Factor (Soil Anchor & Ballast)

$F_{safetyV} := \left| \frac{F_{ballast} + F_{gv} + F_{av}}{Fb_{struct} + F_{lift}} \right| = 3.5$

Safety Factor (Ballast Only)

$F_{safetyV} := \left| \frac{F_{ballast} + F_{gv} + 0}{Fb_{struct} + F_{lift}} \right| = 1.68$



Horizontal Force

$\mu_{bed} := \tan(30\ °)$

$F_{normal} := Fb_{struct} + F_{lift} + F_{ballast} = 5306\ lbf$

Friction Force:

$F_{friction} := \mu_{bed} \cdot F_{normal} = 3063\ lbf$

$C_D := 0.8$

Drag Force

$$F_{drag} := \frac{C_D \cdot A \cdot \gamma_{water} \cdot U_0^{\ 2}}{2 \cdot 32.2 \frac{ft}{s^2}} = 558\ lbf$$

Geotechnical Restraint (bury log, etc)

$F_{gh} := 0\ lbf$

Anchor Restraint

$F_{ah} := AnchorLoad \cdot \cos(AnchorAngle) = \frac{20000}{\sqrt{2}}\ lbf$

Safety Factor (Soil Anchor & Ballast)

$$F_{safetyH} := \left| \frac{F_{friction} + F_{gh} + F_{ah}}{F_{drag}} \right| = 30.83$$

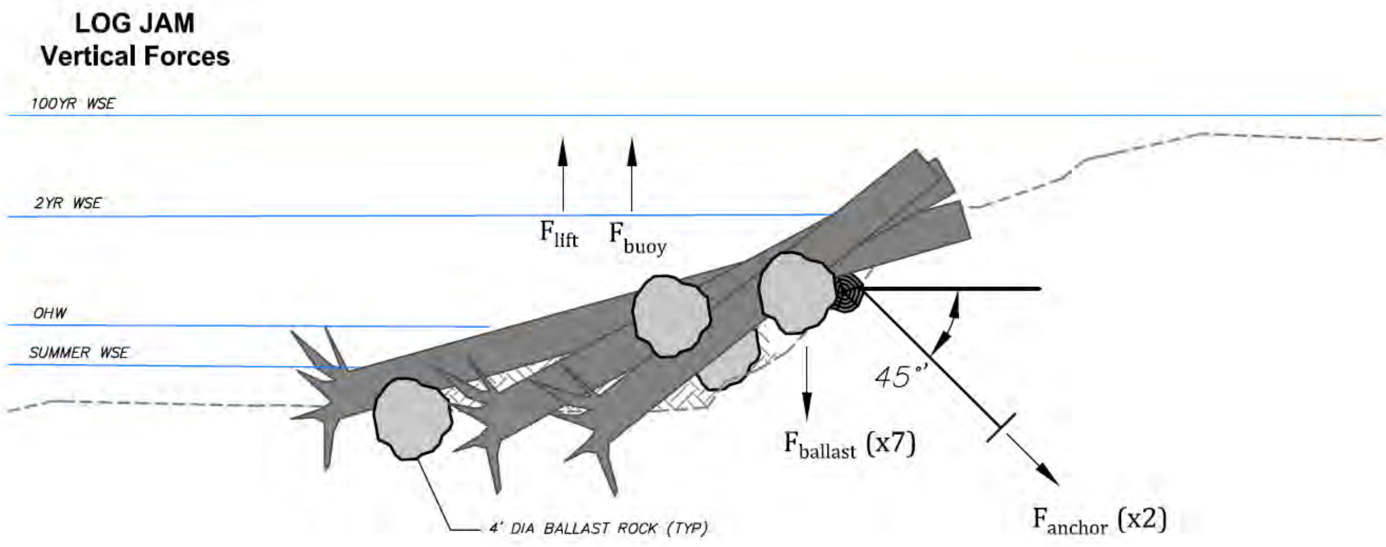
Safety Factor (Ballast Only)

$$F_{safetyH} := \left| \frac{F_{friction} + F_{gh} + 0}{F_{drag}} \right| = 5.49$$

Rotational Moment

$$\left(-F_{drag}\right) \cdot 15\ ft + \left(-F_{friction}\right) \cdot 10\ ft + \frac{F_{ah}}{2} \cdot 3\ ft + \frac{F_{ah}}{2} \cdot 10\ ft = 52920\ lbf\ ft$$

Type 2 (Logjam #2-6)



Vertical Force

Area of Structure Project into Flow

$A := 150\ ft^2$

Depth Average Approach Vel (from SRH2D)

$U_0 := 4.5 \frac{ft}{s}$

Bouyancy Force of Woody Structure

$Fb(D, L) := (\gamma_{wood} - \gamma_{water}) \cdot \frac{D^2}{4} \cdot \pi \cdot L$

$C_L := 1$

Rock Ballast Weight

$F_{rock}(D) := \frac{4}{3} \cdot \pi \cdot \left(\frac{D}{2}\right)^3 \cdot (\gamma_{rock} - \gamma_{water})$

Lift Force of Woody Structure

$$F_{lift} := (-1) \cdot \frac{C_L \cdot A \cdot \gamma_{water} \cdot U_0^2}{2 \cdot 32.2 \frac{ft}{s^2}} = -2943\ lbf$$

4x log each 35,30,28,22 inch Root BDH and ft long, 1x logs each 24 inch BDH and 25 ft long

