APPENDIX C
Pearson Eddy Wetland Restoration Environmental Assessment

Hydrology and Hydraulics Report

1. Pearson's Eddy WRP Hydraulic Analysis; dated March 6th, 2009, by Tetra Tech

2. *Beginning page 128*: Hydraulic Impact Analysis of WRP project features along Pearson Eddy Slough, ADDENDUM, two-dimensional hydraulic analysis, dated March 2017, by Daniel Moore, P.E, Hydraulic Engineer.

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Pearson's Eddy WRP Hydraulic Analysis

Snohomish County, WA

Final Report
March 6, 2009

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Snohomish County, WA

Pearson's Eddy WRP Hydraulic Analysis – Final Report
March 6, 2009

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USDA Natural Resource Conservation Service
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Appendix B Pearson's Eddy Project Survey (2008)

1 Study Objectives and Scope

This report summarizes the hydraulic analysis of the Pearson's Eddy Slough in Snohomish County, Washington. The objective of this analysis is to compare alternatives for removal and/or replacement of tide gate structures on the slough in order to support planning decisions for wetland restoration at the Pearson's Eddy site.

The scope of the study included: site investigation and data collection, hydrologic analysis, and hydraulic analysis and inundation mapping of the existing condition and proposed alternative projects.

This analysis evaluates the potential effects of modifications to the floodgate structures on upstream and/or adjacent landowners. Increased water depth and quantities on the mitigation bank property are not a concern, however increased water depth, quantity, or duration of flooding is a concern for other adjacent or upstream properties.

This report briefly describes the project site, describes the technical approach for the hydrologic and hydraulic analyses, and summarizes the results of the analysis.

2 Project Description

This section describes the project site and briefly summarizes the previous studies and available data used in completing the present analysis.

2.1 Site Description

The Pearson's Eddy project site is located approximately 3.25 miles north of the City of Duvall, Washington along the Snoqualmie River at its confluence with Pearson's Eddy (at approximately River Mile 4.4). The site comprises former agricultural land in the Snoqualmie River Valley. Figure 1 provides a vicinity map of the project site. Snohomish County property records indicate that conservation organizations currently own approximately 488 acres in the vicinity of Pearson's Eddy Slough.

The site is relatively flat and is drained by a network of ditches and drainage tile, the largest of which we are calling the Pearson's Eddy Slough. Figure 2 illustrates the major features of the project site. Pearson's Eddy Slough runs south to north through the project site toward the Snoqualmie River, and has an average longitudinal slope of approximately 0.07 percent. The typical cross-section of Pearson's Eddy Slough in the project study reach is 40-70 feet wide and 8-10 feet deep. The project study area encompasses approximately a 1.5-mile reach of the slough, and features two existing crossings. The North Crossing includes three 60-inch corrugated metal pipes with flap gates. In addition to these pipes, there is an abandoned pump outlet measuring 48 inches in diameter. The South Crossing comprises three pipes, measuring 114 inches, 54 inches, and 48 inches, respectively.

The proposed project would remove the north (downstream) crossing, replace the south (upstream) crossing, place self regulating flood gates at a location to be determined, and enhance the existing swales in the field to reestablish connectivity with the channel and high flow inundation.

This study considers three alternatives:

- Remove the North Crossing flood gates and replace the South Crossing with a bridge.

- Remove the North Crossing flood gates and replace the South Crossing with a self-regulating flood gate structure with an access road on top of it.
- Remove the North Crossing flood gates, replace the South Crossing with a bridge, and install flood gates at the upstream end of the mitigation bank property.

Section 3.3 describes these alternatives in more detail.

2.2 Previous Studies and Available Data

Several previous studies and data were available for use or reference in completing the current analysis at Pearson's Eddy. This section briefly summarizes the data sources used in the analysis.

2.2.1 Flood Insurance Study

King County and Snohomish County recently completed an update of the FEMA Flood Insurance Study (FIS) for the Snoqualmie, Skykomish, and Snohomish Rivers (FEMA, 2007). The FIS analysis applied the U.S. Army Corps of Engineers' Hydrologic Engineering Center's River Analysis System (HEC-RAS) to simulate the hydraulic conditions of the Snoqualmie/Skykomish/Snohomish River system. The models were calibrated to gauge records at several points throughout the river system. Though it is not one of the calibrated reaches, the Pearson's Eddy Slough is included in the FIS hydraulic analysis as "Overflow Channel 1." The FIS addresses only large (10-year, 50-year, 100-year, 500-year) flood events on the major river system, and therefore the hydraulic simulations do not include culverts or gates on the Pearson's Eddy Slough.

2.2.2 King County's Tuck Creek Hydrology Study

King County developed an HSPF (Hydrologic Simulation Program FORTRAN) hydrologic model of the Tuck Creek basin in 1999 (King County, 1999). Figure 3 illustrates the location and size of the Tuck Creek watershed. The model, based upon regional parameters, simulated stream discharge on Tuck Creek from 1949 to 1997 using precipitation data scaled from Carnation, WA. The Tuck Creek HSPF model uses regional hydrologic parameters (generally correlated to land use), and was not calibrated to stream flow or stage data. The FIS used this model (with some modifications, including precipitation data supplemented to include WY 2004) to approximate flows in the "Overflow Channel" along the west side of the Snoqualmie River valley during large flood events (Hartley, 2008).

2.2.3 Survey Data

The FIS hydraulic model geometry comprises in-channel bathymetry from field surveys for areas below the waterline, which was merged with floodplain data from 2-ft contour LiDAR mapping completed in 2004. Ducks Unlimited commissioned a topographic survey of properties including the project site, and captured invert elevations of most of the culverts and other structures on the Pearson's Eddy Slough. The current project effort included additional survey to confirm existing geometry and gather additional cross-sections for the hydraulic model. The FIS topographic data, Ducks Unlimited Survey, and current project surveys all utilized the same survey benchmarks (set by the Washington Department of Transportation), and a comparison of the data confirmed their consistency

with one another. Section 3.2.1 describes the project survey and modifications to the FIS hydraulic model geometry in more detail.

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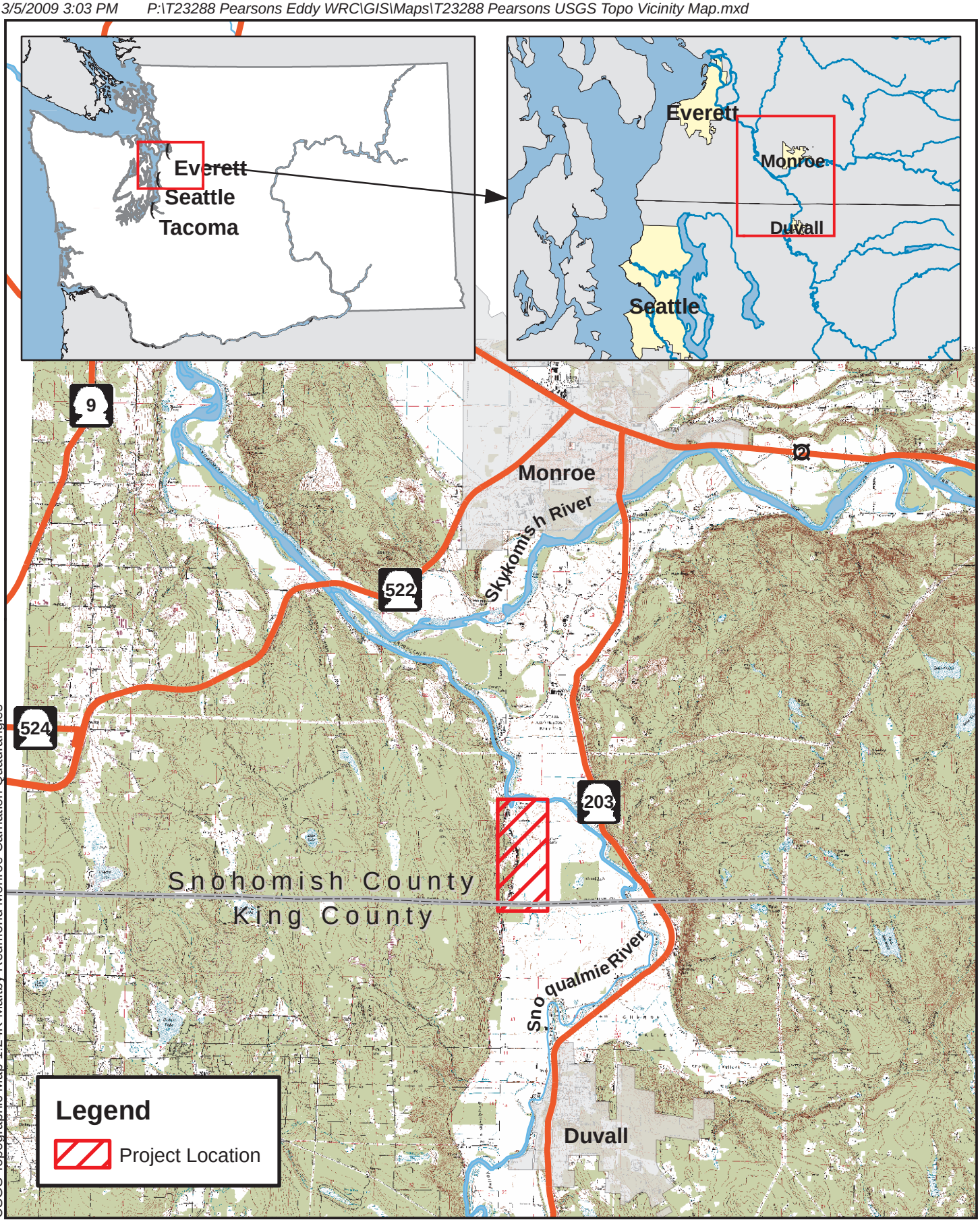

0 0.5 1 2 Miles

Pearsons Eddy WRC

Project Vicinity Map

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FIGURE 1

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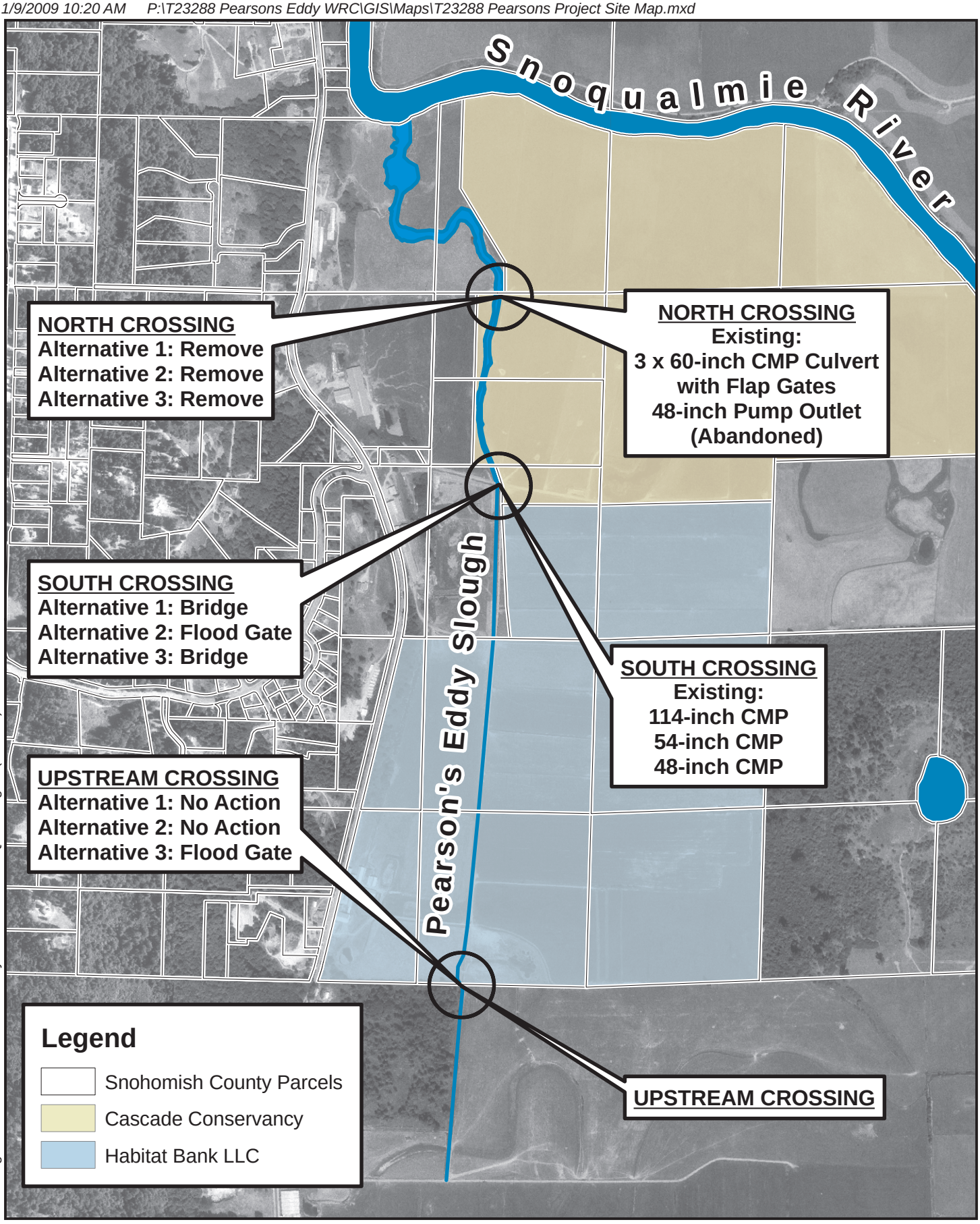

USGS Digital Ortho Photos 1:24K Maltby SE Monroe SW Quadrangles (c. 1990)

0 500 1,000 2,000 Feet

Pearsons Eddy WRC

Project Site Map

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FIGURE 2

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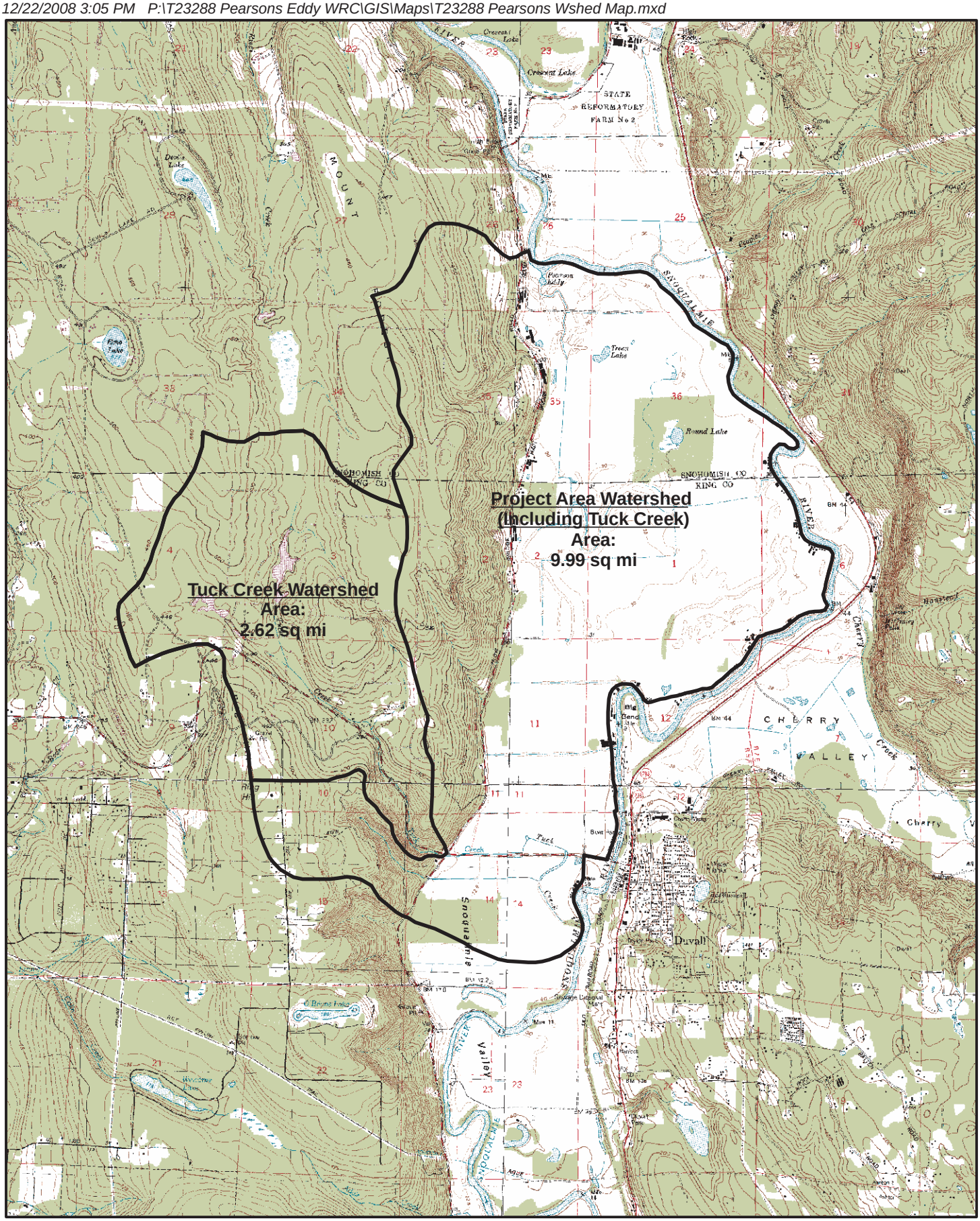

4,000 2,000 0 4,000 Feet

Pearsons Eddy WRC

Watershed Map

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FIGURE 3

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3 Technical Approach (Methods)

This section discusses the technical approach, including methods and assumptions, used in completing the Pearson's Eddy Slough alternative analysis.

3.1 Hydrology

This section described the methods applied to evaluate the hydrology of the Pearson's Eddy Slough site. The project team considered regional flood events and local flood events at Pearson's Eddy Slough in the analysis.

3.1.1 Regional Flood Events

Pearson's Eddy Slough is highly influenced by conditions on the Snoqualmie River during large, regional flood events. The FIS completed a detailed analysis of such events. In their memorandum to King County, Northwest Hydraulic Consultants (2004) described the approach used to evaluate extreme regional flood events on the Snoqualmie/Skykomish/Snohomish River system. In summary, the FIS approach used "production runs" simulating over a dozen historical flood events, which were used to characterize the flood frequency and calibrate water surface profile conditions on the Snoqualmie/Skykomish/Snohomish River system. The analysis used curves developed from these production runs to interpolate specific 10-year, 50-year, and 100-year regional flood events for the purposes of developing the regulatory flood plain. The hydrographs for these events were scaled from "template" historical events of similar magnitude.

Rather than replicate all the production runs and frequency analysis from the FIS, the project team chose the 10-year design event and two historical events that correlated most closely to the return periods under consideration (5-year and 25-year) to describe the hydrology for the Pearson's Eddy Slough hydraulic models.

Table 1. Selected historic events representing return periods under consideration.

Event	Date	Calculated Return Period	Peak Flow on Snoqualmie River near Carnation WA
		(years)	(cfs)
5-year	December 1977	5.0	47,600
10-year	December 1975	10.6	52,100
25-year	November 1995	23.9	61,200

3.1.2 Local Flood Events

In addition to regional flood events, Pearson's Eddy Slough may also experience local flood events, which may or may not coincide with regional flood events on the Snoqualmie/Skykomish/Snohomish River main stems.

The first alternative for describing local hydrology was to use flows directly from the FIS hydraulic simulations. The FIS hydraulic models use flow time series matching the time window of the regional peak flow event simulations, which were extracted from a continuous simulation HSPF model (which spanned 1947-2002) representing the watershed of Tuck Creek and nearby tributaries (named "Tuck Creek + 8" in the FIS documentation). In using these data, the Pearson's Eddy Slough analysis would assume

that peak flow events on the slough are directly correlated to the regional flow frequency curve.

Plotting the peak flows from the FIS hydraulic model's "Overflow Channel 1" (i.e., Pearson's Eddy Slough) against corresponding peak flows on the Snoqualmie River (Figure 4) demonstrated that local peak flows found in the FIS model do not necessarily match regional flow frequency relationships. In other words, the FIS hydrology could not reasonably be used as a basis for a flow-frequency relationship on Pearson's Eddy Slough. Thus, the project team rejected this method for describing local hydrology.

A second method followed the approach of the FIS, and scaled the results of King County's Tuck Creek analysis (1999) to represent all the catchments flowing to Pearson's Eddy Slough. Like the FIS approach, this approach simulates conditions where flows in the Snoqualmie River are high enough to close the flap gates at Tuck Creek but not high enough to overtop any levees or the highway to fill the valley – that is to say at least a "bank full" condition on the Snoqualmie River. In such a scenario, flows from Tuck Creek and other local streams are prevented from entering the Snoqualmie River main stem and are directed down-valley, primarily through Pearson's Eddy Slough.

Figure 3 illustrates the relative size of the Tuck Creek and Pearson's Eddy Slough watershed areas. This approach took the HSPF results from the Tuck Creek study (approximately 2.6 square miles) and scaled up for the area that would drain through Pearson's Eddy Slough (approximately 10.0 square miles) using the following relationship:

$$Q_2 = \left(\frac{A_2}{A_1} \right)^b$$

Where Q_2 and A_2 were the peak flow (in cubic feet per second) and watershed area (in square miles) of Pearson's Eddy Slough, respectively, and Q_1 and A_1 were the peak flow and watershed area from the Tuck Creek study. The scaling factor exponent b=0.98 was as prescribed by the U.S.G.S. regional regression method (U.S.G.S., 1997).

In order to confirm the reasonableness of this approach, the project team compared the scaled HSPF model peak flows with those calculated by the U.S.G.S. regional regression method. Table 2 and Figure 5 summarize the results of this analysis. The scaled HSPF peak flows plot very closely to the peak flows predicted by the U.S.G.S. regional regression equation, and well within the standard error (illustrated by the error bars on the plot) prescribed by the U.S.G.S method. Therefore, the project team concluded that scaling the results of the Tuck Creek HSPF model would adequately represent local flows at Pearson's Eddy Slough, and adequate for comparing the relative performance of the alternatives in this study. But because the hydrology is scaled up from an uncalibrated model of a smaller watershed, the results should be used with caution.

3.1.1 Spring Hydrology

A period of critical hydrologic and hydraulic conditions in Pearson's Eddy and its vicinity occurs commonly during spring months when high water levels in the Snoqualmie River 'back up' into the slough. The high water levels in the Snoqualmie are

the result of snowmelt events and are commonly not coincident with rainfall runoff events in upstream tributaries of Pearson's Eddy. They also occur relatively frequently, roughly annual events on average. These high water levels are problematic because they are enough to cause flooding of neighboring properties.

Table 2. Comparison of peak flow calculated by U.S.G.S. regression equations and by scaling up results of Tuck Creek HSPF analysis.

Return Period	Peak Flow by U.S.G.S. Regression Equation	Peak Flow by Scaled HSPF Model Results	% Difference from U.S.G.S. Regression Method
(years)	(cfs)	(cfs)	
1	n/a	48	n/a
2	233	208	-11%
5	n/a	327	n/a
10	413	412	0%
25	507	520	3%
50	596	598	0%
100	665	680	2%

Time-varying stage and inflow input data were developed to represent these frequent events in the model. Stages in the Snoqualmie River at the confluence with Pearson's Eddy (Snoqualmie RS 2.94) were extracted from the FIS model results. Output from one of the FIS production runs (Plan 02, February 1972) was used because this run represented approximately a 3-year event, one of the smaller and more frequent events considered in the FIS. Figure 6 illustrates the stage hydrograph from this scenario (dashed blue line).

After selecting a representative FIS scenario, the next step was to 'scale-down' the 3-year event stages because they were still well in excess of the general tops of bank in Pearson's Eddy. The FIS results show stages above 35 feet NAVD88 for several days during this scenario, and the tops of bank in Pearson's Eddy generally vary from 28 to 35 feet NAVD88 in the lower sections of the channel. To better approximate more frequent and slightly lower stages in the river, and thus at the mouth of the slough, the stages were scaled down as shown in Figure 6 (solid blue line). This scaled curve was used to represents the time-varying downstream stage boundary condition of the unsteady flow model of the spring hydrology conditions.

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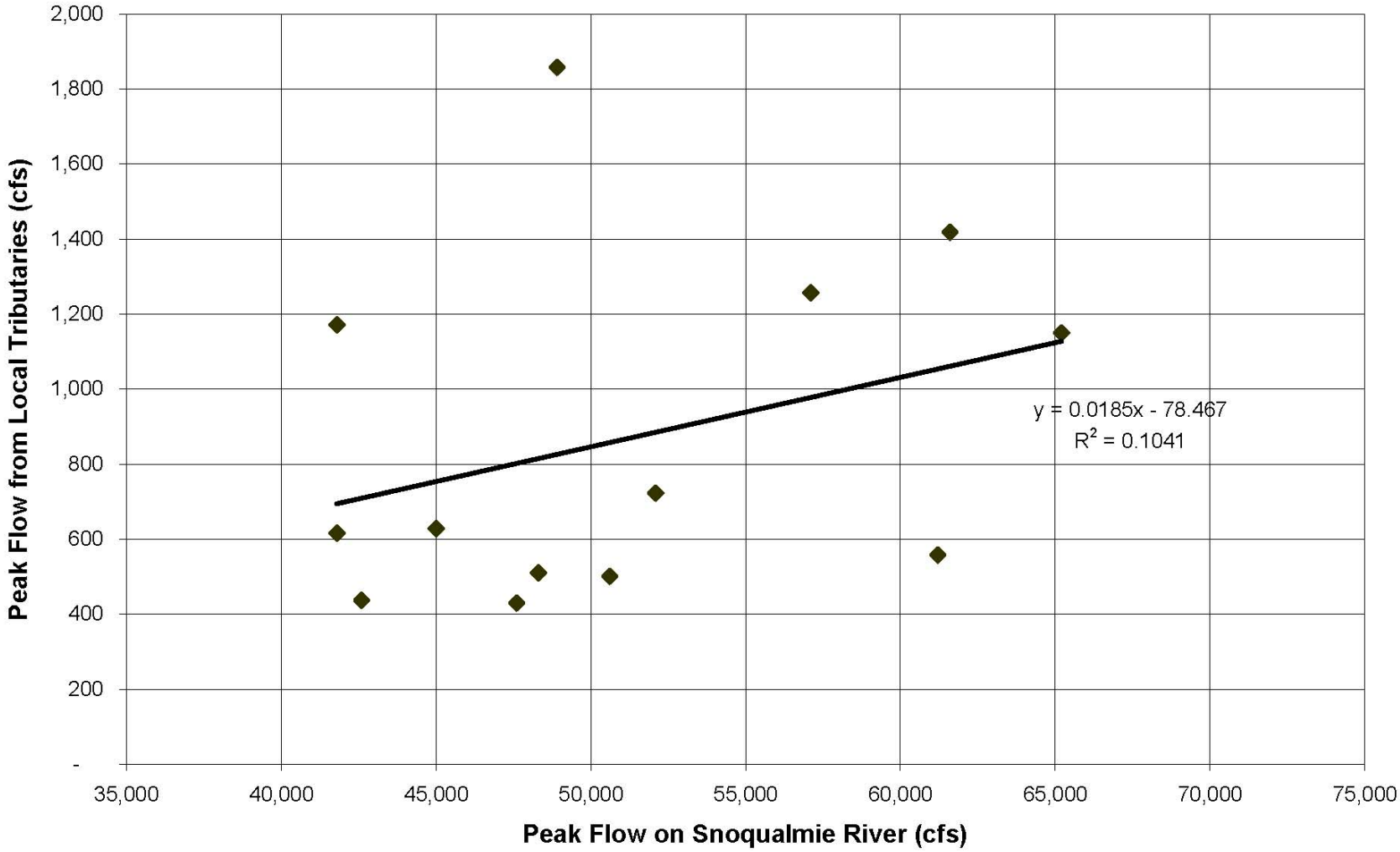

Figure 4. Correlation of flows on Snoqualmie River and "Tuck Creek + 8" data from FIS model.

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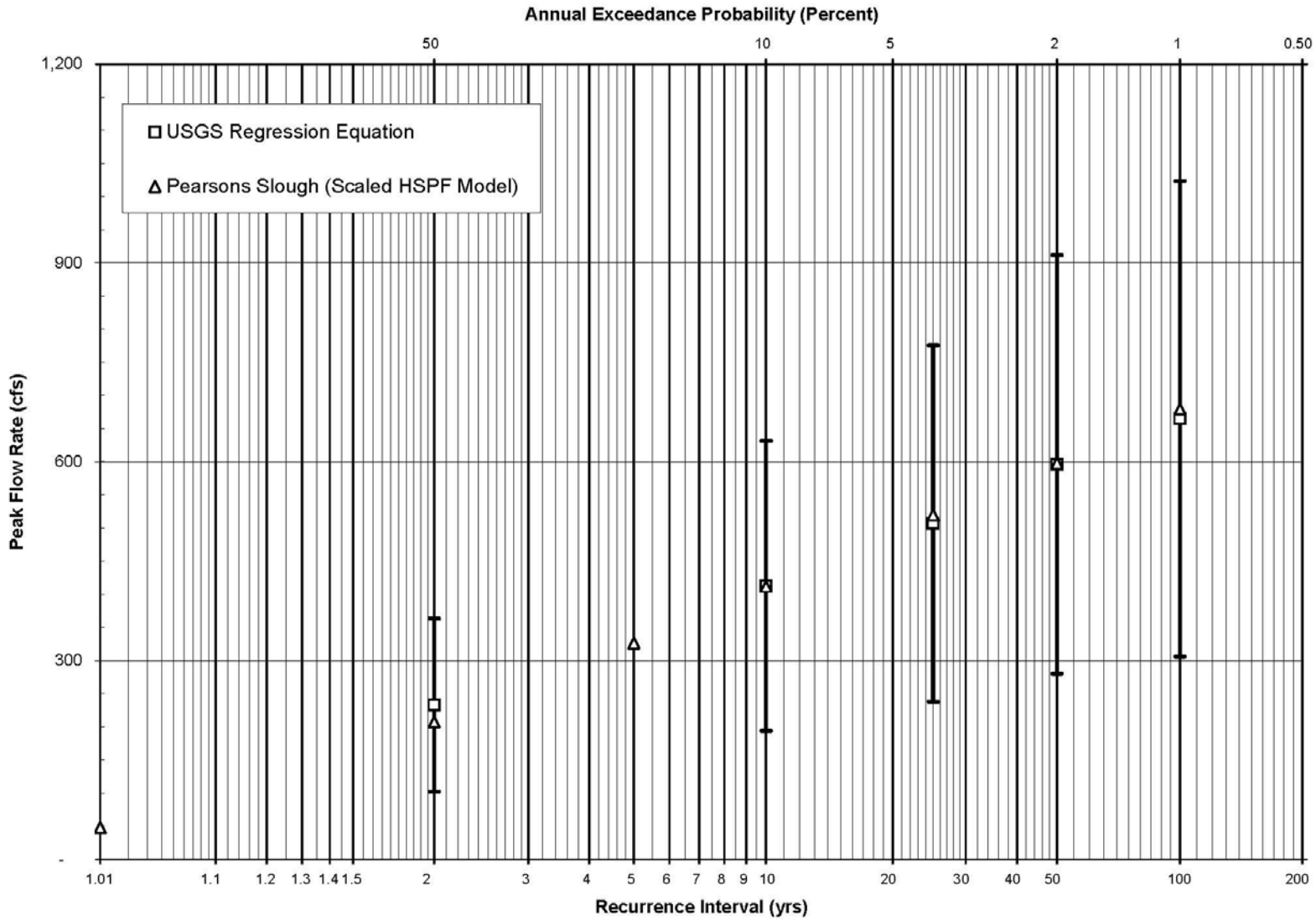

Figure 5. Calculated annual peak stream flow, with whisker plot illustrating standard error of U.S.G.S. regression method, for Pearson's Eddy Slough, Snohomish County WA (Extreme Value Type I Plot).

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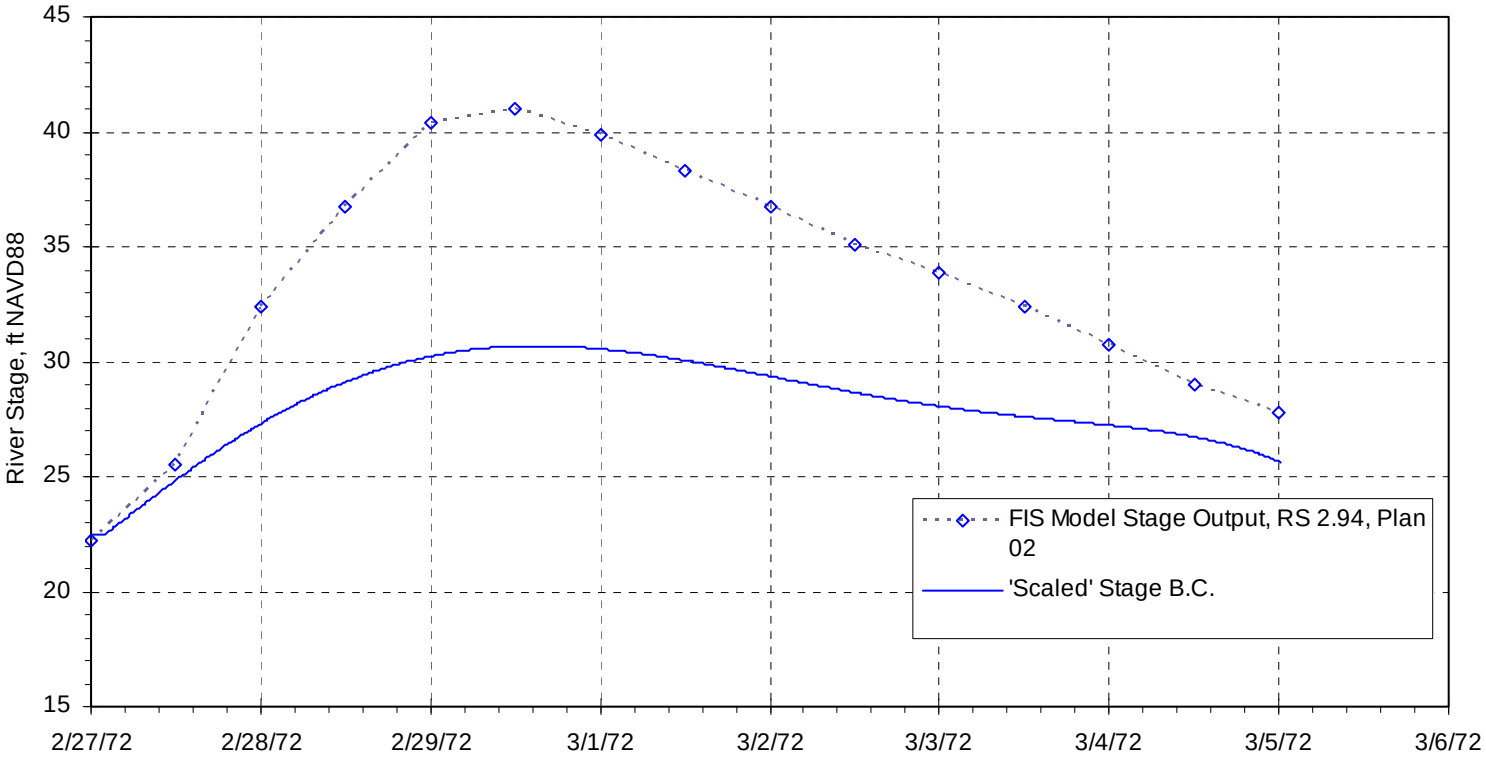

Figure 6. Stages in the Snoqualmie River (FIS Model Plan 02, February 1972 Production Run), and scaled stages used to represent spring high stage conditions in Pearson's Eddy.

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3.2 Hydraulics

The analysis applied HEC-RAS (USACE, 2005) to simulate and compare the hydraulic conditions at the project site. This section discusses the method and major assumptions involved in the hydraulic model effort. This section discusses the methods and major assumptions involved in the hydraulic modeling effort.

3.2.1 Geometry

The analysis used the geometry from the existing FIS model geometry as a based for construction of the project hydraulic models. The FIS geometry is a combination of channel bathymetry for main stems and major tributaries of the Snoqulamie/Skykomish/Snohomish system and LiDAR data flown in 2004. Due to computational limitations, the FIS hydraulic models were split into north and south. components. The north hydraulic model included the Pearsons Eddy Slough as "Overflow Channel 1." The FIS hydraulic models do not include culverts or gates on the Pearson's Eddy Slough.

The analysis modified the FIS hydraulic model to include the culverts and structures on Pearson's Eddy Slough. This more detailed existing condition model provided the basis for the subsequent alternative analysis.

- Addition of cross-sections from the project survey;
- Substitution of more detailed cross-section geometry for the Pearson's Eddy Slough from the project survey; and
- Addition of culvert and hydraulic structure geometry.

3.2.1.1 Verification of Channel Survey

A survey of the Pearson's Eddy Slough was conducted in October 2008. The purpose of the survey was to update previous topographic surveys and provide better definition of the slough channel. The survey also provided adequate topographic definition of the two road crossings so that the existing culverts could be added to the model. The survey recorded main channel features such as tops and toes of bank and generally ranged from 200 feet to 400 feet across the channel into the overbank sections.

The locations of each transect and cross section plots are included in Appendix B. The survey included 17 transects along approximately 8,000 feet of the slough. Table 3 summarizes the surveyed cross sections and their correlation with the existing FIS model cross sections.

3.2.1.2 Merging Cross Section Data

The previous FIS model includes detailed topography over the large floodplain between Pearson's Eddy and the Snoqualmie River, while the recent survey includes high definition within Pearson's Eddy channel. Consequently, these two sections required merging to form an accurate representation of both the floodplain and channel. These two sections were combined by plotting, overlaying and aligning, and then clipping the appropriate sections to form a single, continuous cross-section.

Figure 8 illustrates an example of this procedure. The dashed gray line is river station 0.16 from the FIS model. The thick red line shows cross section "OF1-B" from the recent channel survey. These two lines were overlaid and aligned to best fit. The thin blue line shows the resulting merger of the two sections.

Table 3. Correlation between the recent survey cross sections and the existing FIS model cross sections (ordered upstream to downstream).

No.	Survey XS Name	Distance to Downstream XS	Existing FIS River Station	Notes
		(ft)		
1	OF1-J	65	Na	Section adjacent to "Cul Prop," not used
2	CUL PROP	580	1.52	
3	OF1-I	505	1.33	Labeled "XS OF1-EYE"
4	OF1-H	720	1.21	
5	OF1-G	690	1.08	
6	OF1-F	740	0.92	
7	OF1-E	410	0.79	
8	U/S of S. Culv.	640	0.71	Upstream face of south culvert x-ing
9	S. Culvert	640	NA	Road x-ing
10	D/S of S. Culv.	0	.705*	Downstream face of south culvert x-ing
11	OF1-D	860	0.62	
--	--	--	0.525	No survey section at RS .525
12	U/S of N. Culv.	50	0.44	Upstream face of north culvert x-ing
13	N. Culvert	0	NA	Road x-ing
14	D/S of N. Cul.	305	0.43	Downstream face of north culvert x-ing
15	OF1-B1	1020	--	No FIS model section here
16	OF1-B	585	0.16	
17	OF1-A	315	0.06	

3.2.1.3 Channel and Floodplain Storage Volume

The project team conducted a comparative analysis of channel storage volume in order to better understand flow volumes of storm events relative to the geometric volume of Pearson's Eddy Slough and the adjacent floodplain region. The analysis was intended to indicate the effectiveness of the Pearson's Eddy Slough in containing or conveying a particular flood event, as well as the effects of restoration actions on flood storage volumes in the channel.

The channel and floodplain volume analysis was an average-end-area calculation based on merged cross section geometry (as described in Section 3.2.1.2) and the downstream reach distance of each cross section (i.e., the geometry used in the hydraulic model). The resulting stage versus volume curves were broken down by reach, as described in Table 4. The analysis included the channel and floodplain only up to River Station 1.92 primarily because the Pearson's Eddy channel is poorly defined and relatively small upstream of this location.

Table 4. Channel and floodplain storage volume analysis reaches.

Reach	Downstream River Station	Upstream River Station	Description
1. North Reach	0.00	0.43	From confluence with the Snoqualmie River to the North Crossing
2. Middle Reach – North	0.43	0.71	Between the North Crossing and South Crossing
3. Middle Reach – South	0.71	1.52	Between South Crossing and proposed Upstream Crossing
4. South Reach	1.52	1.92	Upstream of the proposed upstream crossing

The analysis considered two cases: (1) the main Pearson's channel (no overbank volume) and (2) the volume available in the entire cross section (channel plus floodplain). Figure 9 illustrates the resulting volume curves developed for the 'channel-only' case. Volumes for the four reaches are shown along with the total volume, which is the sum of those for the four reaches. This figure shows the variation in channel volume between 24 and 35 feet NAVD88. The total volume approaches 500 acre-feet at a stage of 35 feet NAVD88. At this same elevation, the total volumes of Reaches 1, 2, 3 and 4 are approximately 70, 10, 280, and 110 acre-feet, respectively. Reach 3 (Middle Reach – South) contains the highest channel volume of the individual reaches and comprises the majority of the total volume, especially at higher stages (those over 30 feet NAVD88).

Reach 2 (Middle Reach – North) has the smallest volume. At stage 32 feet NAVD88, it has only 7 acre-feet of volume. This may imply that restoration actions involving this reach may not have particularly significant impacts on upstream or downstream water surface elevations or flood storage potential.

Figure 10 illustrates the total volumes including both the channel and floodplain areas. The total volume up to a stage of 35 feet NAVD88 approaches 5,000 acre-feet. This is approximately 10 times the volume within the main Pearson's Eddy Slough channel alone. The total volumes for each reach are also approximately one order of magnitude larger than their respective 'channel-only' volume. This result is as expected given the relatively broad, low-lying floodplain between Pearson's Eddy and the Snoqualmie River.

3.2.2 Culvert Data

3.2.2.1 Steady State – Backwater Analysis

For the existing channel conditions, the North and South Crossings (culverts) were entered into the model via the HEC-RAS Bridge/Culvert and Culvert Data Editors. Figure 11 and Figure 12 illustrate the upstream and downstream elevation view schematics of the culverts. Table 5 summarizes culvert characteristics for the North and South Crossings. These culverts were used in the steady flow hydraulic analysis to simulate the backwater effects of high upstream inflow conditions within Pearson's Eddy.

3.2.2.2 Unsteady Flow – Spring Conditions Analysis

To represent conditions within the channel that occur commonly in the spring when stages in the Snoqualmie River are often higher than those in Pearson's Eddy, the project team developed simulations utilizing self-regulating flap gates. Flap gates open to allow

Table 5. Culvert summary characteristics – existing conditions.

NORTH CROSSING – RS 0.435					
No.	**Types**	**Size x Length**	**U/S Invert**	**D/S Invert**	**Notes**
			(ft NAVD 88)	**(ft NAVD 88)**	
1	Round CMP	60" Dia .x 60'	20.69	19.98	Flap on D/S End
2	Round CMP	60" Dia. x 55'	20.08	20.11	Flap on D/S End
3	Round CMP	60" Dia. x 55'	20.31	20.79	Flap on D/S End
4	Round CMP	48" Dia. x 60'	27.33	27.53	Pump Outlet Not Modeled (Assume No Conveyance)
NORTH CROSSING – FLAP GATE STRUCTURES USED IN UNSTEADY SIMULATION					
No.	**Types**	**Size x Length**	**U/S Invert**	**D/S Invert**	**Notes**
			(ft NAVD 88)	**(ft NAVD 88)**	
1	Rectangular	4' x 5'	20	n/a	Self-regulating
2	Rectangular	4' x 5'	20	n/a	Self-regulating
3	Rectangular	4' x 5'	20	n/a	Self-regulating
SOUTH CROSSING – RS 0.7075					
No.	**Types**	**Size x Length**	**U/S Invert**	**D/S Invert**	**Notes**
			(ft NAVD 88)	**(ft NAVD 88)**	
1	Round CMP	48" Dia. x 32'	22.19	22.72	Open culvert
2	Round CMP	114" Dia. x 30'	20.98	20.41	Open culvert
3	Round CMP	54" Dia. x 32'	22.13	22.02	Open culvert

Note: Culverts are listed west to east.

downstream flow when the upstream WSE is greater than the downstream WSE, and they close to prevent flow when the opposite WSE gradient (negative flow) occurs.

HEC-RAS simulates self-regulating flap gates across main conveyance channels as inline structures. The choice of inline structures in the model are limited and require flap gates that are rectangular in shape. Figure 13 illustrates the flap gates used to simulate the unsteady flow spring conditions. The figure shows three identical gates, each with a 5 foot rise by 4 foot span opening. These gates were sized to approximate flow areas of the existing 60-inch culverts that they represent. The flap gates utilized during the unsteady flow simulations are also summarized in Table 5.

3.3 Descriptions of Alternatives

The analysis considers the existing condition (see description, Section 2.1), plus three alternative configurations for removing and/or replacing the existing crossings on Pearson's Eddy Slough.

3.3.1 Culvert Design

The design proposes replacement culvert crossings that will meet minimum flow conveyance and fish passage design criteria. Washington NRCS Conservation Practice Standard 578 (NRCS, 2005) requires that culverts meet local codes or regulations, and at minimum be able convey the stream bank-full flow or the 2-year/24-hour peak discharge, whichever is less, and with a headwater depth not greater than 1.5 times the culvert diameter. In addition, culverts in fish-bearing streams must comply with Washington Administrative Code (WAC) 220-110-070 in regard to capacity, design flow velocities, and other design and construction requirements.

3.3.1.1 Local Design Criteria

Snohomish County currently requires that new conveyance systems have sufficient capacity to convey the peak flow from the 25-year storm event for smaller development activity and from the 100-year storm event for larger development activity. The Snohomish County standards define smaller development activity as activities that would create less than 5,000 square feet of new impervious surfaces, while larger development activity would create more than 5,000 square feet. Under this definition, proposed restoration activities at Pearson's Eddy should easily qualify as a smaller development activity.

Snohomish County requires that the maximum headwater depth for the design storm not exceed 1.5 times the culvert height for culverts greater than 18 inches, which matches Washington NRCS minimum design requirements. For bottomless culverts, Snohomish County requires that headwater depth of the 100-year storm shall not exceed the top of the culvert. While Snohomish County code prefers that pipes have a minimum slope of 0.5 percent, it allows pipes installed as water level equalizers, fish passages, and/or internal components of a detention/retention system to have a flatter slope.

3.3.1.2 Fish Passage Design Criteria

Washington Department of Fish and Wildlife (2003) offers several approaches to provide fish passable culvert design, including a hydraulic design approach, stream simulation, and zero-slope culvert design. The hydraulic design approach involves comparison of design flow velocity, flow depth, and hydraulic drop at a calculated fish passage design flow against certain threshold criteria. The design criteria for adult fish depend upon the target species and length of culvert, and are intended to provide passage conditions for the weakest and smallest individuals of each species. The stream simulation approach attempts to maintain or mimic natural stream conditions as through the culvert. A third approach no-slope (or zero-slope) approach assumes that if a large enough culvert is installed flat, the installation will allow the natural movement of bedload to form a stable bed inside the culvert and provide suitable fish passage conditions.

The preliminary culvert design analysis followed the zero-slope approach. This was for several reasons, first being the simplicity of implementation. Moreover, sloped culverts offer little conveyance advantage over zero-slope installations in the context of severe backwater conditions such as those found on Pearson's Eddy Slough. Zero-slope culvert installations are also generally preferred from a permitting perspective.

The zero-slope culvert approach assumes that culvert design meets fish-passage criteria and therefore does not require comparisons to fish passage velocity and depth criteria. Therefore, it was not strictly necessary to compute the fish passage design flows. But because the method (Powers and Saunders, 2002) is based upon U.S.G.S. regional regression equation methods (see Section 3.1.2), the additional calculation was very simple. Table 6 summarizes the fish passage design flows.

3.3.1.3 Flap Gates

The installation of flap gates at the proposed culvert crossings will also present fish passage design challenges. Flap gate hydraulics should be examined carefully to evaluate

Table 6 Fish passage design flows.

Design Event	Fish Passage Design Flow
	(cfs)
January	135
May	65

their effect upon upstream water-level fluctuations, water quality, and fish passage. WDFW (2003) discusses several issues that should be considered when proposing flap gates in fish-bearing streams. These considerations and recommendations include:

- *Gate Orientation*. Flap gates can be oriented so that the hinge is mounted vertically on the side of the gate to allow better fish passage.
- *Gate Materials*. Using lightweight materials can allow gates to open wider with lower head differential.
- *Gate Operators*. Gate operators (such as floats, latches, or other actuators) can help optimize gates for fish passage.
- *Gate Orifices*. In some cases, gates can include an orifice for fish passage without significantly compromising flood protection.
- *Parallel Gates*. Installing parallel gates at multiple elevations can enable gates to open wider and/or more frequently at low flows

Installing flap gates on a zero-slope culvert might also present additional challenges, since the zero-slope configuration encourages bed material deposition inside the culvert that could hinder effective flap gate operation. Unfortunately, these issues are beyond the scope of this preliminary analysis and will need to be addressed during future design development.

3.3.2 Alternative 1

Alternative 1 would remove the North Crossing flood gates and replace the South Crossing with a bridge. The primary advantage of this alternative would be its minimal construction cost, as well as the avoidance of flood gate operation and maintenance costs through the project life span.

Figure 14 and Figure 15 provide the elevation view of the North Crossing (now an open channel) and the bridge at the South Crossing, respectively. At the North Crossing, the culverts were removed from the model, leaving only the cross sections immediately upstream (not shown) and downstream (Figure 14). The South Crossing improvements for Alternative 1 were evaluated in the same way; the culverts were removed, and a basic bridge was placed across the channel from top of bank to top of bank (Figure 15). Table 7 provides the general dimensions of the resulting channels.

Table 7. General channel characteristics after culvert removal.

Location	River Station	Bottom Width	Top Width	Approx. Depth	Flow Area
		(ft)	(ft)	(ft)	(sq ft)
North Crossing	0.44	32.6	78.1	16.9	937.1
South Crossing	0.71	66.9	117.4	11.8	1084.6

3.3.3 Alternative 2

Alternative 2 would remove the North Crossing flood gates and replace the South Crossing with a self-regulating flood gate structure with an access road on top of it. Based on local conveyance requirements and fish passage criteria, the South Crossing flood gate will require a triple-barrel 4-ft rise by 5.5-ft span box culvert (Table 8). The flap gates on this crossing will allow flow the Snoqualmie River to inundate Pearson's Eddy Slough and portions of the floodplain north of the crossing, while preventing flow from propagating upstream (i.e., the Habitat Bank Property).

Table 8. Culvert summary characteristics – Alternative 2.

SOUTH CROSSING – RS 0.7075					
No.	**Type**	**Size x Length**	**U/S Invert**	**D/S Invert**	**Notes**
			(ft NAVD 88)	**(ft NAVD 88)**	
1	RCB	4' x 5.5' x 60'	20.0	20.0	Flap on D/S End
2	RCB	4' x 5.5' x 60'	20.0	20.0	Flap on D/S End
3	RCB	4' x 5.5' x 60'	20.0	20.0	Flap on D/S End

3.3.4 Alternative 3

Alternative 3 would remove the North Crossing flood gates, replace the South Crossing with a bridge, and install flood gates at the upstream end of the mitigation bank property. Based on local conveyance requirements and fish passage criteria, the Upstream Crossing flood gate will require a triple-barrel 4-ft rise by 5.5-ft span box culvert (Table 9). The flap gates on this crossing will allow flow the Snoqualmie River to inundate Pearson's Eddy Slough and portions of the floodplain north of the crossing, while preventing flow from propagating upstream (i.e., the property located in King County).