$Fb_{struct} := (Fb(24\ in, 28.75\ ft) + Fb(2\ in, 2\ ft) \cdot 6) \cdot 4 + Fb(24\ in, 15\ ft) \cdot 1 = -15314\ lbf$

Ballast Weight

$F_{ballast} := F_{rock}(4\ ft) \cdot 7 = 22894\ lbf$

$AnchorLoad := 10000 \cdot 2\ lbf$ $\quad AnchorAngle := 45\ °$

Geotechnical Restraint (bury log, etc)

$F_{gv} := 0\ lbf$

Anchor Restraint

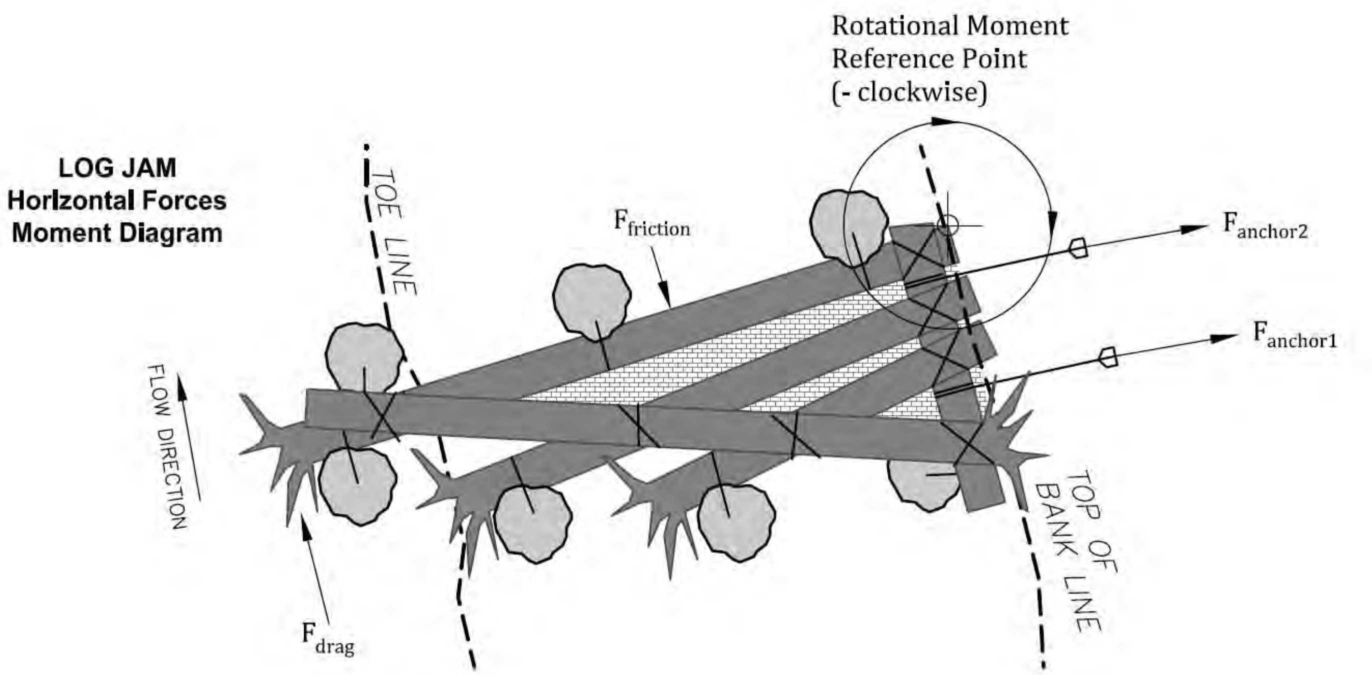
$F_{av} := AnchorLoad \cdot \sin(AnchorAngle) = 14142.1356\ lbf$

Safety Factor (Ballast & Soil Anchor)

$$F_{safetyV} := \left| \frac{F_{ballast} + F_{gv} + F_{av}}{Fb_{struct} + F_{lift}} \right| = 2.03$$

Safety Factor (Ballast Only)

$$F_{safetyV} := \left| \frac{F_{ballast} + F_{gv} + 0}{Fb_{struct} + F_{lift}} \right| = 1.25$$



Horizontal Force

$\mu_{bed} := \tan(30\,°)$

$F_{normal} := Fb_{struct} + F_{lift} + F_{ballast} = 4637\ lbf$

Friction Force:

$F_{friction} := \mu_{bed} \cdot F_{normal} = 2677\ lbf$

$C_D := 0.8$

Drag Force

$$F_{drag} := \frac{C_D \cdot A \cdot \gamma_{water} \cdot U_0^{\,2}}{2 \cdot 32.2 \dfrac{ft}{s^2}} = 2355\ lbf$$

Geotechnical Restraint (bury log, etc)

$F_{gh} := 0\ lbf$

Anchor Restraint

$F_{ah} := AnchorLoad \cdot \cos(AnchorAngle) = \dfrac{20000}{\sqrt{2}}\ lbf$

Safety Factor (Ballast and Soil Anchor)

$$F_{safetyH} := \left|\frac{F_{friction} + F_{gh} + F_{ah}}{F_{drag}}\right| = 7.14$$

Safety Factor (Ballast Only)

$$F_{safetyH} := \left|\frac{F_{friction} + F_{gh} + 0}{F_{drag}}\right| = 1.14$$

Rotational Moment

$$(-F_{drag}) \cdot 35\ ft + (-F_{friction}) \cdot 10\ ft + \frac{F_{ah}}{2} \cdot 5\ ft + \frac{F_{ah}}{2} \cdot 12\ ft = 11025\ lbf\ ft$$