Table 9. Culvert summary characteristics – Alternative 2.

UPSTREAM CROSSING – RS 1.52					
No.	**Type**	**Size x Length**	**U/S Invert**	**D/S Invert**	**Notes**
			(ft NAVD 88)	**(ft NAVD 88)**	
1	RCB	4' x 5.5' x 60'	24.6	24.6	Flap on D/S End
2	RCB	4' x 5.5' x 60'	24.6	24.6	Flap on D/S End
3	RCB	4' x 5.5' x 60'	24.6	24.6	Flap on D/S End

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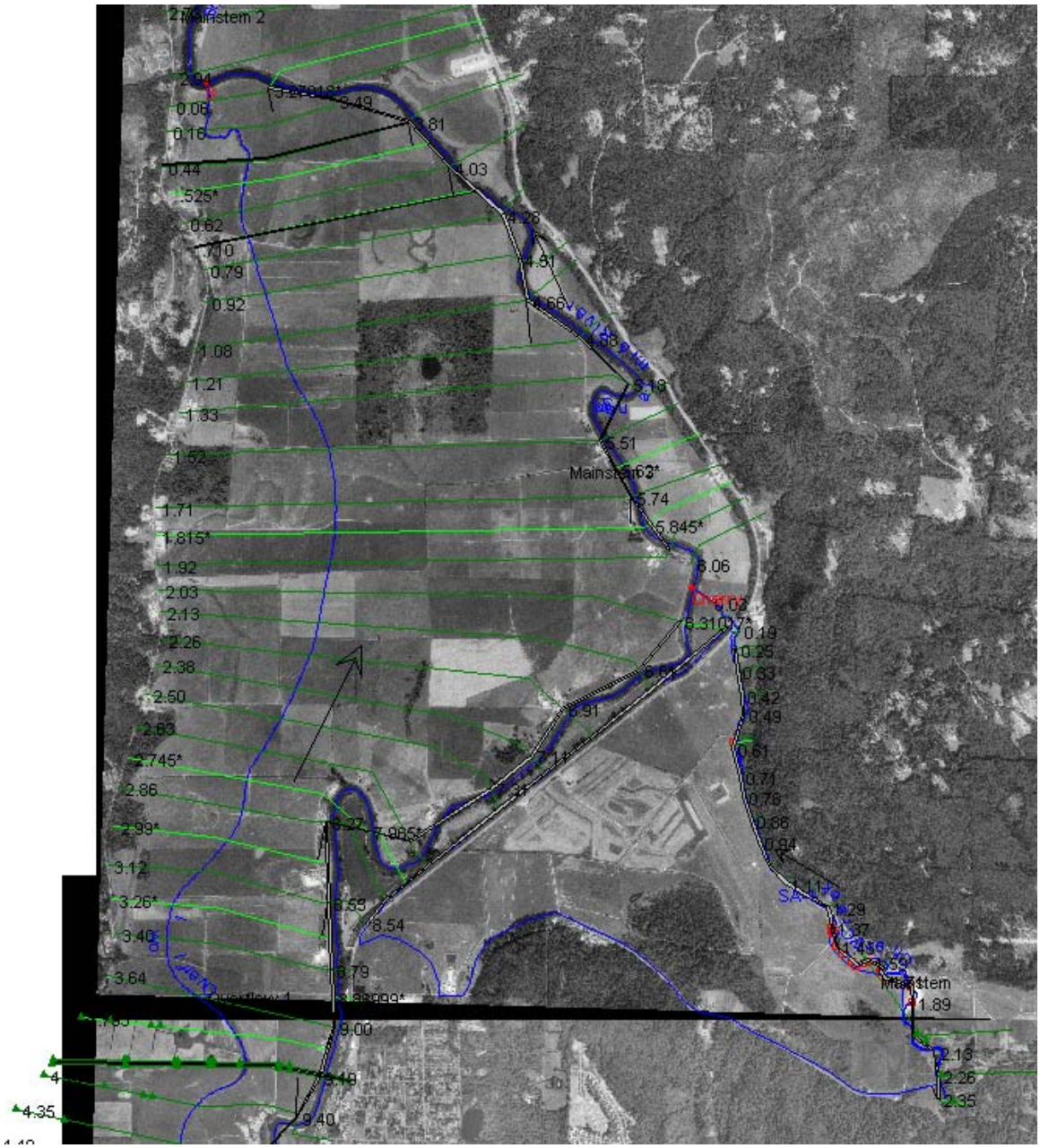

Figure 7. Schematic of the FIS HEC-RAS hydraulic model schematic showing Pearson's Eddy and floodplain adjacent to the Snoqualmie River. Note the reach "Overflow1" is Pearson's Eddy and flows south to north.

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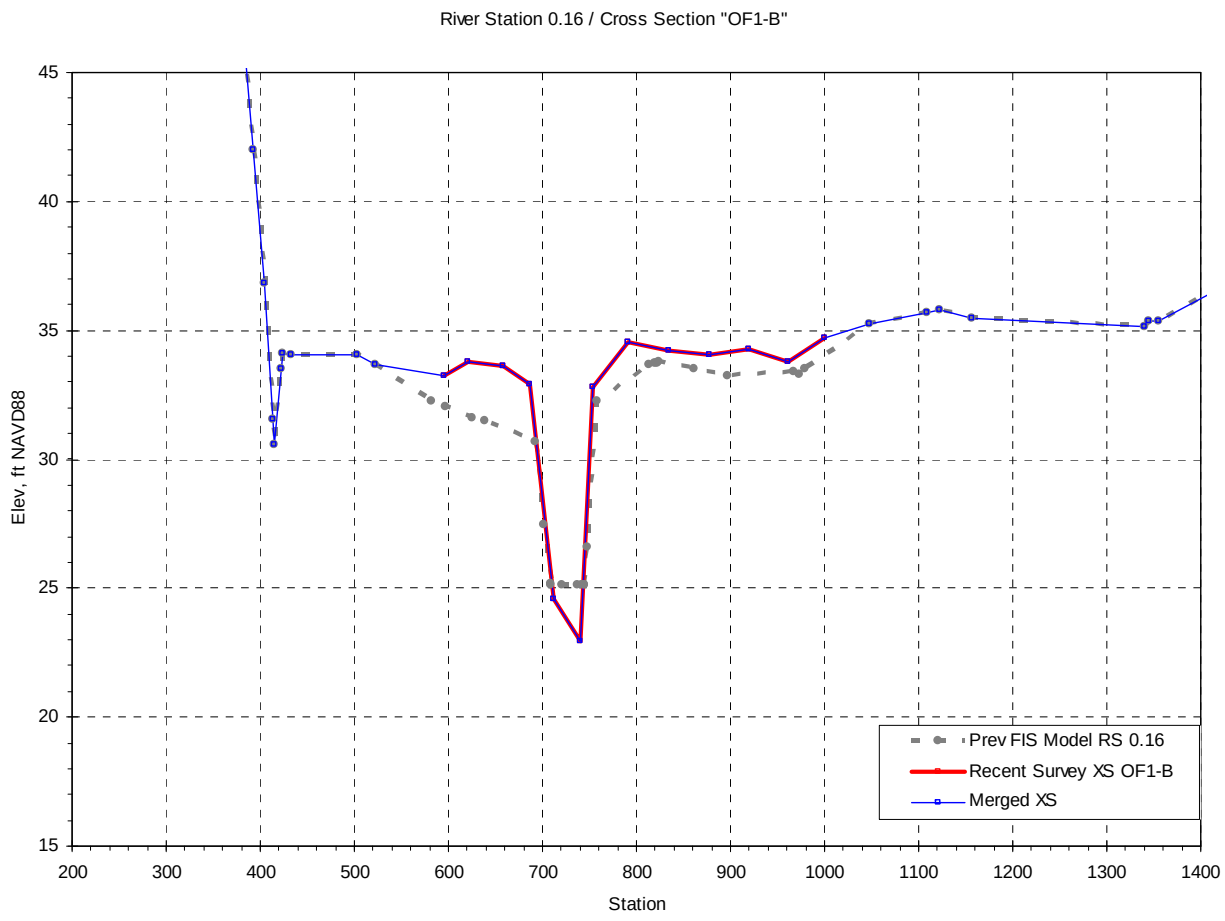

Figure 8. Plot showing merging of channel data (thick red line) with floodplain data (dashed gray line) to form a complete cross section (thin blue line) used in the updated hydraulic model.

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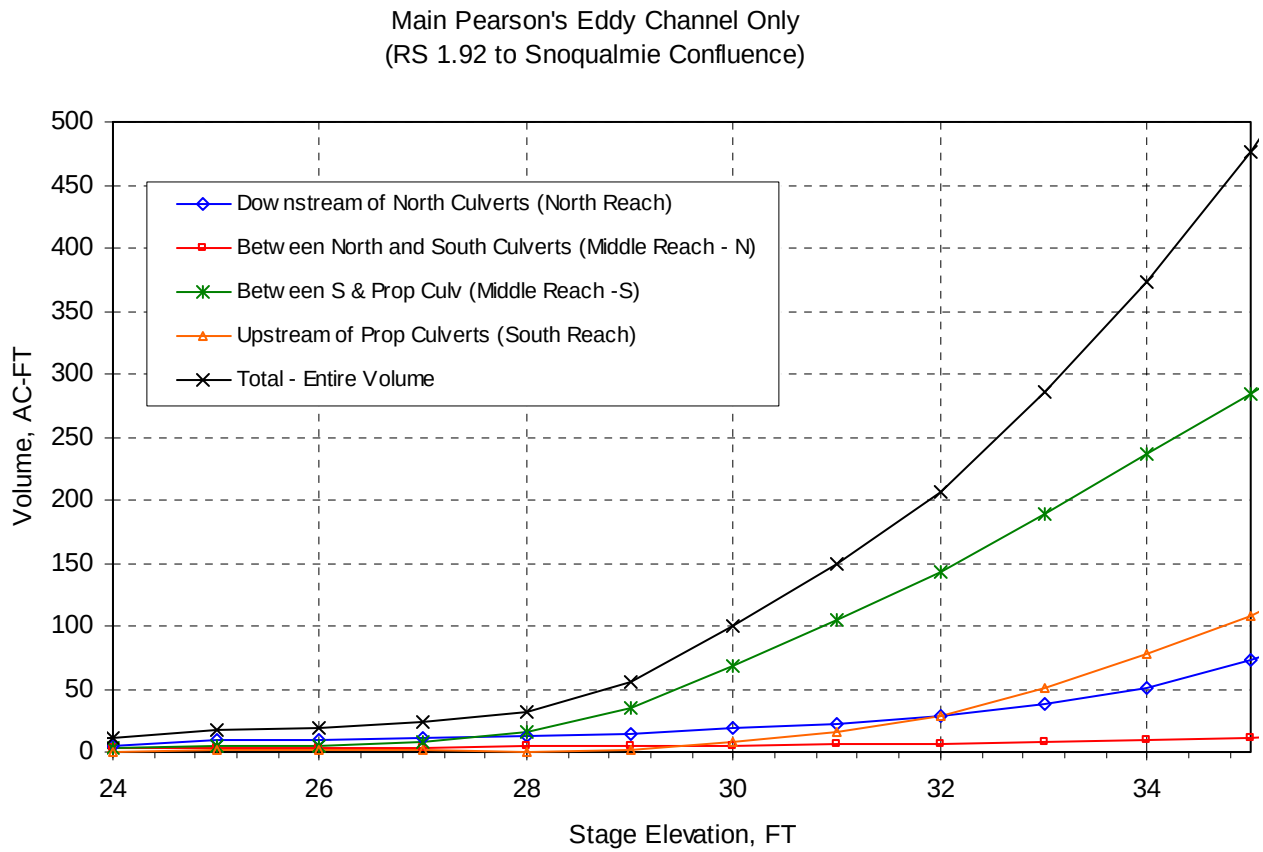

Figure 9. Total channel volume for Pearson's Eddy Slough.

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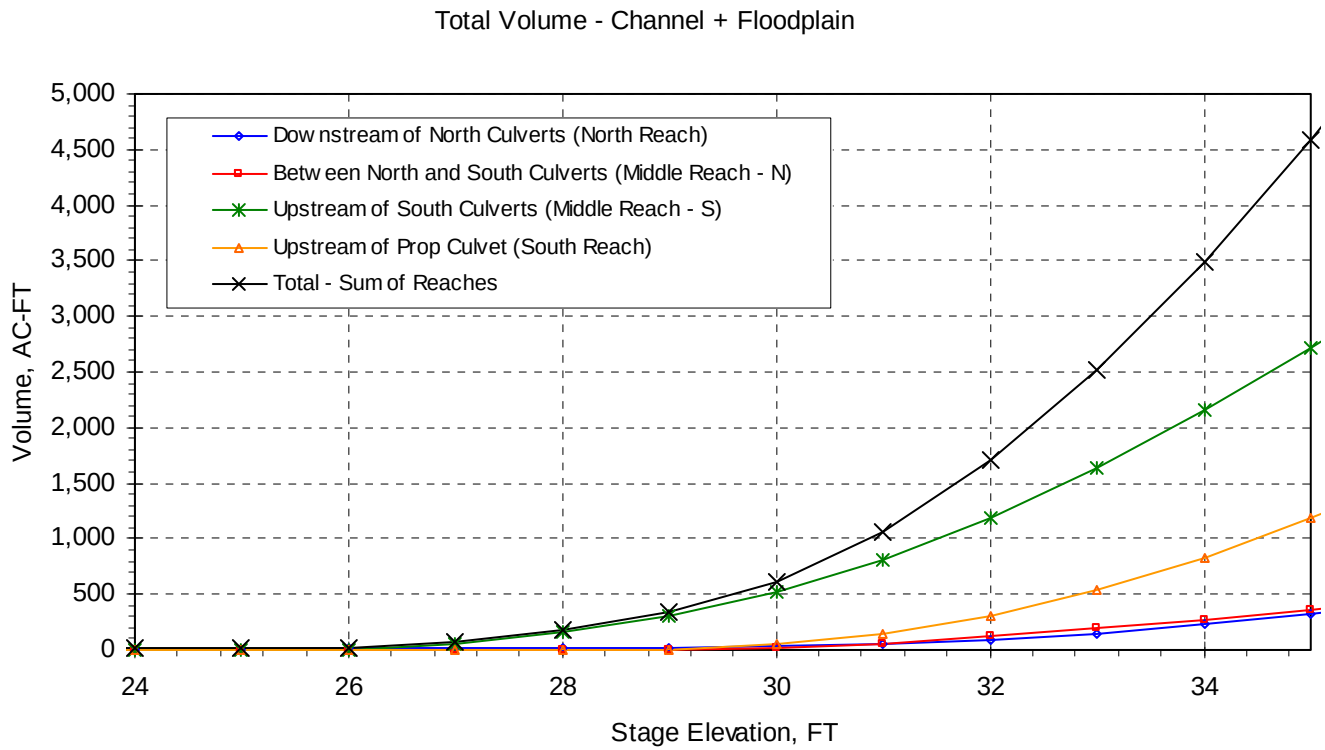

Figure 10. Total channel and floodplain volume for Pearson's Eddy Slough.

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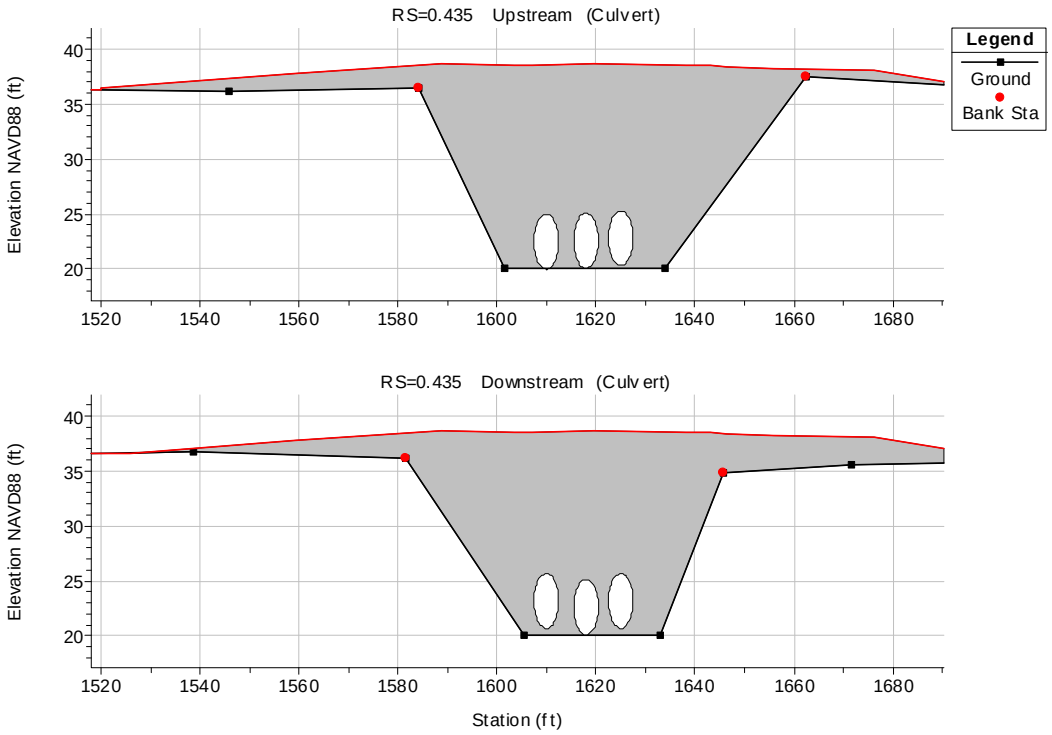

Figure 11. Existing North Crossing culvert configuration (Pearson's Eddy RS 0.435, upstream and downstream faces shown).

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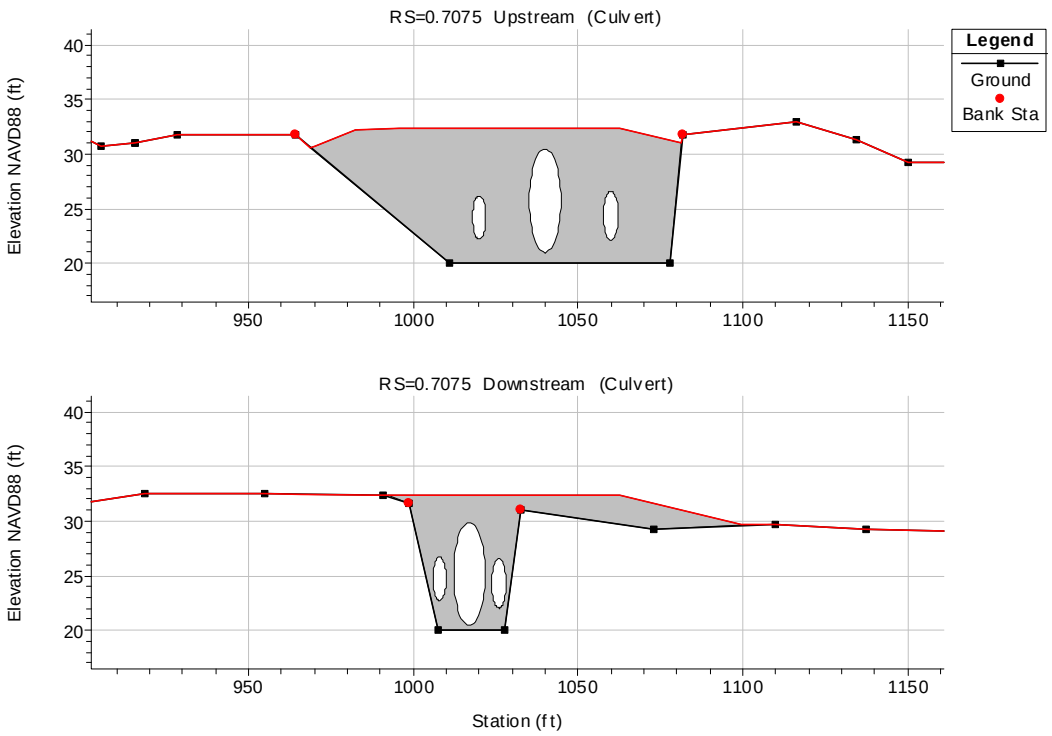

Figure 12. Existing South Crossing culvert configuration (Pearson's Eddy RS 0.7075, upstream and downstream faces shown).

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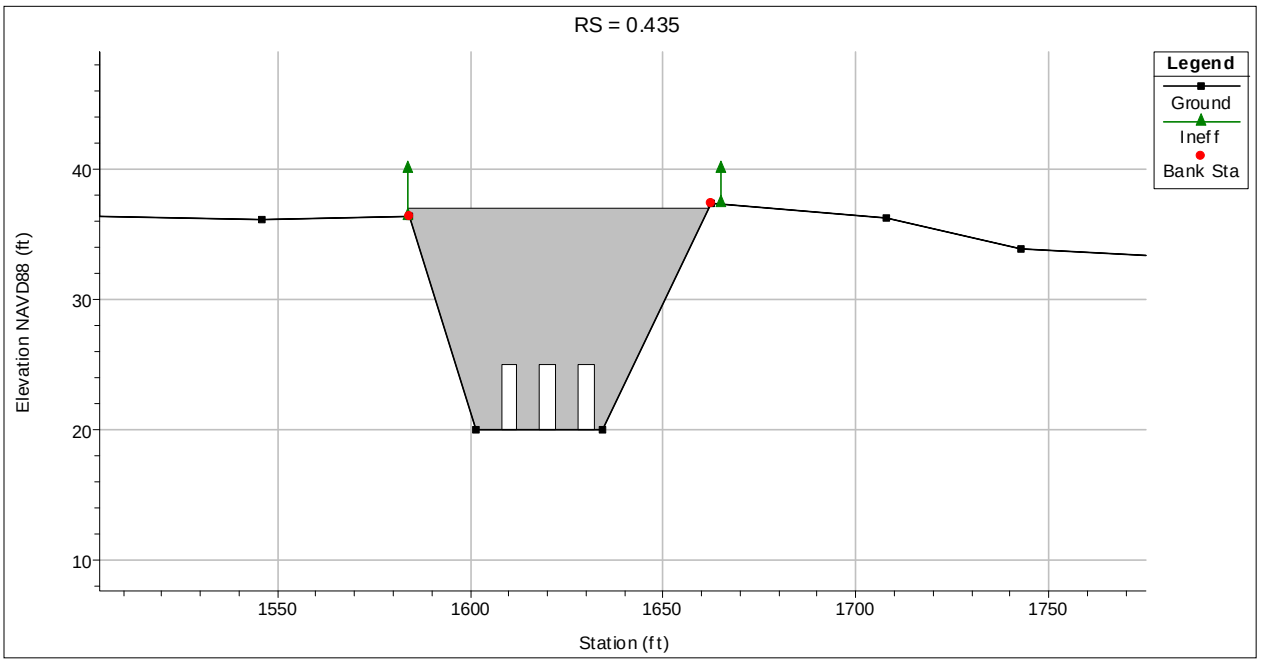

Figure 13. Existing North Crossing culvert configuration with flap gates for unsteady analysis (Pearson's Eddy RS 0.435).

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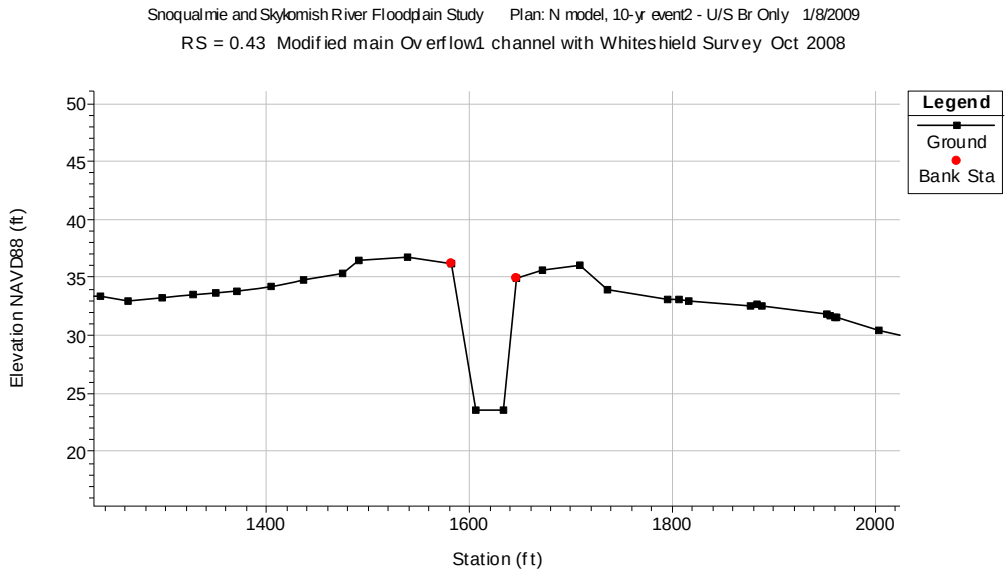

Figure 14. Alternative 1 – Open channel at North Crossing (RS 0.435).

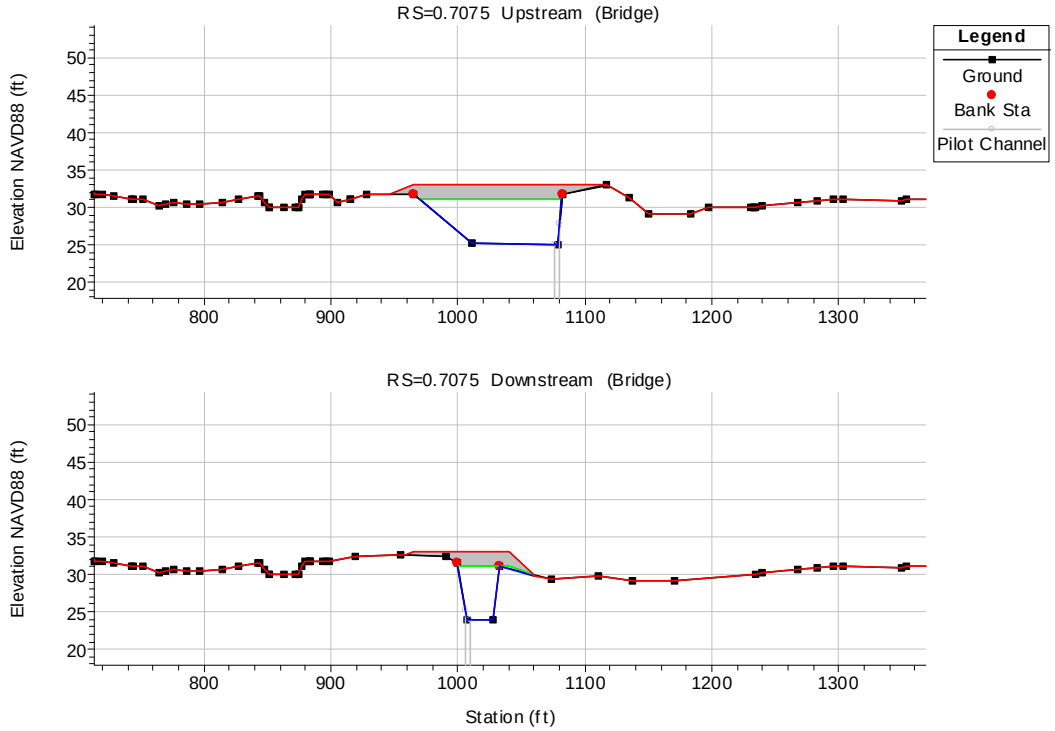

Figure 15. Alternative 1 – Bridge at South Crossing (RS 0.7075).

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4 Results

This section describes the results of the regional and local hydraulic analyses. The discussion first addresses the limitations and interpretation of the model results, then summarizes results for the existing condition and each alternative, and finally narrates the inundation mapping developed from the hydraulic models.

4.1 Limitations and Interpretation of Results

HEC-RAS is a one-dimensional hydraulic model that can simulate steady state or unsteady flow (time-varying) conditions. The model is designed to predict variations in water surfaces, velocities, and other hydraulic properties along the main axis of flow (i.e., the channel thalweg or centerline). While the model geometry definition includes floodplain and overbank regions adjacent to Pearson's Eddy Slough, the WSE results do not vary between the main channel and overbank. For example, a water surface elevation result at any given cross section is assumed to be constant however far that cross section extends beyond the main channel. In reality, the water surface may vary over a cross section, especially at locations well beyond the main channel where hydraulic structures (e.g., culverts, berms, drainage tile) impede the hydraulic connection between the main channel and the floodplain.

Therefore, analysis and interpretation of simulation results should consider the limitations of extrapolating two-dimensional inundation mapping from one-dimensional hydraulic model. Inundation maps are generated by projecting the maximum water surface elevation calculated by the HEC-RAS model across the study area. As a result, the inundation maps represent only a potential inundation area, based on the premise that off-channel areas are hydraulically connected to the Pearson's Eddy Slough. The inundation maps are also predicated on the assumption of a peak stage of 30.7 ft NAVD88 on the Snoqualmie River at the confluence with Pearson's Eddy.

The current analysis does not address the frequency or duration of the potential inundation at the Pearson's Eddy site. The current analysis is an event-based simulation, based on a hydrograph scaled from a historical event extracted from the FIS model. Frequency and duration analysis were outside the scope of the present study. Such an analysis might involve the analysis of springtime mean daily flow gauging data on the Snoqualmie River, and possibly the installation of water level-loggers at the project site, to provide a statistical model (elevation, frequency, and duration) of springtime stages on the Snoqualmie River stage at Pearson's Eddy. The analysis would need to recognize the possible influence of the downstream river system on water elevations at the project site.

4.2 Regional Flood Simulations

Regional flood simulations of the existing condition reflect the current Pearson's Eddy channel, with upstream and downstream road crossings and culverts. Figure 7 illustrates the hydraulic model schematic. The existing condition scenarios are identical, with the exception of using the different flow boundary conditions listed in Table 1.

The analysis ran the existing condition hydraulic model under the three boundary condition scenarios as per Table 1. The selected FIS-modeled events (5-year, 10.6-year, and 23.9-year) were chosen as the most closely representative of the 5-year, 10-year, and

25-year return period. The simulated water surface elevations (WSEs) for these three scenarios are compared in Figure 18 and at discrete locations in Table 10.

Table 10. Maximum WSEs under the existing condition for selected regional flow events.

Location	Mouth	South Face of DS Culverts	South Face of US Culverts	Intermediate Location	US Limit of Slough
River Station	**0.060**	**0.435**	**0.710**	**1.510**	**4.800**
5-year	43.73	43.92	44.07	44.22	47.36
10-year	46.32	46.44	46.54	46.66	48.77
25-year	46.60	46.72	46.82	46.93	49.07

Initial hydraulic runs of the existing and alternative configurations on Pearson's Eddy Slough indicated that modifications to the structures in Pearson's Eddy Slough would have little appreciable difference in flood elevations, even during smaller regional flood events. This was primarily due to the scale of the over-bank flow from the Snoqualmie River, which involved over 20,000 cfs and several feet of depth over the valley floodplain.

In light of these initial results, the project team decided to decouple Pearson's Eddy Slough from regional flood flows on the Snoqualmie/Skykomish/Snohomish River system in order to isolate the slough and obtain more relevant results. Similar to the local hydrology analysis, this de-coupled approach would represent conditions where flows in the Snoqualmie River are high enough to prevent flows from Tuck Creek and other local streams from entering the Snoqualmie River main stem, which would result in their direction down-valley, through Pearson's Eddy Slough.

4.3 Existing Condition

The 'local model,' or the Pearson's Eddy Slough decoupled from the Snoqualmie River, includes the main slough drainage from RS 4.8 upstream to the Snoqualmie confluence (downstream-most RS 0.06). Figure 17 illustrates a schematic of the decoupled model, which correlates the river station and cumulative channel distance.

4.3.1 Steady Flow Analysis

The 'local model' existing conditions were first analyzed under various steady state downstream stage and upstream inflow boundary conditions. The primary purpose of this steady state analysis was to show the potential effect of the alternatives on water surface elevations (i.e. reductions in stage) that might result of removal of existing structures.

The analysis includes a range of flows and stage boundaries to evaluate the sensitivity of the model to water stages and inflows likely to occur under more frequent, lower flow conditions in the Snoqualmie River and local subwatershed.

Downstream stages of 28 feet NAVD88 and 32 feet NAVD88 were selected because they occur within the range of the Snoqualmie River during smaller regional flood events (reference – FIS model, 3 year event, Feb. 1972 – elevations ranged from 22 feet to 41 feet NAVD88). This range of stages is also largely within the main bank elevations of Pearson's Eddy, which are generally above 30 feet NAVD88. Also, these represent stages above and below the proposed upstream bridge deck elevation of approximately 30 feet NAVD analyzed in Alternative 1.

Table 11. Stage and inflow boundary conditions analyzed in the existing conditions scenarios.

Profile (PF) No.	Downstream Stage (ft NAVD88)	Upstream Inflow (cfs)	Estimated Recurrence Interval
1	28	20	Common Storm Event
2	32	20	Common Storm Event
3	28	48	1-year
4	32	48	1-year
5	28	208	2-year
6	32	208	2-year
7	28	327	5-year
8	32	327	5-year

The flows selected range from 20 cfs, which is approximately half of the 1-year flow (representing a common storm flow that might occur several times a year), to the 5-year flow of 327 cfs. Table 11 summarizes these flow and stage profiles.

Figure 19 and Figure 20 illustrate the results of the existing condition steady-state scenarios. These show variations in the longitudinal water surface profile along Pearson's Eddy. For reference, the downstream and upstream road/culvert crossings are located 1,932 feet and 2,970 feet from the mouth of the slough, respectively. The other bridge located 20,120 feet from the mouth is the Woodinville-Duvall Bridge.

One result of these sensitivity runs is that both the downstream and upstream culverts appear to impact water surfaces only for flows larger than the 1-year event. For the 1-year flow, the head differential across either road crossings/culverts is less than 0.1 feet (see profile (PF) No. 3 in Figure 19, and PF No. 4 in Figure 20).

Under the 28 foot stage boundary (and higher 2-year and 5-year flow) conditions, head differentials range from 0.2 feet to 1.1 feet across the culverts (PF Nos. 5 and 7 in Figure 19). Under the 32-foot boundary at the downstream culvert, there are 0.5-foot and 1.1-foot differentials (PF Nos. 6 and 8 in Figure 20). However, under the 32-foot stage condition, there is only a marginal differential across the upstream culvert (PF Nos. 6 and 8, Figure 20). This is because flows overtop this road crossing at higher stages but not at lower stages. Thus the most sensitive boundary conditions are those occurring at higher local flows and lower stages in the Snoqualmie River. Table 12 summarizes these results.

4.3.2 Unsteady Flow Analysis of Spring Conditions

In addition to the steady state analysis described above, the project team also conducted an analysis under time-varying (unsteady) conditions. Unsteady analysis was required primarily to show the effects of the alternatives during common spring conditions during which time stages in the Snoqualmie River dominate conditions on Pearson's Eddy Slough. During these relative frequent (almost yearly) events, stages on Snoqualmie River rise considerably, causing the waters to back-up into Pearson's Eddy Slough, often resulting in property flooding. These conditions are transitory in nature—occurring over several days up to a week or more, and they can not be captured using steady state analysis.

Table 12. Existing condition difference in WSE across downstream and upstream crossings/culverts.

Profile No.	Scenario	Head Differential – Downstream Crossing/ Culverts	Head Differential – Upstream Crossing/ Culverts
		(ft)	(ft)
1	'Typical'+28' Stage	0.0	0.0
2	'Typical'+32' Stage	0.0	0.0
3	1 Year+28' Stage	0.02	0.01
4	1 Year+32' Stage	0.02	0.01
5	2 Year+28' Stage	0.5	0.2
6	2 Year+32' Stage	0.5	0.04
7	5 Year+28' Stage	1.1	0.4
8	5 Year+32' Stage	1.1	0.03

The analysis simulated the effects of the existing self-regulating tide or flap gates located at the north crossing/culverts. Each of the three culverts at this location has a flap gate that prevents downstream stages from propagating upstream during the spring conditions.

Figure 23 summarizes the results of the unsteady analysis of the existing conditions. The figure shows a snapshot of the maximum WSE (i.e., the worst-case or critical condition) along the main flow profile of Pearson's Eddy. Two inflow conditions are shown in the figure: Plan 85 assumes an inflow to Pearson's Eddy of 8 cfs, and Plan 89 assumes an inflow of 208 cfs. The lower flow rate is estimated to be the baseflow of the system, and the upper flow represents a 2-year flow event.

The abrupt drop in WSE at approximately channel distance 2,000 feet shows the effect of the existing flap gates. Downstream of the flap gates where the high Snoqualmie River stage directly influences the WSE in Pearson's Eddy Slough, the WSE is slightly over 30 feet NAVD88; while upstream, the WSE is approximately 27.5 feet NAVD88. The WSE does not reach a stage of over 30 feet NAVD88 again until approximately channel distance 8,000 feet (RS 1.71).

Figure 23 also shows the sensitivity of different upstream inflows under the same downstream stage variation. Plan 85 (inflow of 8 cfs) represents an estimate of lower end inflow likely to occur during spring conditions. Plan 89 is an estimate of the upper end inflows (208 cfs, approximately a 2-year flow event), above which the system is probably less dominated by downstream stages.

The interesting result is that WSEs directly upstream of the existing flap gates do not differ significantly under the low inflow and high inflow cases. The WSEs are nearly identical over a region of greater than one mile upstream of the flap gates. The lack of sensitivity to these flows is believed to be caused by the large broad overbank regions (in the vicinity of RS 1.08) that carry shallow, inactive flow and dampen changes in WSEs under both of the inflow cases.

4.4 Alternative 1

Results for Alternative 1 were simulated using the local, decoupled model. This scenario was evaluated under the same eight steady state stage/flow profiles as the existing condition scenario (see Section 4.3.1). Figure 21 and Figure 22 illustrate the water surface profile results for the 28 foot and 32 foot stage boundaries, respectively. For the 28 foot stage boundary shown in Figure 21, none of the flow profiles overtop the proposed upstream bridge. However, for the 32 foot boundary case, all profiles overtop the bridge.

Figure 24 compares the existing and proposed conditions under a 1-year flow and varying downstream stages. Results show insignificant differences in WSE when removing the downstream culvert and replacing the upstream culverts with a bridge under typical (not shown) and 1 year flows. This plot is shown to demonstrate that impacts of proposed Alternative 1 are negligible for the lower flow levels regardless of the downstream stage.

Figure 25 shows this same comparison but under the 2-year flow conditions. Decreases in the WSE under Alternative 1 are greatest near the removed downstream culvert where they approach 0.5 feet under both stage conditions. Under the 5-year flows shown in Figure 26, differences in the WSE near the downstream culvert are over 1 foot for both stage conditions. The differences in WSEs under existing conditions and Alternative 1 at various locations are summarized in Table 13.

Table 13. Decrease in WSE under Alternative 1 relative to existing conditions at various locations under different flow and stage conditions.

		Mouth of Slough	South of DS Culverts (Removed)	South of US Culverts (Replaced w/ Bridge)	Inter-mediate Location	US Limit of Slough
Profile	**Scenario**	**RS 0.06**	**RS 0.44**	**RS 0.710**	**RS 1.52**	**RS 4.800**
PF1	'Typical'+28' Stage	0.0	0.01	0.01	0.00	0.0
PF2	'Typical'+32' Stage	0.0	0.00	0.01	0.01	0.0
PF3	1 Year+28' Stage	0.0	0.03	0.03	0.00	0.0
PF4	1 Year+32' Stage	0.0	0.03	0.04	0.03	0.0
PF5	2 Year+28' Stage	0.0	0.45	0.59	0.01	0.0
PF6	2 Year+32' Stage	0.0	0.46	0.48	0.42	0.0
PF7	5 Year+28' Stage	0.0	1.12	1.39	0.13	0.0
PF8	5 Year+32' Stage	0.0	1.13	1.13	0.87	0.0

Alternative 1 was also compared to existing conditions under unsteady flow conditions to assess effects during spring stages and inflows of 8 cfs (see Figure 27). Table 14 tabulates the changes in WSE at critical locations for each of the alternatives. Results show a considerably higher WSE upstream of the existing flap gate culverts under Alternative 1. The WSE elevations under the existing and Alternative 1 scenarios are 27.5 feet and 30.5 feet NAVD88, respectively. This approximately 3-foot difference extends from RS 0.44 (existing flap gates) to RS 2.03.

4.5 Alternative 2

Alternative 2 includes removal of the North Crossing flood gates and a flap gate structure at the South Crossing location to replace the existing free-flowing culverts. Figure 28 presents a comparison of maximum WSE from the unsteady simulation for Alternative 2 to the Existing Condition. Table 14 tabulates the changes in WSE at critical locations for each of the alternatives. The simulated profiles show differences only between the North Crossing (at the existing flap gate) and the South Crossing (where the Alternative 2 proposed flap gate is placed). The difference in WSE between the crossings is again approximately 3 feet. Upstream of the flap gates in Alternative 2 at the South Crossing, the WSEs are identical. The WSEs are also identical downstream of the North Crossing. This result appears reasonable as Alternative 2 effectively relocates the limit of Snoqualmie River intrusion from the North Crossing to the South Crossing.

Though the Alternative 2 flood gate flood would stop backwater at the South Crossing in the immediate vicinity of Pearson's Eddy Slough, an examination of local topography indicates that the backwater could spill over the South Crossing embankment through Treen Lake. Treen Lake is located approximately 800 feet east of the Pearson's Eddy Slough South Crossing. Topographic mapping indicates a small drainage channel through the South Crossing embankment on the south side of Treen Lake, and that the spill elevation on the east side of the lake is approximately 29 feet NAVD88. The one-dimensional HEC-RAS model does not fully capture the dynamics of the flanking of the South Crossing embankment via Treen Lake. However, it is apparent that if Treen Lake is hydraulically connected to Pearsons Slough (or the project improves the hydraulic connection between the slough and the lake), there is potential for backwater to inundate the property to the south via Treen Lake despite the flood gate at the South Crossing. Mapping of the potential inundation (see Section 4.7 and Figure 33) notes the potential spill on the east side of Treen Lake. Similarly, restoring the hydraulic connection between Pearsons Eddy Slough and the remnant channels on the east side of the site could create inundation beyond the boundaries of the Cascade Conservancy parcels.

Therefore, Alternative 2 would need to include measures at Treen Lake and points east, in addition to the flood gate on the slough, to prevent inundation beyond the upstream boundaries of the Cascade Conservancy parcels. These measure might include additional flood gates, or berms or embankments. The height of the berms or embankments would need to match the elevation of top of road elevation at the South Crossing to avoid flanking of the flood gate and inundations of upstream properties. Similar measures would be needed to prevent inundation of remnant channels upstream of the Cascade Conservancy parcels.

4.6 Alternative 3

Alternative 3 includes removal of the North Crossing flood gates, an open bridge at the South Crossing, and flap gate structures at the upstream end of the mitigation bank property. Figure 29 compares Alternative 3 with the Existing Condition. Alternative 3 (Plan 98 in the figure) shows high stages extending upstream to the end of the mitigation bank property at RS 1.52, and then a drop in stage of nearly 1 foot (30.59 feet NAVD88 downstream of the flap gate and 29.69 feet NAVD88 upstream of the gate). Compared to

the Existing Condition scenario, the stages are higher at all locations between the North Crossing and RS 2.03 (about a half mile above the end of the mitigation bank property).

Figure 30 compares the Existing WSE and those under each of the three alternatives (again comparing the maximum water surface during the simulation, and inflow of 8 cfs). Table 14 tabulates the changes in WSE at critical locations for each of the alternatives. Alternative 2 shows the greatest effectiveness (and least variation from existing conditions) in terms of limiting high stages in the Snoqualmie River from propagating upstream into Pearson's Eddy Slough. Alternative 1 (removal of all culverts) and Alternative 3 (flap gate at the upstream end of the mitigation bank property) result in very similar profiles throughout the channel, with only a small difference upstream of the proposed flap gate location.

Additionally, results of the hydraulic analysis show that restoration actions can be both effective and targeted at specific regions of the Pearson's Eddy channel. For example, Alternative 2 (new flap gate at the South Crossing) allows downstream stages to propogate up to the South Crossing, limits water depths upstream of this location (similar to the Existing Conditions, and does not cause other undesired impacts such as higher WSEs at upstream locations. Alternative 3 shows a similar targeted and predictable result. This information can potentially be used to further refine alternatives that seek to allow inundation in some regions and limit flooding in others.

Table 14. Comparison of maximum WSEs on Pearson's Eddy Slough (unsteady flow - spring conditions, upstream inflow of 8 cfs).

Reach	1. Mouth to North Crossing	2. North Crossing to South Crossing	3. South Crossing to Upstream Crossing	4. Above Upstream Crossing	5. Above Upstream Crossing
River Station	0.16	0.62	1.08	1.71	2.13
Existing	30.7 ft NAVD88	27.5 ft NAVD88	27.5 ft NAVD88	28.6 ft NAVD88	31.2 ft NAVD88
Alternative 1	30.7 *(+0.0 ft)*	30.6 *(+3.1 ft)*	30.5 *(+3.0 ft)*	30.5 *(+1.9 ft)*	31.2 *(+0.0 ft)*
Alternative 2	30.7 *(+0.0 ft)*	30.7 *(+3.2 ft)*	27.5 *(+0.0 ft)*	28.6 *(+0.0 ft)*	31.2 *(+0.0 ft)*
Alternative 3	30.7 *(+0.0 ft)*	30.6 *(+3.1 ft)*	30.6 *(+3.1 ft)*	28.7 *(+0.1 ft)*	31.2 *(+0.0 ft)*

4.7 Inundation Mapping

Inundation maps are a common way of displaying the extent and depth of water over large regions. They are useful to display differences in water surface elevations between scenarios. The inundation maps created for this project are derived from HEC-RAS model results. There are limitations to this method of extrapolating one-dimensional hydraulic simulation water surfaces to a two-dimensional floodplain (see discussion in Section 4.1) It is important to understand these limitations to avoid misrepresenting or misinterpreting the maps.

Figure 31 through Figure 34 (at the end of the report) display the simulation results for the existing and alternative scenarios as inundation maps. Figure 16 provides a histogram, developed by counting the grid cells on the inundation maps, of the cumulative potential inundation area for the existing condition and proposed alternatives. The inundation maps show depths of water through the Pearson's Eddy channel and adjacent floodplain regions. Color shaded regions correspond to water depths ranging from 0 feet to greater

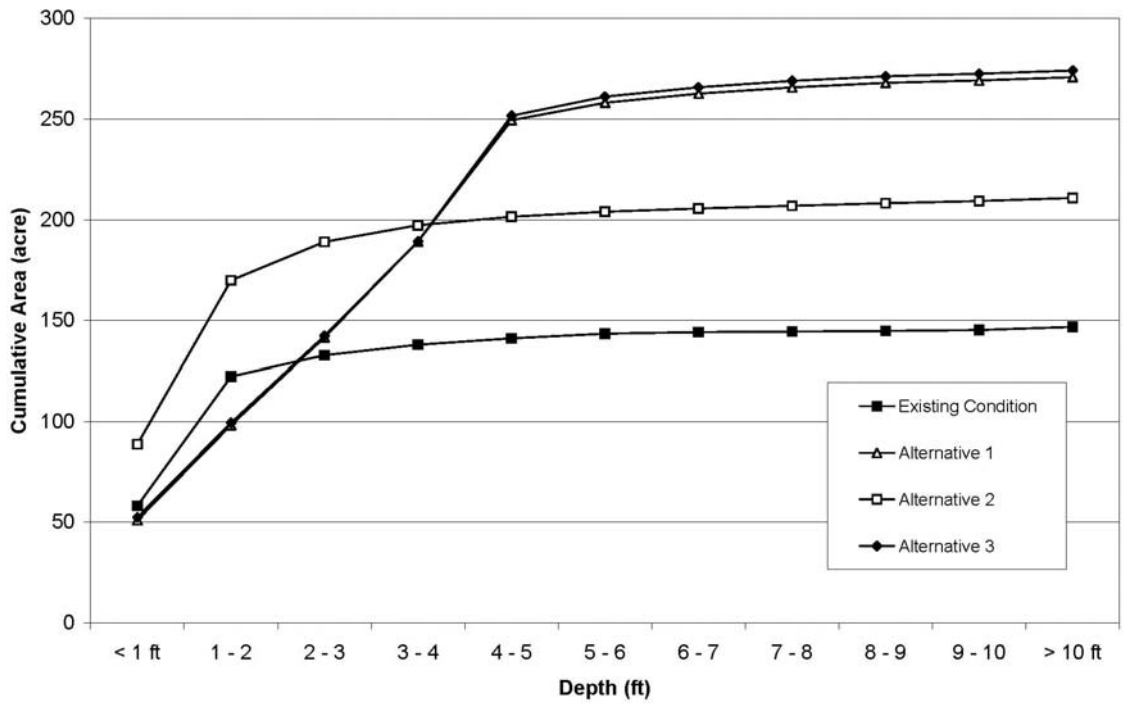

Figure 16. Cumulative potential inundation area for the existing condition and proposed alternatives.

than 10 feet according to the legend of each figure. The maps display inundation for the maximum water surface profile from the unsteady flow simulations of the spring conditions. These maps correspond with the water surface profiles shown in Figure 27.

4.7.1 Existing Condition

Figure 31 shows the estimated inundation for the project vicinity under the existing condition. This map indicates maximum water depths of 10 feet or higher in the main channel north of the North Crossing. This is due to the existing flap gates and the high stages encroaching into the channel from the Snoqualmie River.

The map also shows significant extents where depths range from 0-2 feet upstream of the South Crossing. This is due to the low bank elevations along the channel from RS 0.79 to RS 1.33. The right and left banks at RS 1.33, for example, are at elevations 29.1 and 28.8 feet NAVD88, respectively. As a result, relatively low flows rise slightly above the banks and inundate the overbank region.

The total potential inundation area under the existing condition is approximately 147 acres, with a mean inundation depth of 1.5 ft. Under the existing condition, it appears that the roadway embankment associated with the North Crossing is high enough to prevent significant inundation of the remnant channels located on the east side of the Cascade Conservancy and neighboring parcels. Treen Lake, located approximately 800 feet east of the South Crossing, would be inundated if it were hydraulically connected to Pearson's Eddy Slough or to ponded areas on adjacent parcels. However, the water elevation would not be high enough to spill over on the east side of the lake.

4.7.2 Alternative 1

Figure 32 displays the estimated extent and magnitude of inundation for the project vicinity under Alternative 1. Alternative 1 shows high depths of water in the main channel extending farther upstream than under the existing condition. The overbank region between the South Crossing and Upstream Crossing locations is also inundated, but with greater extent and magnitude (depths range up to 5 or 6 feet).

It is clear that both Treen Lake and the remnant channels on the east side of the Cascade Conservancy property (and adjacent parcels) would be inundated under the Alternative 1 scenario. Under Alternative 1, the total potential inundation area is approximately 271 acres, representing an 85 percent increase over the existing condition. The mean potential inundation depth under Alternative 1 is 2.9 ft. The histogram in Figure 16, perhaps counter-intuitively, indicates a decrease in shallow flooding (1-2 feet depth) between the existing condition and Alternative 1. Examining the inundation map, it appears that this reduction is likely due to the deeper inundation depth of the overbank area between the South Crossing and the Upstream Crossing (i.e., the Habitat Bank parcels). This area shifts from a 0-2 feet depth range to the 3-5 feet depth range, without a directly corresponding increase in the 0-2 feet depth area around the margins.

4.7.3 Alternative 2

Figure 33 displays the estimated extent and magnitude of inundation for the project vicinity under Alternative 2. Alternative 2 and the existing condition scenario are similar. The primary difference is between the North and South Crossings, where Alternative 2 shows greater depth and inundation of significant portions of the overbank region. This is a result of the flap gates being relocated from the North Crossing location to the South Crossing location under this alternative. Under Alternative 2, the total potential inundation area is approximately 211 acres, a 44 percent increase over the existing condition. The mean inundation depth under Alternative 2 is 1.5 ft.

As discussed in Section 4.5, if Treen Lake is hydraulically connected to Pearsons Slough under Alternative 2, there is potential for backwater to inundate the property to the south via Treen Lake despite the flood gate at the South Crossing. Similarly, restoring the hydraulic connection between Pearsons Eddy Slough and the remnant channels on the east side of the site would allow water beyond the boundaries of the Cascade Conservancy parcels under Alternative 2.

4.7.4 Alternative 3

Figure 34 shows the estimated extent and magnitude of inundation for the project vicinity under Alternative 3. Alternative 3 inundation patterns are very similar to Alternative 1. The main channel and floodplain regions under Alternative 3 show significantly greater depths and inundated extents than under the existing condition. The inundation patterns upstream of the South Crossing is also similar to that of Alternative 1. The total potential inundation area under Alternative 3 is approximately 274 acres, representing a 87 percent increase over existing condition. The mean inundation depth under Alternative 3 is 2.9 feet. The primary difference between Alternative 1 and Alternative 3 will be seen in inundation upstream of the Habitat Bank parcels (see previous discussion at Section 4.4 and Section 4.6, and the summary found in Table 14).

5 Conclusions

This report evaluated the potential effects of three alternatives for removal and/or replacement of tide gate structures on the Pearson's Eddy Slough in order to support planning decisions for wetland restoration at the Pearson's Eddy site. The hydraulic response of the slough and the adjacent overbank regions were compared under the existing condition and under three alternative scenarios. The purpose of the analysis was to better understand the likely flows, depths of water, and inundation extents under proposed options in order to support planning decisions for wetland restoration at the Pearson's Eddy site.

The analysis included modification of an existing HEC-RAS hydraulic model to simulate the impacts of the existing and proposed channel configurations under regional flood and spring snow-melt driven hydrologic conditions. The primary conclusions made as a result of these analyses are summarized below:

Pearson's Eddy Channel and Floodplain Storage Capacity

The storage capacity (volume) of Pearson's Eddy Slough relative to that of the adjacent floodplain is small. The existing channel, with its relatively low top of bank elevations and small cross sectional area, has a limited capacity to contain or store flows. The overbank region between Pearson's Eddy and the Snoqualmie River has an order of magnitude greater volume, and accordingly, would be of benefit if modified to store water as desired during high flow/stage events in the system.

Regional Flood Simulations

Initial simulations of regional flood conditions were conducted using the estimated 5-year, 10-year, and 25-year events. Results of these simulations under the existing conditions demonstrated that Pearson's Eddy was substantially inundated under these conditions; in many cases the slough and overbank regions were under at least 5 to 10 feet of water. Thus, the analysis of the high flow and low frequency regional flood conditions was not well-suited to evaluation of the proposed alternatives.

Local (Decoupled) Model Backwater Analysis

A steady state analysis was performed to simulate the backwater effects of the existing culverts and Alternative 1 to determine the general range of conditions under which the model would be sensitive to modifications to the crossings/flap gates. The steady state analysis simulations were representative of typical to high inflows from local watersheds while the Snoqualmie River was near bank-full stage.

Results showed the model to be sensitive with respect to head loss across the North and South Crossings, primarily to relatively high upstream inflow and low downstream stage conditions. The steady-state analysis also indicated the potential benefits of removing the existing flow constrictions at the North and South Crossings under high, sustained (steady state) inflows lasting more than approximately 1 or 2 days due to the increased downstream conveyance capacity.

Unsteady Flow Analysis of Spring Conditions

Under high downstream stage and low local inflow conditions (typical of snowmelt periods common during the spring), simulation of the existing conditions showed that the existing flap gates were effective in preventing downstream water levels from encroaching into the upper reaches of the channel and overbank regions. The WSE differential across the North Crossing was approximately 3 feet, and this effect extended upstream over 1 mile of the channel.

Alternative 1 would remove the North Crossing flood gates and replace the South Crossing culverts. Without any structures to impede the flow, relatively high downstream water levels could extend approximately 2 miles upstream. The calculated increase in WSE at the King County/Snohomish County line was approximately 1.9 feet. The depths and extent of this inundation were also considerably higher compared to the existing condition.

Alternative 2 would remove the North Crossing flood gates and replace the South Crossing culverts with a new flood gate structure. This flood gate structure would prevent flood water from the Snoqualmie River from propagating upstream beyond the South Crossing. Under Alternative 2, the extent and depth of inundation between the North and South Crossings was significant (over 3 feet deeper than under the existing condition), but there was no significant change to water surface elevation or inundation upstream of the South Crossing. Additional measures would be required to prevent flanking of the flood gate at the South Crossing under Alternative 2 via Treen Lake.

Simulation of Alternative 3 showed results similar to Alternative 1, but with only a 0.1-foot increase over the existing condition WSE at the Upstream Crossing (proposed flap gate) location. There did not seem to be other significant differences such as in the magnitude or extent of inundation between these scenarios north of the King County/Snohomish County line. The depths and areal extents of flooding under both Alternative 1 and Alternative 3 were considerably greater than those under existing conditions.

General Conclusions

In general, model results show that it would be feasible to manage backwater from the Snoqualmie River propagating upstream through Pearson's Eddy Slough while limiting high water levels in regions further upstream. Removal of the existing flap gates at the North Crossing will increase local flood elevations significantly, but relocating the flap gates (either at the South Crossing or further upstream) can limit the potential increases to flood elevations in this regime.

Volume analysis of the Pearson's Eddy Slough indicates that the reach between the North and South Crossing has relatively little storage volume. Therefore, it should be feasible to reclaim possible lost flood storage volume due to removal or relocation of flap gates. The flood volume could be accomplished by re-connecting existing remnant channels in the floodplain at the Pearson's Eddy site that are below predicted flood WSEs (for instance, see Figure 32).

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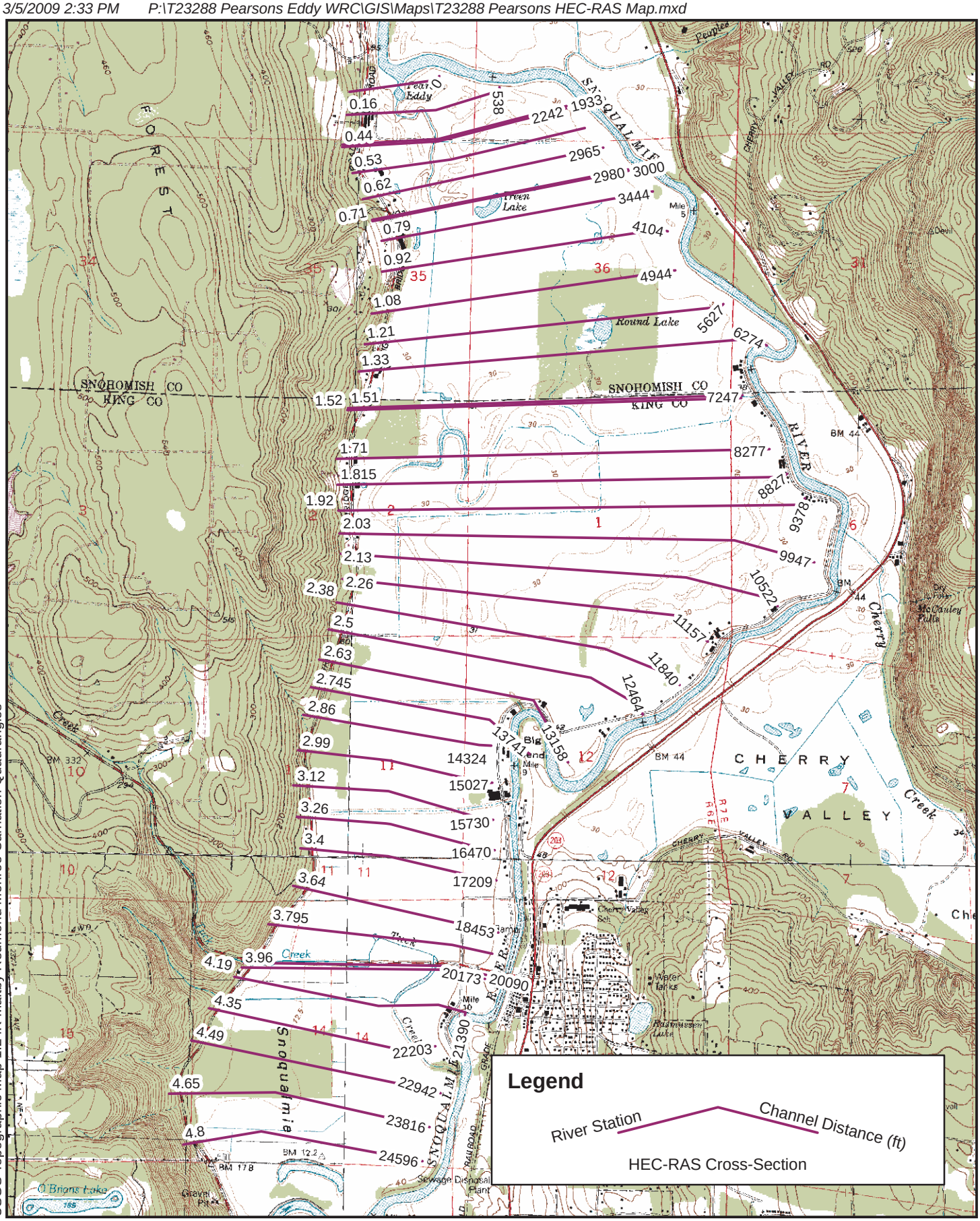

USGS Topographic Map 1:24K Maltby Redmond Monroe Carnation Quadrangles

1 inch equals 0.5 miles

0 0.25 0.5 1

Pearsons Eddy WRC

HEC-RAS Model Cross-Sections

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FIGURE 17

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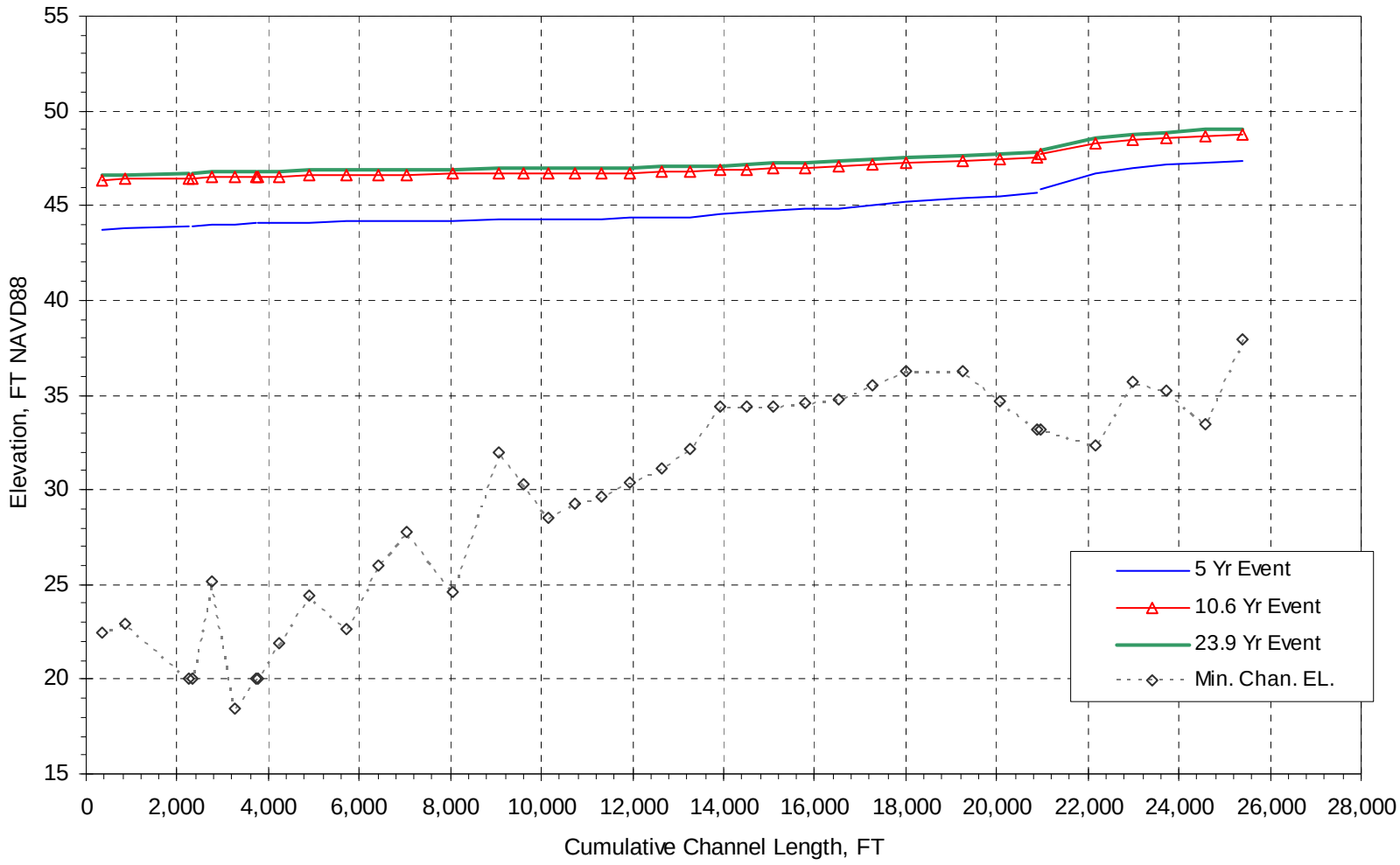

Figure 18. Existing condition WSE of for selected regional flood events.

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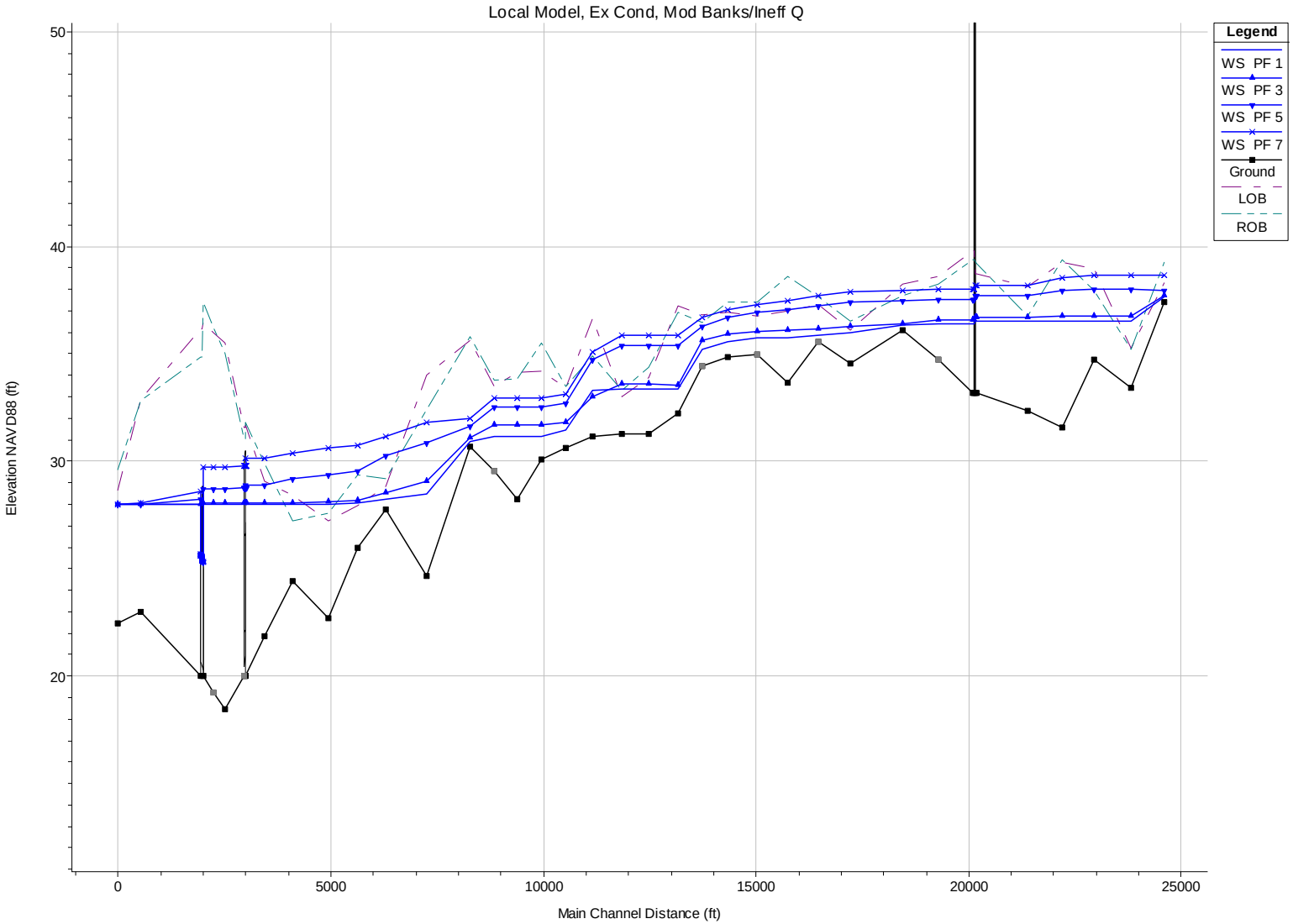

Figure 19. Steady-state WSE profiles, existing condition (20 cfs, 48 cfs, 208 cfs, and 327 cfs inflows and 28 ft NAVD88 downstream stage).

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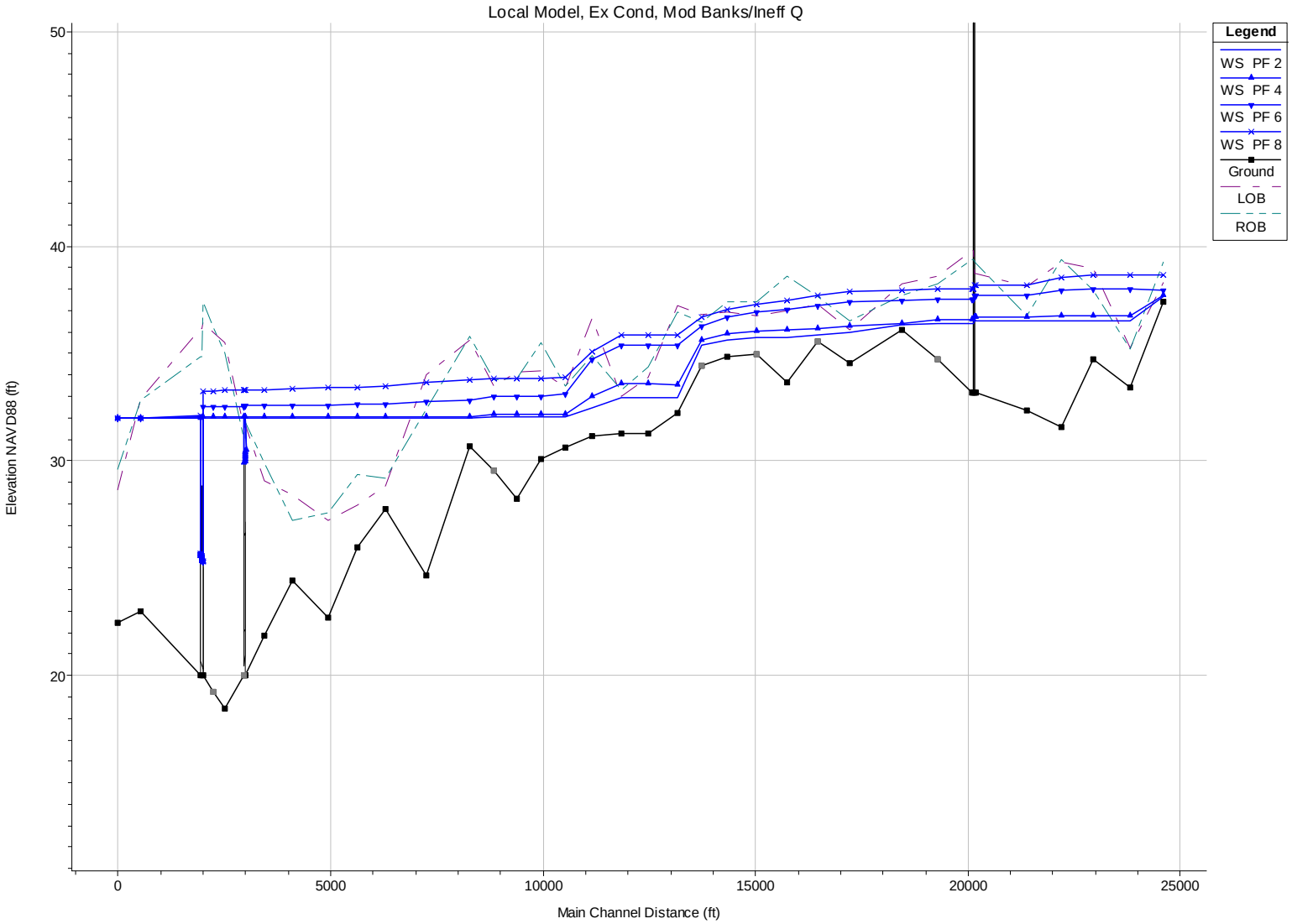

Figure 20. Steady-state WSE profiles, existing condition (20 cfs, 48 cfs, 208 cfs, and 327 cfs inflows, and 32 ft NAVD88 downstream stage).

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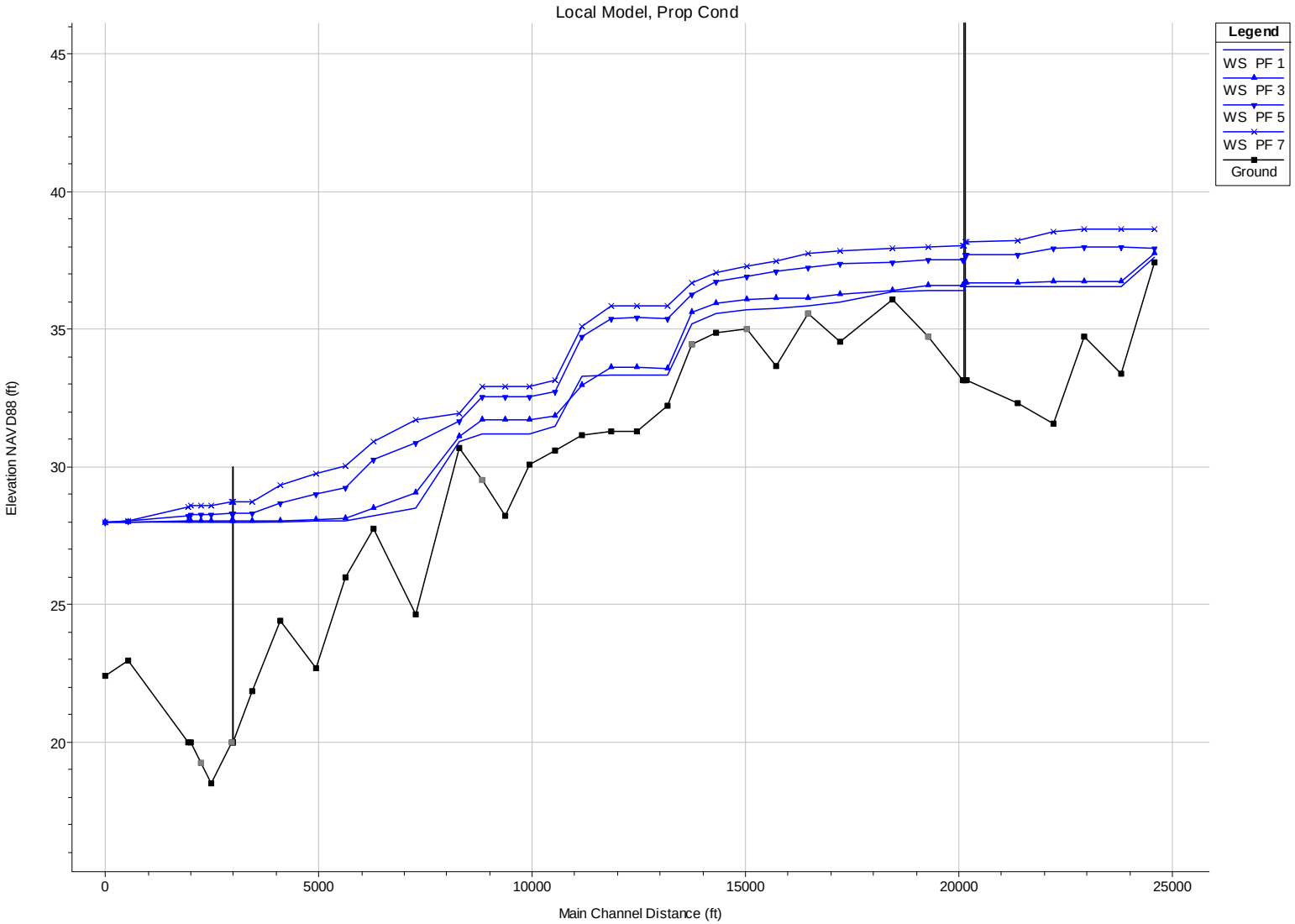

Figure 21. Steady-state WSE profiles, Alternative 1 (20 cfs, 48 cfs, 208 cfs, and 327 cfs inflows, 28 ft NAVD88 downstream stage).

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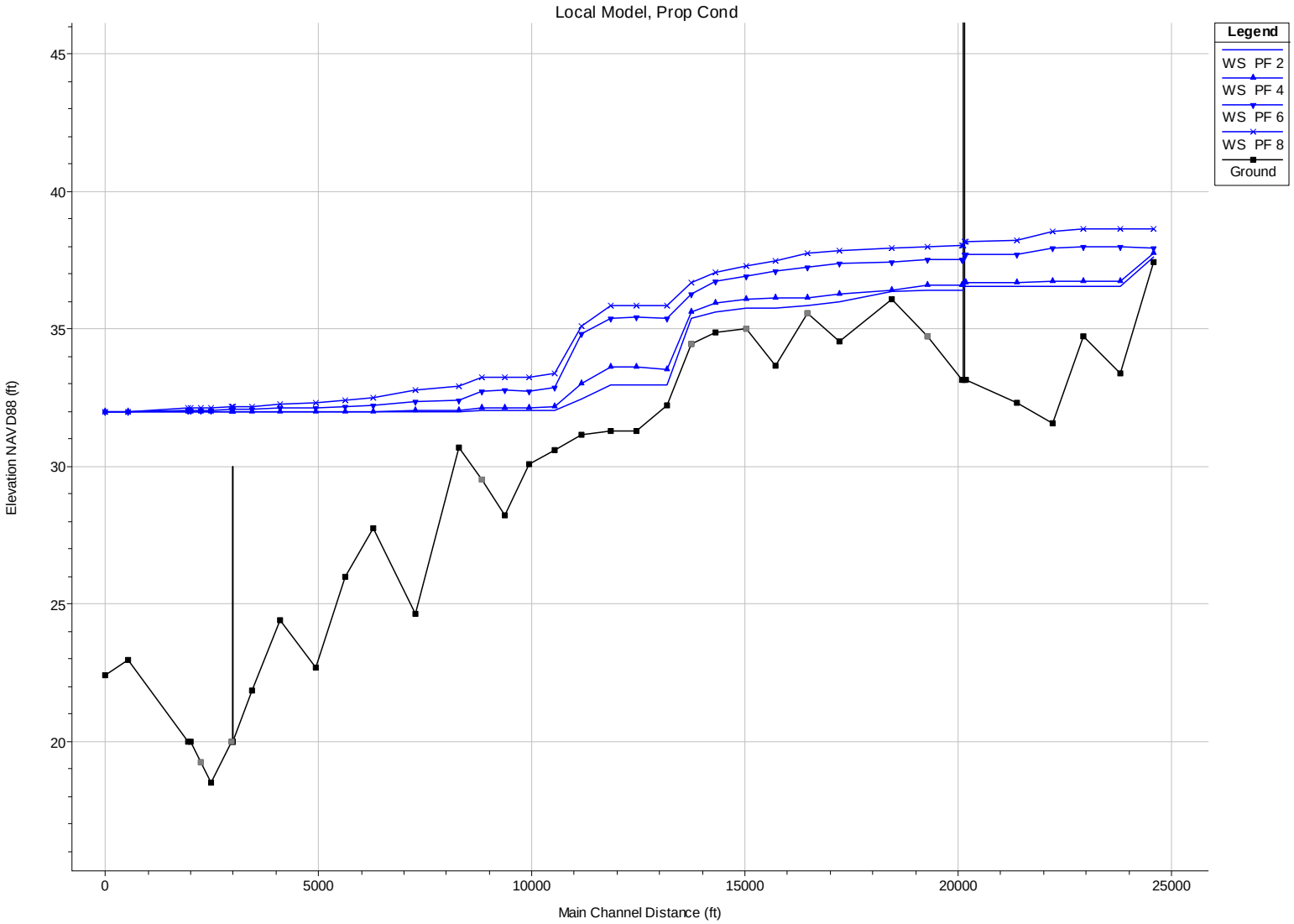

Figure 22. Steady-state WSE profiles, Alternative 1 (20 cfs, 48 cfs, 208 cfs, and 327 cfs inflows, 32 ft NAVD88 downstream stage).

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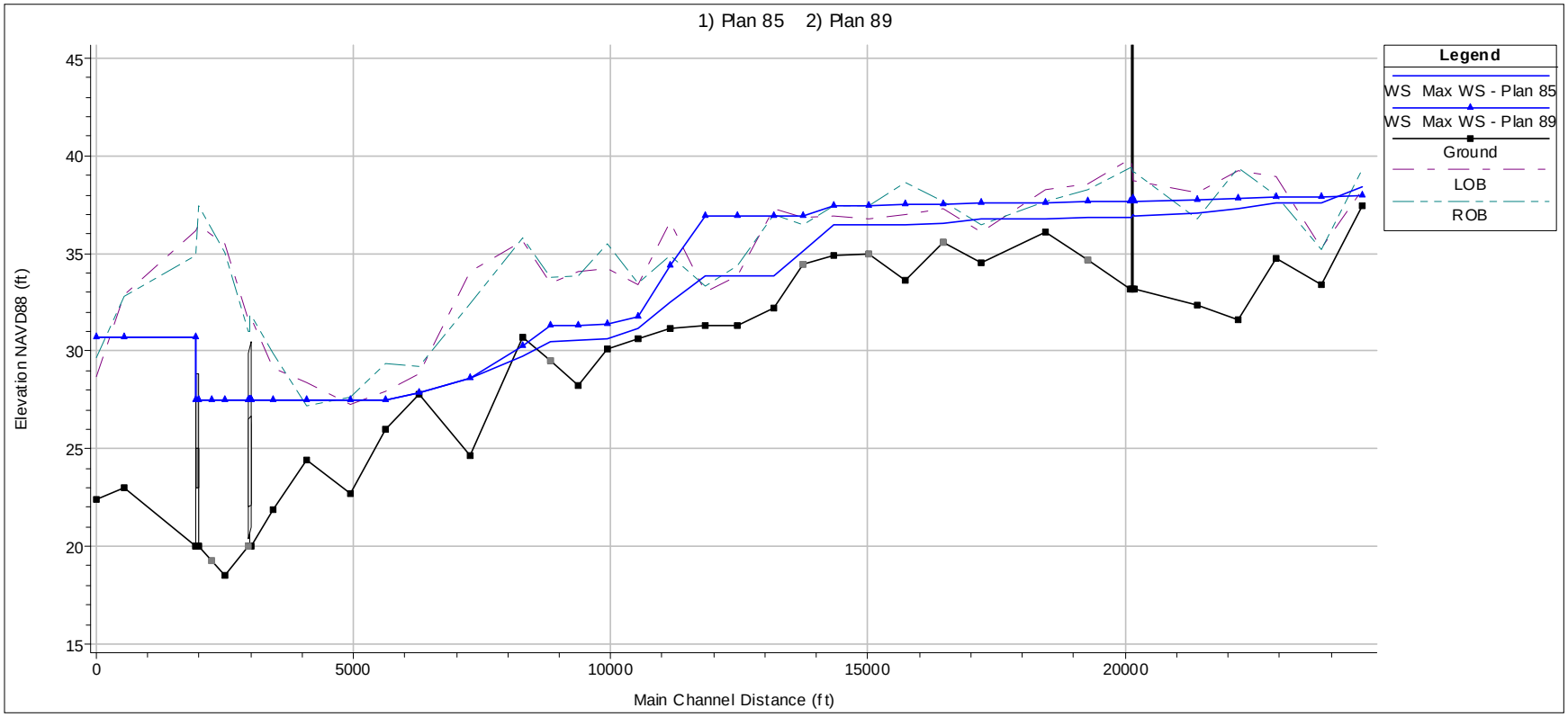

Figure 23. Unsteady WSE profiles for the existing condition show the effects of the flap gate culverts during spring flood events with high downstream stage and relatively low upstream inflows (Plan 85 inflow of 8 cfs, Plan 89 inflow of 208 cfs).

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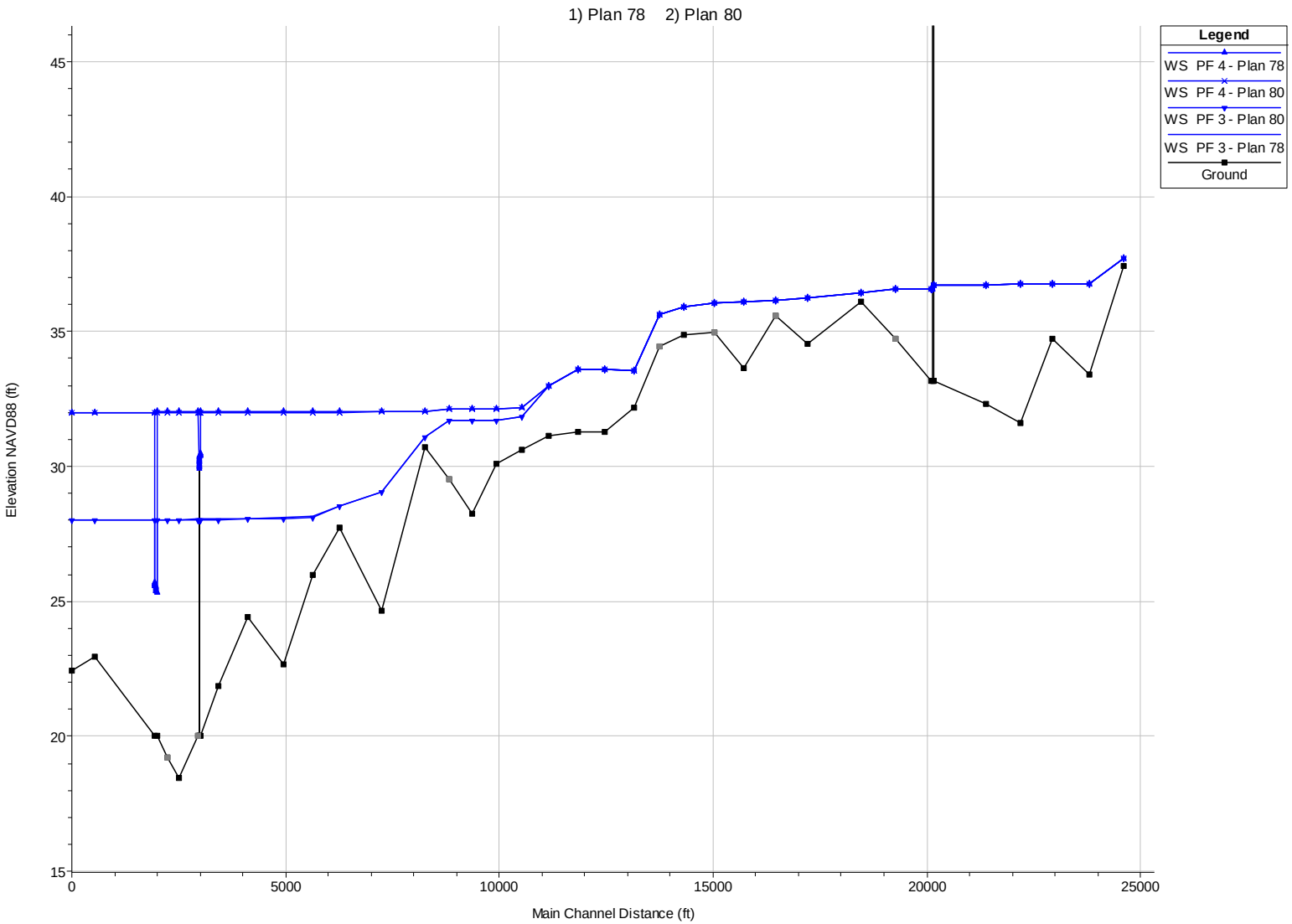

Figure 24. Comparison of existing condition (Plan 78) and Alternative 1 (Plan 80) with 1-year local inflow (steady state; PF 3 28-ft downstream boundary, PF 4 32-ft downstream boundary.)

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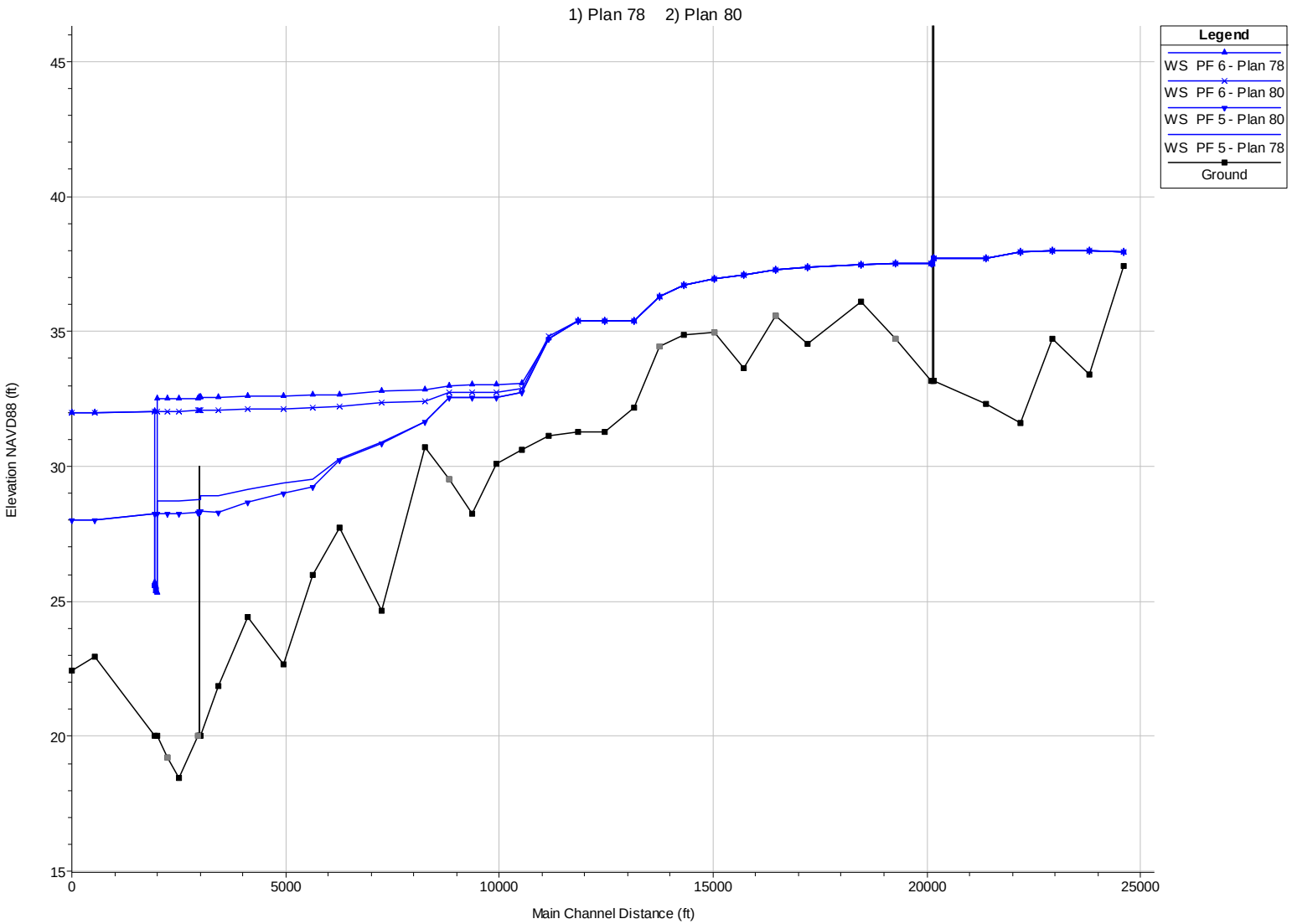

Figure 25. Comparison of existing condition (Plan 78) and Alternative 1 (Plan 80) with 2-year local inflow (steady state; PF 3 28-ft downstream boundary, PF 4 32-ft downstream boundary.)

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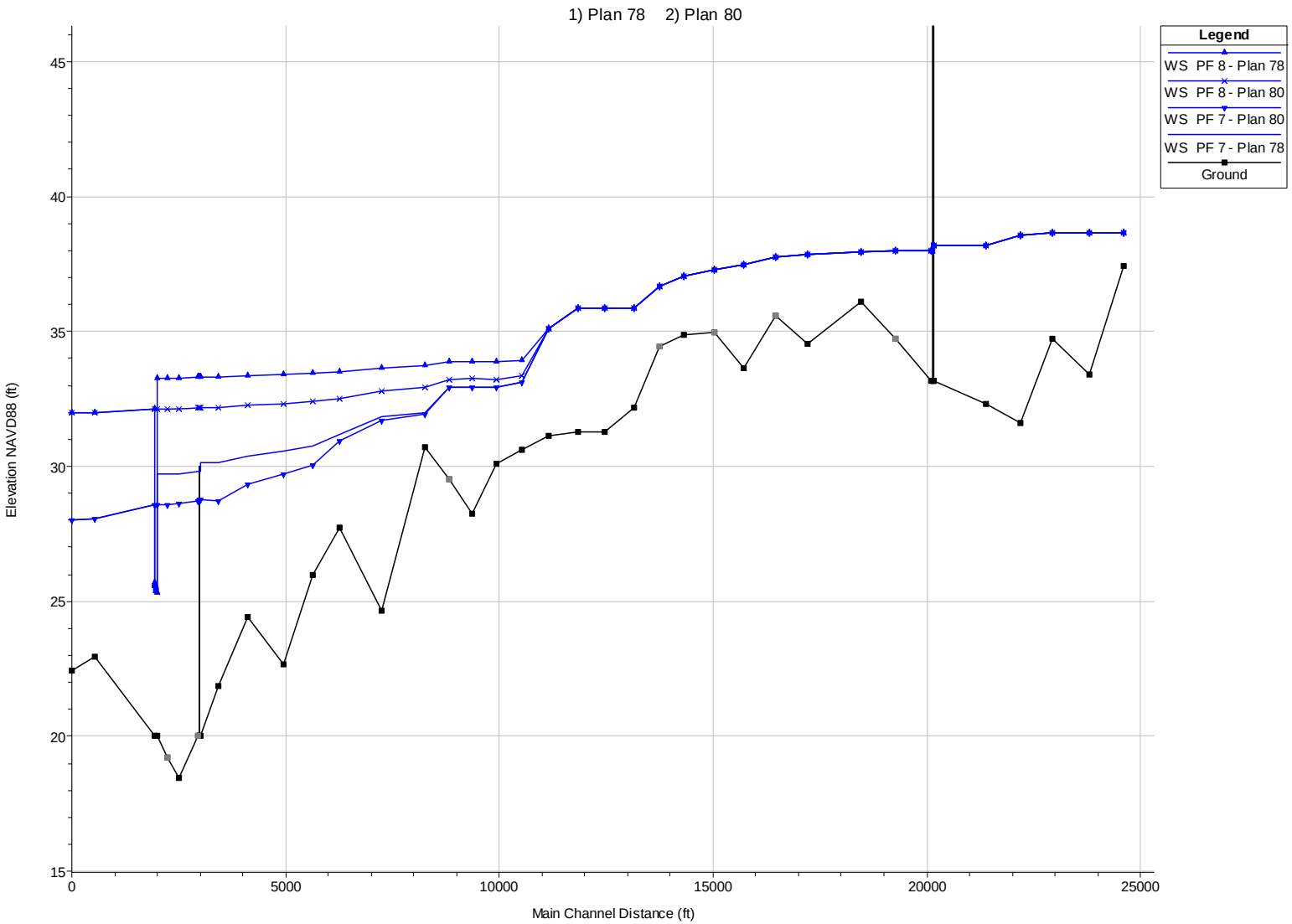

Figure 26. Comparison of existing condition (Plan 78) and Alternative 1 (Plan 80) with 5-year local inflow (steady state; PF 3 28-ft downstream boundary, PF 4 32-ft downstream boundary.)

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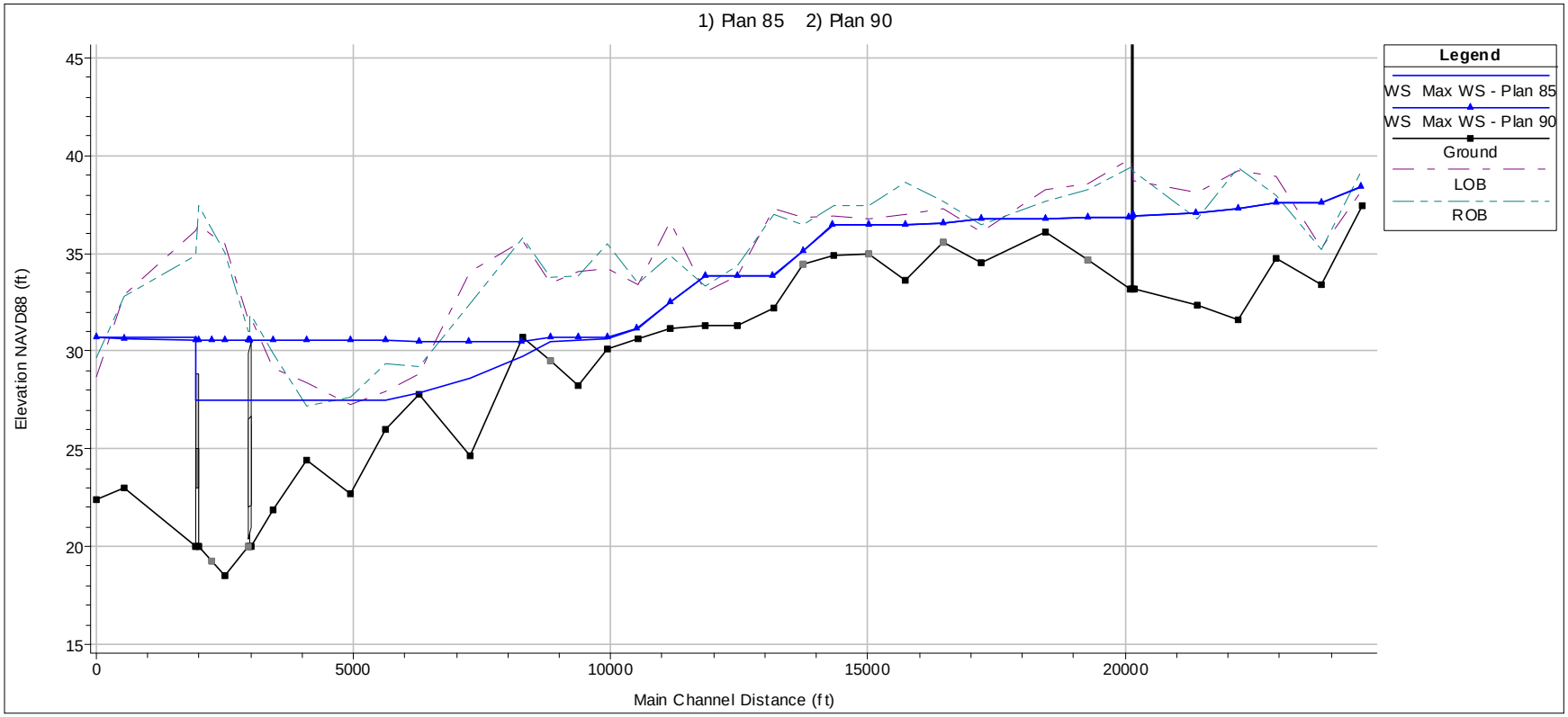

Figure 27. Unsteady flow comparison of Existing Conditions (Plan 85) and Alternative 1 (Plan 90) with inflow of 8 cfs.

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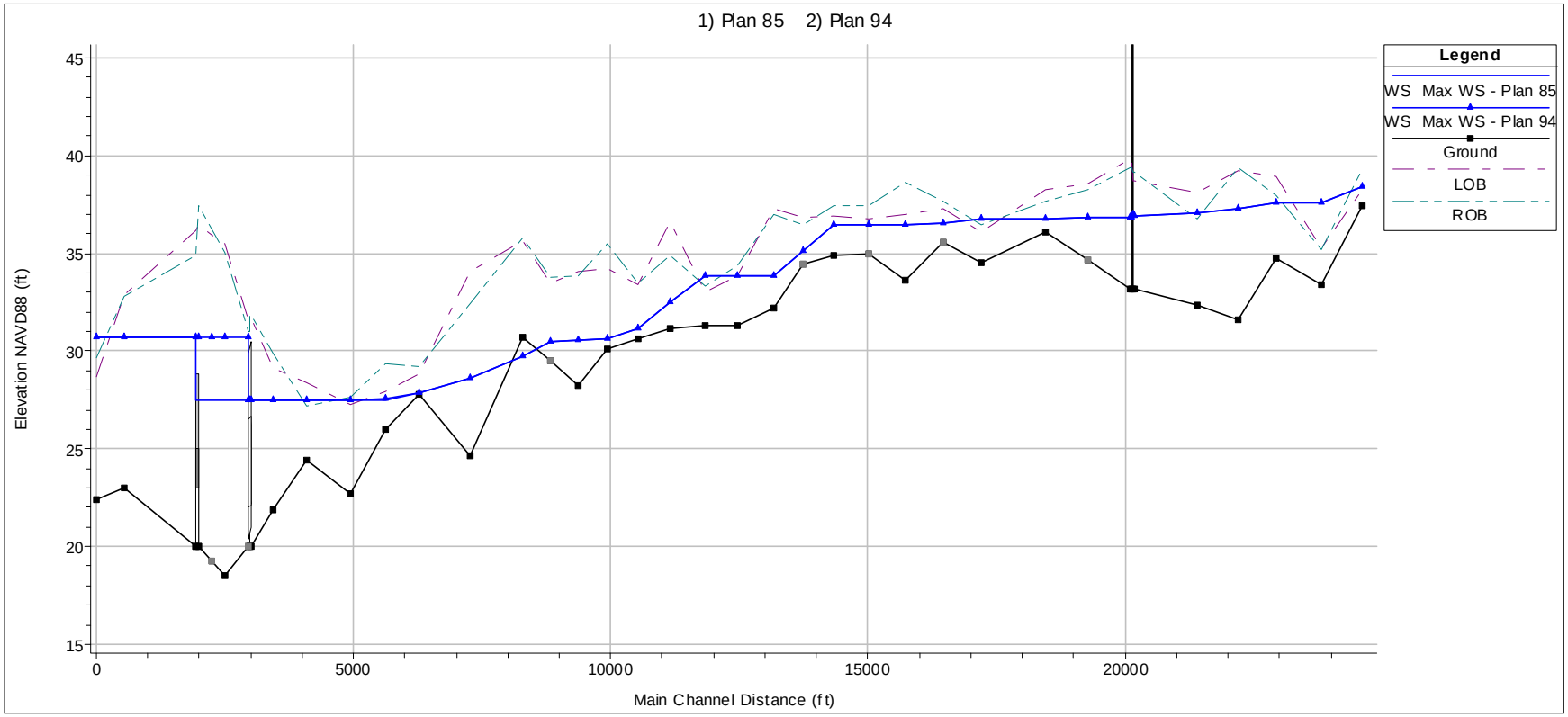

Figure 28. Unsteady flow comparison of Existing Conditions (Plan 85) and Alternative 2 (Plan 94) with inflow of 8 cfs.

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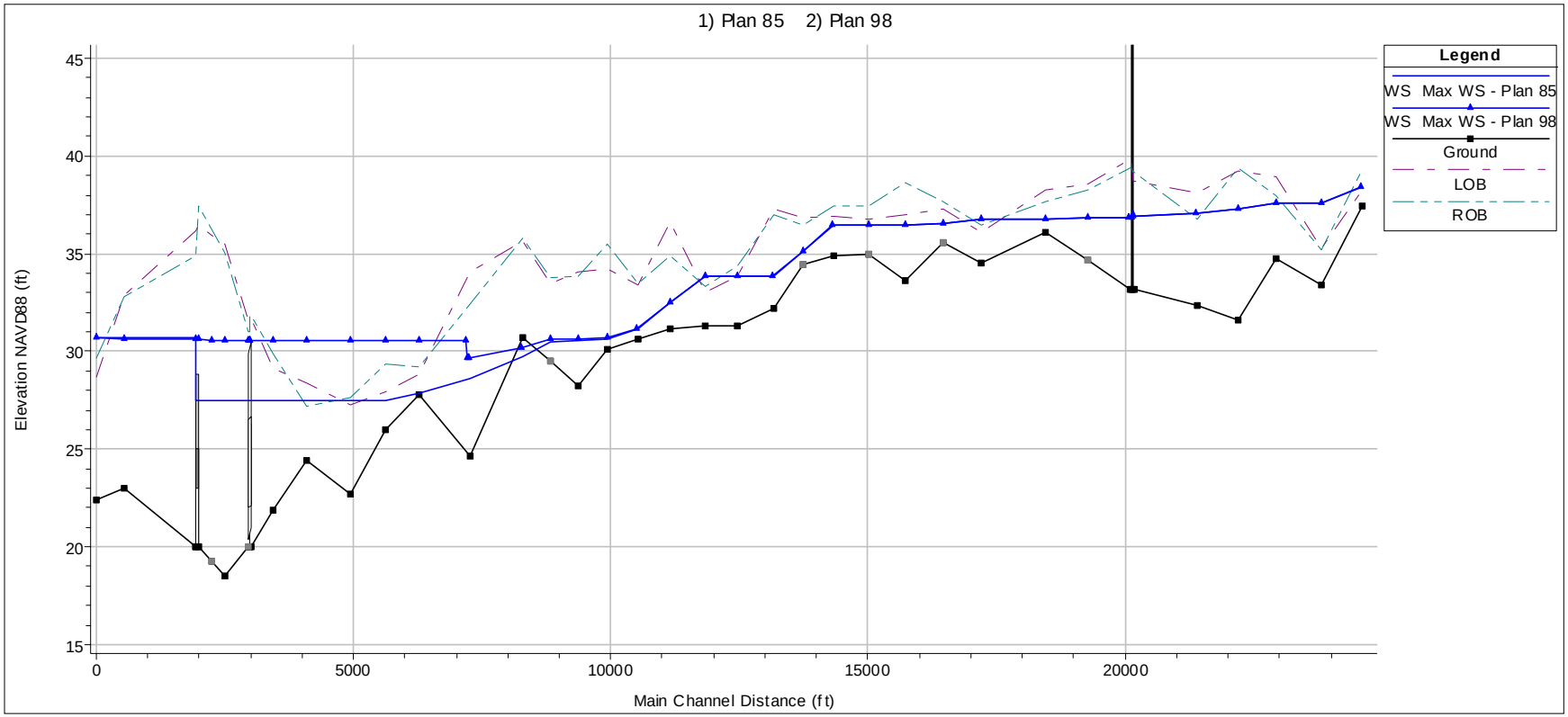

Figure 29. Unsteady flow comparison of Existing Conditions (Plan 85) and Alternative 3 (Plan 98) with inflow of 8 cfs.

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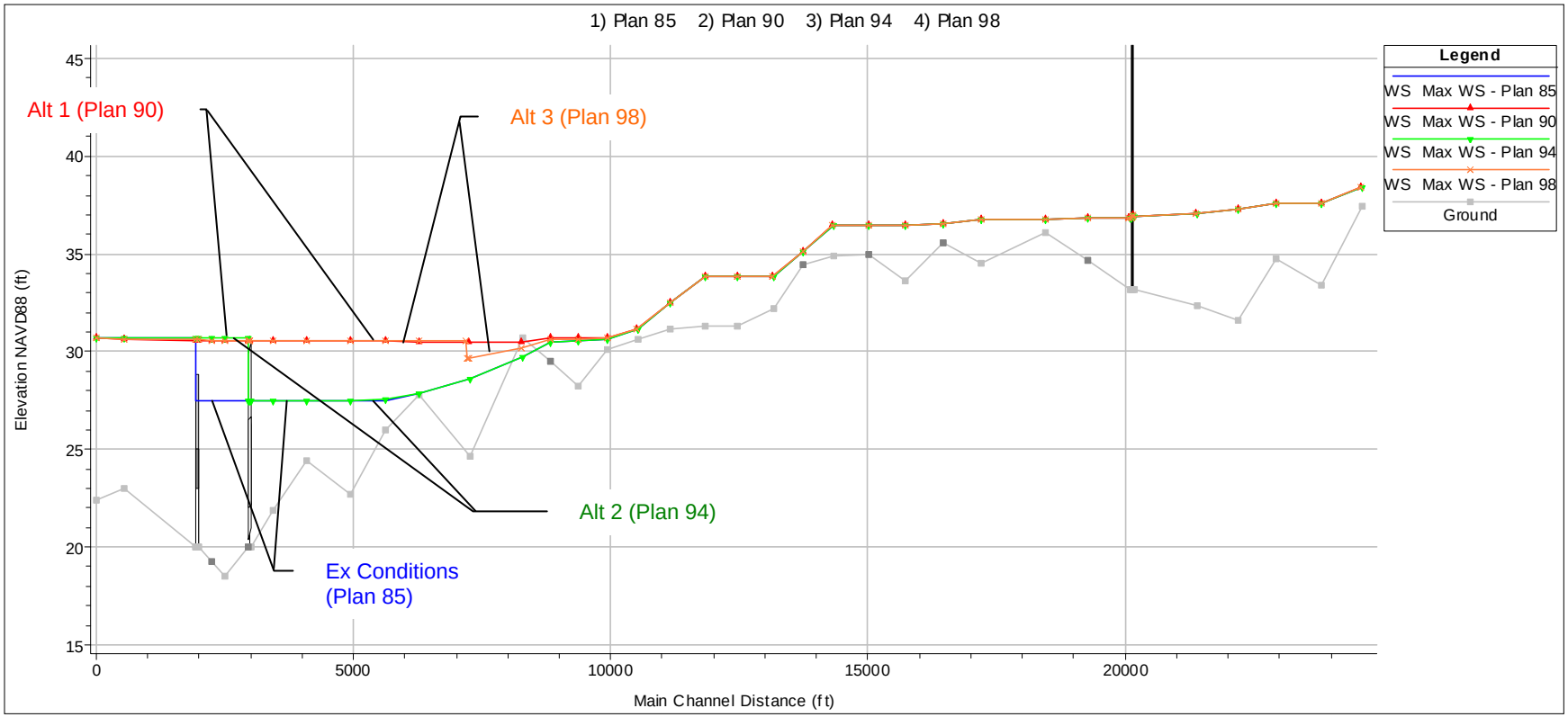

Figure 30. Unsteady flow comparison of Existing Conditions (Plan 85) and Alternatives 1 (Plan 90), 2 (Plan 94), and 3 (Plan 98).

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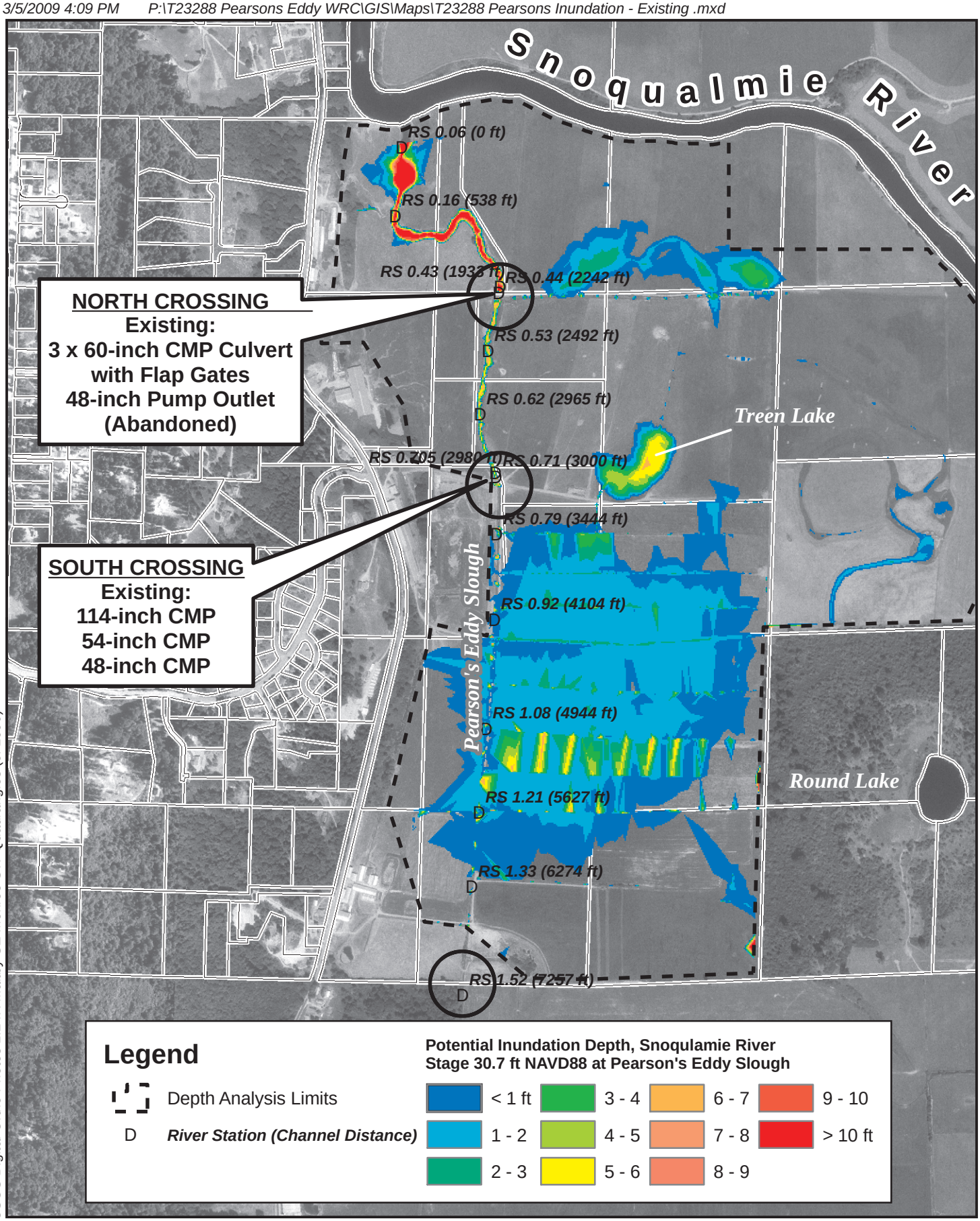

USGS Digital Ortho Photos 1:24K Maltby SE Monroe SW Quadrangles (c. 1990)

0 500 1,000 2,000 Feet

Pearsons Eddy WRC

Inundation Map - Existing Condition

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FIGURE 31

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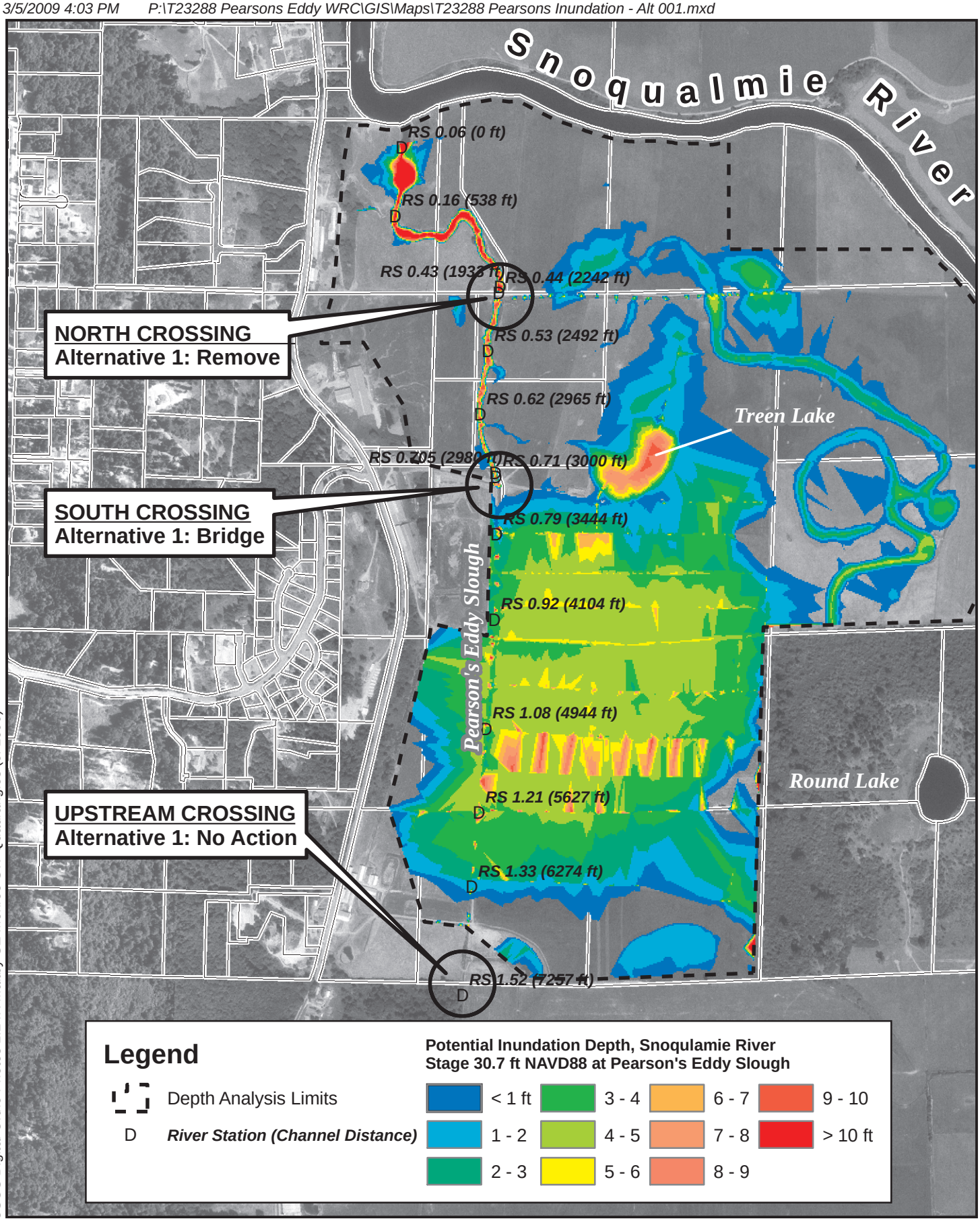

USGS Digital Ortho Photos 1:24K Maltby SE Monroe SW Quadrangles (c. 1990)

0 500 1,000 2,000 Feet

Pearsons Eddy WRC

Inundation Map - Alternative 1

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FIGURE 32

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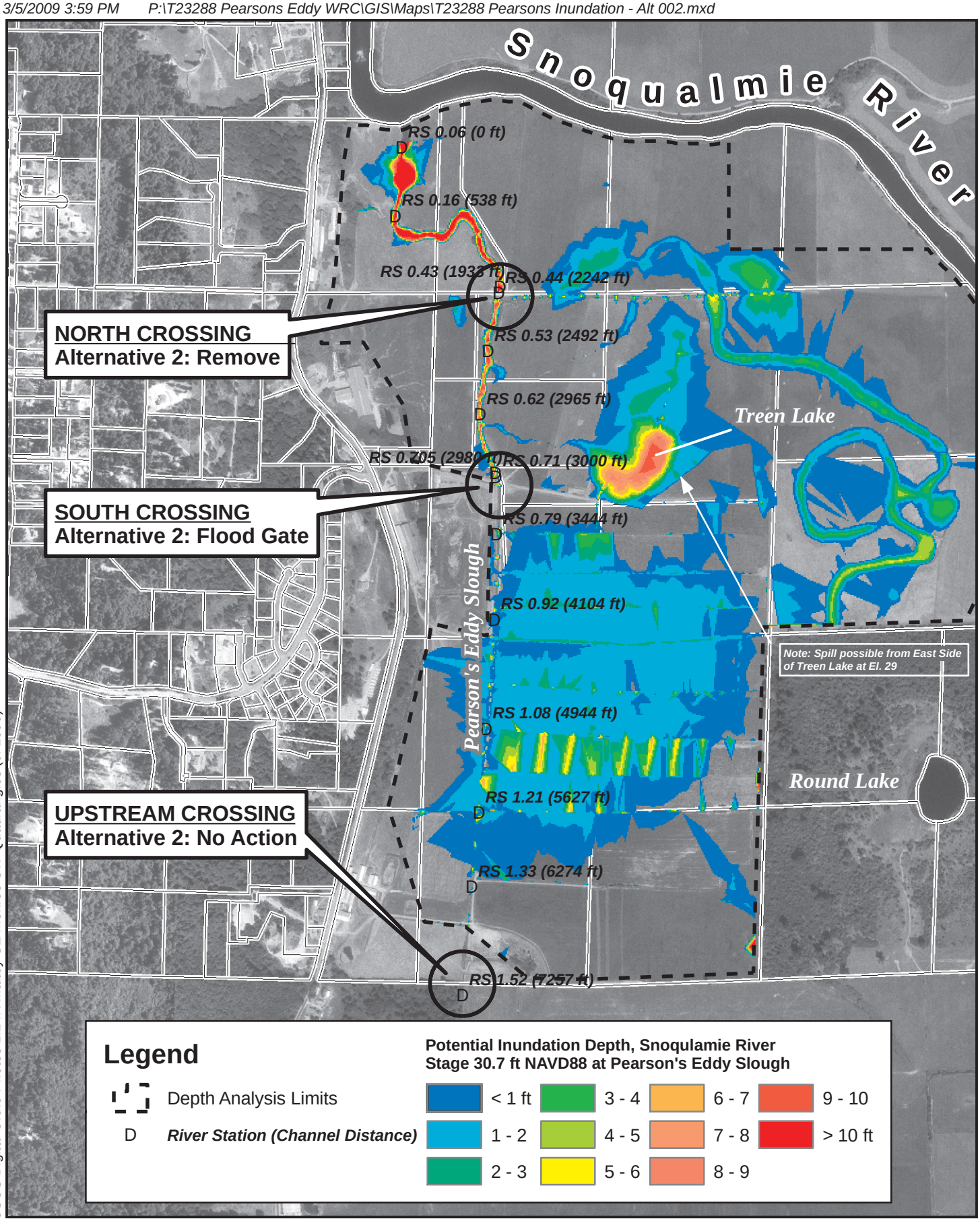

0 500 1,000 2,000 Feet

Pearsons Eddy WRC

Inundation Map - Alternative 2

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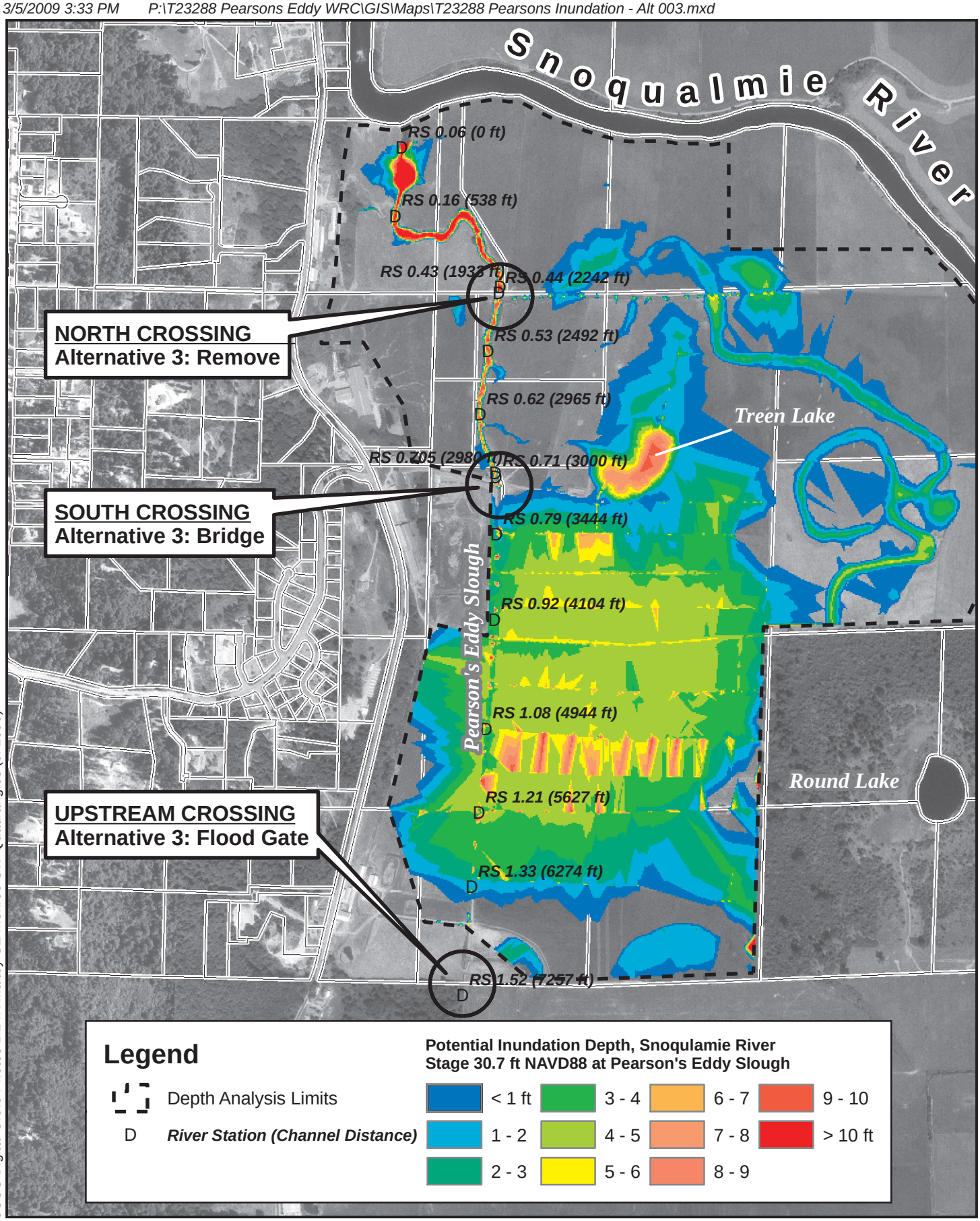

USGS Digital Ortho Photos 1:24K Maltby SE Monroe SW Quadrangles (c. 1990)

0 500 1,000 2,000 Feet

Pearsons Eddy WRC

Inundation Map - Alternative 3

T23288 - MARCH 2009

FIGURE 34

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APPENDIX A

Photo Reconnaissance Log

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APPENDIX B

Pearson's Eddy Project Survey (2008)

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APPENDIX A

Photo Reconnaissance Log

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Meeting Notes

September 25, 2008

Pearson's Eddy Site Visit
Monroe, Washington

Attendees:

Erica Fifer (NRCS)
Kathy Kilcoyne (NRCS)
Marc Schulte (Tetra Tech)

Notes:

Walked the Pearsons Eddy project site near High Bridge Road.

South Crossing – Concrete added on top of crossing to prevent culvert failure, but now concrete is failing also. One of the proposed alternatives would be to replace this with a bridge-type structure. Culvert currently features a large-diameter concrete pipe plus a smaller diameter corrugated metal pipe.

Property to the south is within the conservation bank.

Met one of the survey team members, who were gathering data. They appear to be ahead of schedule.

North Crossing – Farmers used to operate pumps at this location to help keep fields dry. NRCS no longer pumps water here. Kathy noted some of the erosion issues along the downstream bank (right side, looking downstream), which are the result of water re-entering channel after flood events. Some riprap has been placed to slow the erosion, but the problem appears to persist.

Marc described Tetra Tech's general approach to hydraulic modeling analysis. Current FEMA Flood Insurance Study (FIS) models (obtained via Larry Johnson at NRCS) describe the Pearsons Slough as "Overflow Channel 1." The FEMA analysis simulates the hydraulic connection of the slough and the main stem of the Snoqualmie River on the east side of the valley with a lateral weir. The FEMA analysis does not have details such as the culvert crossings on the slough. Tetra Tech's strategy is to update the FIS model, inserting culvert crossings on the slough and modeling adjacent properties as storage areas to calculate relative inundation under different scenarios.

Project communication. Project or site-specific inquiries should be directed to Erica; contract or invoice issues should be forwarded to Larry and Amy in Spokane.

Follow-Up Items:

Marc to forward Erica ftp site access info so Erica can transmit newer aerial photo clip.

Tetra Tech, Inc.
1420 5th Avenue, Suite 550, Seattle, WA 98101
Tel 206.728.9655 Fax 206.728.9670 www.tetratech.com

Site Visit Photo Log

Pearsons Eddy
Monroe, WA

September 25, 2008
Approx. 2:00PM

MAS

001 Looking downstream from south crossing.
002 Looking upstream from south crossing.
003 Upstream side south crossing culvert from right bank.
004 Upstream side south crossing culvert from left bank. Note smaller CMP culvert on far side of large concrete pipe.
005 Upstream side north crossing culvert (grass-covered structure). Pipe in center of picture is from drainage ditch.
006 Upstream side north crossing culvert (grass-covered structure).
007 Downstream face of north culvert crossing (from downstream left bank).
008 Downstream face of north culvert crossing (from downstream left bank).
009 Looking downstream from north culvert crossing.
010 Looking upstream from north culvert crossing.
011 Looking across valley from High Bridge Road, toward east. In distance, hedge row in center of picture is south NRCS property boundary.
012 Looking across valley from High Bridge Road, toward northeast.

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Site Visit Photo Log

October 30, 2008

Pearson's Eddy
Monroe, Washington

Attendees:

Curtis Loeb (Tetra Tech)
Marc Schulte (Tetra Tech)

Notes:

Tetra Tech personnel re-visited the Pearson's Eddy site on October 30, 2008 to review site conditions and investigate hydraulic modeling approach for the Pearson's Eddy Slough and existing culvert crossings.

T23288 Site Visit Notes 2008-10-30.doc

Tetra Tech, Inc.
1420 5th Avenue, Suite 550, Seattle, WA 98101
Tel 206.728.9655 Fax 206.728.9670 www.tetratech.com

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APPENDIX B

Pearson's Eddy Project Survey (2008)

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50 0 50 100

Scale: 1" = 50'

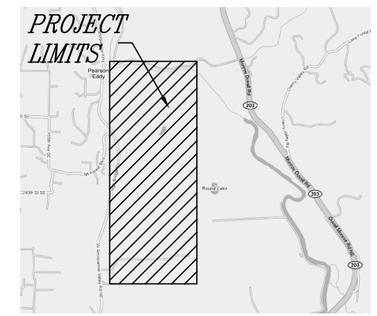
DATUM

WSDOT CONTROL

PNT	NORTH	EAST	ELEV	DESC
2429	286465.470	1362493.824	79.462	GP31203-117
2434	283824.176	1364204.040	49.081	GP17203-158
3246	87492.599	410735.153	0.000	ALLIANT
2453	5000.000	10000.000	47.874	BM31203-67
2430	5500.000	10000.000	47.999	B296
2431	5600.000	10000.000	47.267	C2961947

HORIZONTAL = NAVD 83/91

VERICAL = NAVD 88



VICINITY MAP

NOT TO SCALE

EQUIPMENT AND PROCEDURES

A SOKKIA SET 4 (5 SECOND TOTAL STATION).

SURVEYOR'S CERTIFICATION

THIS MAP CORRECTLY REPRESENTS A SURVEY MADE BY ME OR UNDER MY DIRECT SUPERVISION AT THE REQUEST OF TETRA TECH INC DURING 10-9-08.

DATE:

SIGNED _____
21364, P.L.S.

GARY L. VAN NESS
STATE OF WASHINGTON
21364
REGISTERED PROFESSIONAL LAND SURVEYOR
EXPIRES

REVISIONS

NO.	DATE	DESCRIPTION	BY

SHEET TITLE: TETRA TECH INC
PEARSON EDDY CHANNEL
TOPOGRAPHIC CROSS SECTIONS

CLIENT: Tetra Tech, Inc
Marc Schulte
Snoqualmie river
DUVALL, WA

WHITE SHIELD, INC.
SURVEYING ENVIRONMENTAL SERVICES
23412 68th Ave S.
Kent, WA.
98032
Phone: 253-867-6070
Fax: 425-641-7734

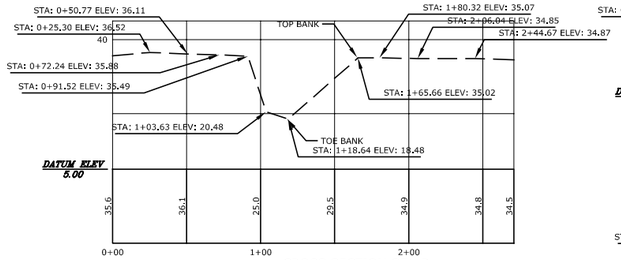
DRAWN BY: NO	CHECKED BY: GLV
SCALE: 1" = 50'	DATE: 10/10/08

JOB NO: 208-045-01

DRAWING NAME: BASEMAP-2007-R1.DWG

SHEET 1 OF 2

File: P:\Projects\Pearsons Eddy\Data\Survey-Whiteshield\CAD-revised 01-06-09 (basemap pdfs not revised)\BASEMAP-2007-R1.dwg ID: curtis.bob Date: 12-Jan-09 2:16:43pm



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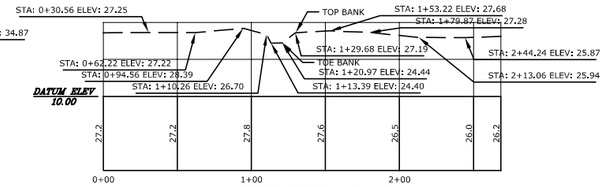
VERT=20'



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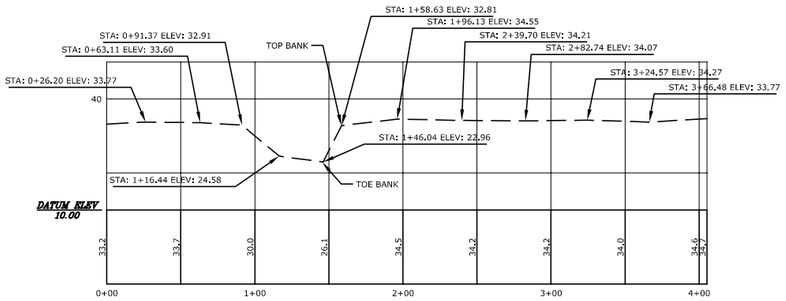

CROSS SECTION OF1-B
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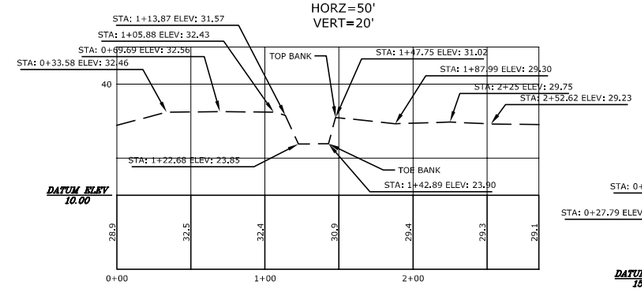
VERT=20'



EXIST NORTH OF SOUTH CULVERT (N)
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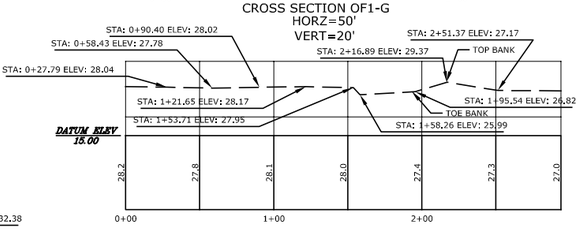
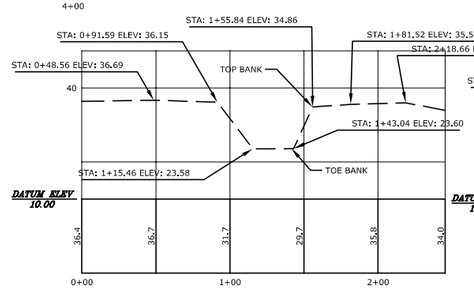
VERT=20'



CROSS SECTION OF1-G
HORZ=50'
VERT=20'

CROSS SECTION OF1-H
HORZ=50'
VERT=20'



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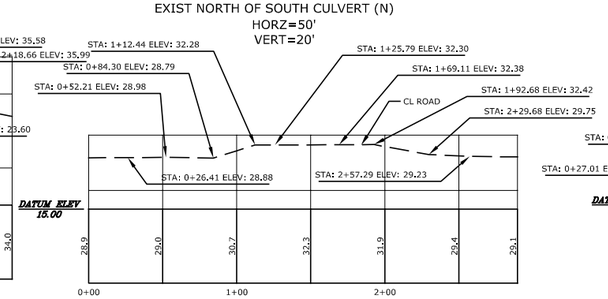
VERT=20'



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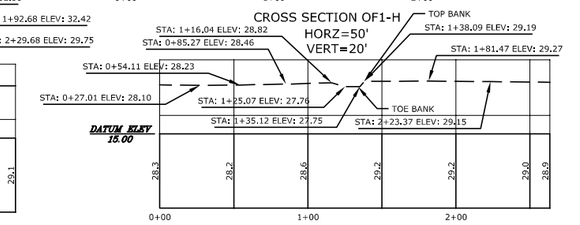
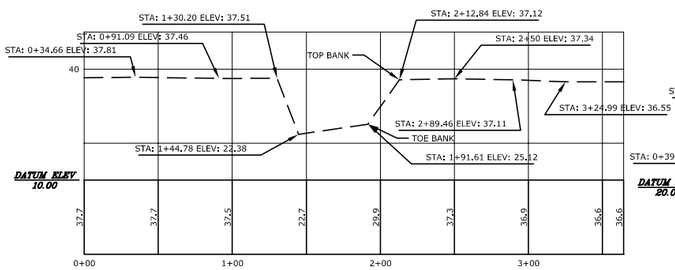
VERT=20'



CROSS SECTION OF 1 EYE
HORZ=50'
VERT=20'



CROSS SECTION OF1-B1
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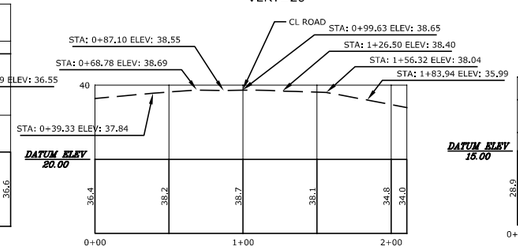

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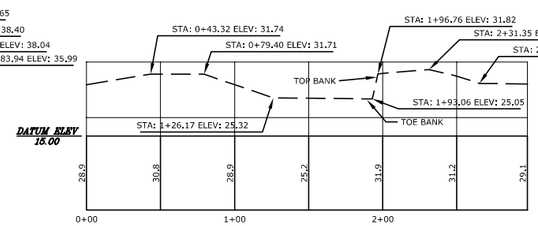

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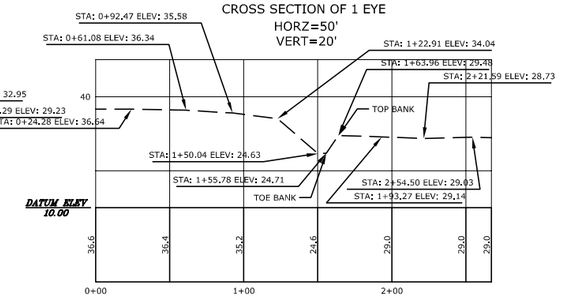
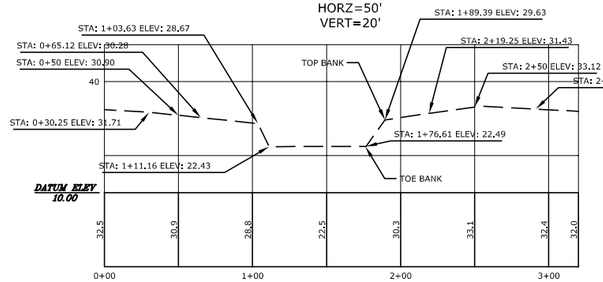
VERT=20'



PROPOSED CULVERT
HORZ=50'
VERT=20'



CROSS SECTION OF1-A
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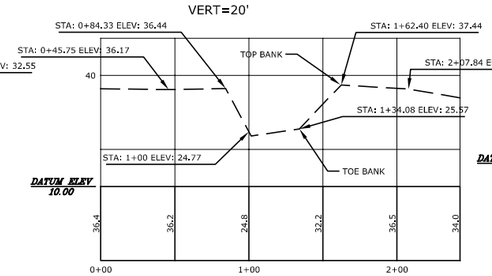
VERT=20'



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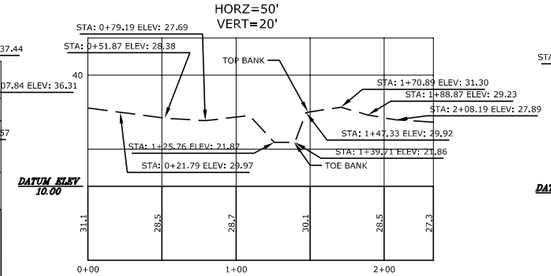

CROSS SECTION OF1-E
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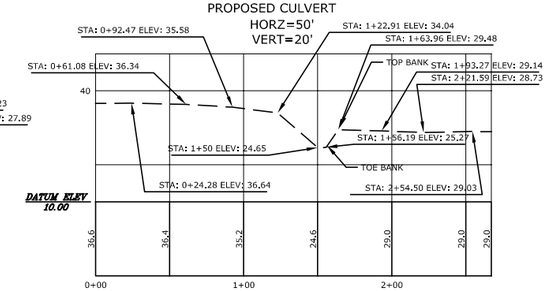

CROSS SECTION I-J
HORZ=50'
VERT=20'

GARY L. VAN NESS
STATE OF WASHINGTON
PROFESSIONAL LAND SURVEYOR
EXPIRES

NO.	DATE	DESCRIPTION	BY

REVISIONS

SHEET TITLE: TETRA TECH INC
PEARSON EDDY CHANNEL
TOPOGRAPHIC CROSS SECTIONS

CLIENT: Tetra Tech, Inc
Marc Schulte
Snoqualmie river
DUVALL, WA

WHITE SHIELD, INC.
23412 68th Ave S.
Kent, WA 98032
Phone: 253-867-6070
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SURVEYING ENVIRONMENTAL SERVICES

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JOB NO: 208-045-01	
DRAWING NAME: BASEMAP-2007-R1.DWG	

SHEET
2 OF 2

File: P:\Projects\Pearsons Eddy\Data\Survey-Whitesheld\DD-revised 01-06-09 (basemap pts not revised)\BASEMAP-2007-R1.dwg ID: curtis.bob Date: 12-Jan-09 2:18:37pm

United States Department of Agriculture

Natural Resources Conservation Service
West National Technology Support Center
1201 NE Lloyd Blvd., Suite 1000
Portland, OR 97232

Hydraulic Impact Analysis

of WRP project features along Pearson Eddy Slough

in the State of Washington, counties of Snohomish and King

ADDENDUM

two-dimensional hydraulic analysis

by Daniel S. Moore, P.E., Hydraulic Engineer

March 2017

Hydraulic Impact Analysis WRP project features along Pearson Eddy Slough

ADDENDUM two-dimensional hydraulic analysis

This addendum discusses further hydraulic analysis of the Wetland Reserve Program (WRP) project features near Pearson Eddy Slough which had received one-dimensional hydraulic analysis in 2012. The 2012 report documents the WRP features and proposed structural improvements at one road crossing on the slough. The 1D analysis of 2012 showed that the flood plain easement (FPE) features installed east of the hunt club property would not result in increased flooding further upstream. The structural improvements at the road crossing was shown to lower upstream flood levels from previously existing conditions.

1. Description of 2012 one-dimensional hydrologic and hydraulic analysis

The hydrologic conditions examined in the 2012 analysis were interior floods caused by local storm events, with stages on the Snoqualmie River low enough that no overbank flow from the main river would exist. An earlier study by TetraTech had analyzed the impact of mainstem Snoqualmie River flooding that would overtop its banks in the Pearson Eddy vicinity.

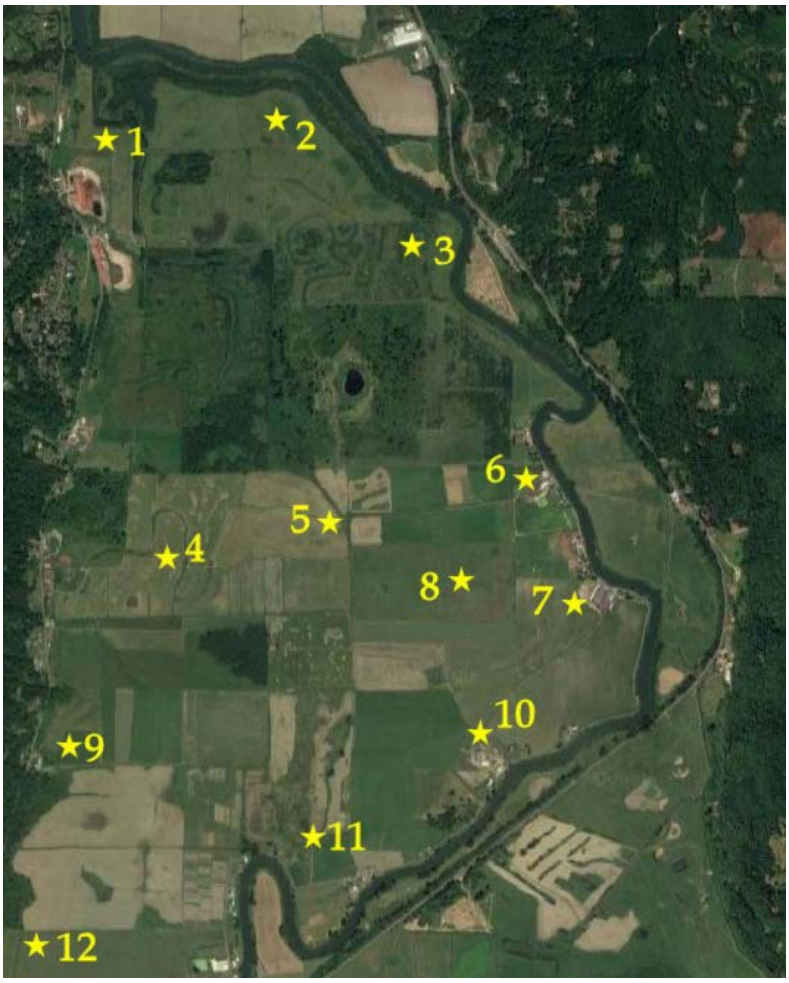
The 1D hydraulic analysis was performed using HecRAS, the Corps of Engineers hydraulic model, the same software used by TetraTech. The version of HecRAS used in 2012 did not have two-dimensional flow analysis capabilities, but in 2016 a new version (5.0) of HecRAS was released, with the addition of these advanced features.

2. Comparison of 1D versus 2D modeling for Pearson Eddy Slough

The physical characteristics of the Pearson Eddy Slough drainage area indicate that two dimensional hydraulic modeling should provide more accurate results compared to one-dimensional modeling. The very wide floodplain and very low velocity overbank flow during even large floods indicate that the unsteady flow storage effects would be better accounted for by momentum equations in the two-dimensional unsteady flow analysis.

The 2D HecRAS modeling of the Pearson Eddy Slough drainage area confirms that for events in which the Snoqualmie River does not overtop its banks, the WRP project vegetation has no effect. However, this analysis does show that larger Snoqualmie River events, such as the 10-year, 50-year, and 100-year floods, do experience a very small rise due to WRP project vegetation. The flood peak results are shown in Tables 1 (depths) and 2 (water surface elevations), with output locations shown in Figure 1. The ground conditions analyzed are labeled 2007, 2010, and 2017, with details of the ground conditions in the vegetation section below.

Figure 1. Pearson Eddy Slough drainage area, HecRAS output locations for Tables 1 and 2.



The output tables show the results of five different runs (run numbers 1-5) which increase in peak flood flowrate and are described as follows:

run 1: 12,000 cfs peak flow on the Snoqualmie River, (exceeding banks only briefly), along with a 5-year flood event on Tuck Creek, the local drainage to the west, and a 5-year precipitation event on the Snoqualmie floodplain area of Pearson Eddy.

The remaining runs do not include local runoff or local precipitation, and Snoqualmie River floods are taken from the analysis by TetraTech for FEMA.

run 2: 34,000 cfs peak flow on the Snoqualmie
run 3: 55,000 cfs peak flow on the Snoqualmie, a 10-year event, according to TetraTech
run 4: 78,000 cfs peak flow on the Snoqualmie, a 50-year event, according to TetraTech
run 5: 87,000 cfs peak flow on the Snoqualmie, a 100-year event, according to TetraTech

Table 1. HecRAS output flood depths at locations in Figure 1,
(run numbers refer to flood levels described in the above paragraph)

run: ground conditions	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
	point #1					point #2					point #3				
2007 (depth, ft.)	0.85	7.71	11.31	14.71	16.08	5.97	12.48	15.78	19.06	20.38	4.29	10.86	14.16	17.44	18.76
2010 (depth, ft.)	0.86	7.81	11.35	14.73	16.06	5.98	12.83	15.92	19.14	20.44	4.30	11.20	14.30	17.52	18.83
rise (2010-2007 inches)	0.0	1.2	0.5	0.3	-0.2	0.1	4.2	1.7	1.0	0.8	0.1	4.2	1.6	1.0	0.8
2017 (depth, ft.)	0.86	7.80	11.35	14.73	16.06	5.97	12.83	15.92	19.14	20.44	4.29	11.23	14.32	17.54	18.84
rise (2017-2010, inches)	0.0	-0.1	0.0	0.0	0.0	-0.1	0.1	0.0	0.0	0.0	-0.1	0.3	0.3	0.2	0.2
	point #4					point #5					point #6				
2007 (depth, ft.)	1.22	7.92	11.22	14.50	15.81	6.29	12.97	16.27	19.55	20.86	2.67	9.36	12.65	15.93	17.24
2010 (depth, ft.)	1.23	8.29	11.39	14.62	15.92	6.30	13.34	16.44	19.67	20.97	2.67	9.72	12.82	16.05	17.33
rise (2010-2007 inches)	0.1	4.4	2.0	1.4	1.3	0.1	4.4	2.1	1.5	1.3	0.1	4.3	2.1	1.5	1.1
2017 (depth, ft.)	1.23	8.32	11.42	14.65	15.94	6.29	13.37	16.47	19.70	20.99	2.67	9.75	12.85	16.06	17.38
rise (2017-2010, inches)	-0.1	0.3	0.3	0.3	0.3	-0.1	0.3	0.3	0.3	0.3	-0.1	0.4	0.3	0.1	0.6
	point #7					point #8					point #9				
2007 (depth, ft.)	3.44	10.13	13.43	16.70	18.02	4.54	11.23	14.52	17.80	19.11	1.16	7.90	11.19	14.46	15.77
2010 (depth, ft.)	3.45	10.50	13.70	16.83	18.13	4.55	11.60	14.69	17.92	19.22	1.17	8.26	11.36	14.58	15.87
rise (2010-2007 inches)	0.1	4.4	3.2	1.5	1.3	0.1	4.4	2.1	1.5	1.3	0.1	4.2	2.0	1.4	1.3
2017 (depth, ft.)	3.45	10.53	13.62	16.85	18.14	4.54	11.63	14.72	17.95	19.25	1.16	8.29	11.38	14.60	15.90
rise (2017-2010, inches)	-0.1	0.3	-1.0	0.3	0.1	-0.1	0.3	0.4	0.3	0.3	-0.1	0.3	0.3	0.3	0.3
	point #10					point #11					point #12				
2007 (depth, ft.)	4.96	11.66	14.95	18.24	19.54	1.25	7.17	10.48	13.75	15.06	3.15	8.30	11.46	14.67	15.95
2010 (depth, ft.)	4.97	12.03	15.13	18.35	19.65	1.25	7.54	10.65	13.87	15.17	3.15	8.59	11.61	14.78	16.05
rise (2010-2007 inches)	0.1	4.4	2.1	1.4	1.3	0.0	4.4	2.1	1.5	1.3	0.0	3.5	1.8	1.3	1.2
2017 (depth, ft.)	4.97	12.06	15.16	18.38	19.68	1.25	7.57	10.68	13.90	15.19	3.15	8.61	11.63	14.81	16.07
rise (2017-2010, inches)	-0.1	0.3	0.4	0.3	0.3	0.0	0.3	0.3	0.3	0.3	0.0	0.3	0.3	0.3	0.3

Table 2. HecRAS output water surface elevations at locations in Figure 1,
(run numbers refer to flood levels described in the above paragraph)

run: ground conditions	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
	point #1					point #2					point #3				
2007 (w.s. elev)	34.06	40.92	44.52	47.91	49.28	34.97	41.48	44.78	48.06	49.38	34.97	41.54	44.84	48.12	49.44
2010 (w.s. elev)	34.06	41.02	44.56	47.93	49.27	34.98	41.83	44.92	48.15	49.44	34.98	41.88	44.97	48.20	49.50
rise (2010-2007 inches)	0.0	1.2	0.5	0.3	-0.2	0.1	4.2	1.7	1.0	0.8	0.1	4.2	1.6	1.0	0.8
2017 (w.s. elev)	34.06	41.01	44.55	47.93	49.26	34.97	41.83	44.92	48.14	49.44	34.97	41.91	45.00	48.22	49.52
rise (2017-2010, inches)	0.0	-0.1	0.0	0.0	0.0	-0.1	0.1	0.0	0.0	0.0	-0.1	0.3	0.3	0.2	0.2
	point #4					point #5					point #6				
2007 (w.s. elev)	34.97	41.67	44.97	48.25	49.56	34.98	41.66	44.96	48.24	49.55	34.98	41.68	44.96	48.24	49.57
2010 (w.s. elev)	34.98	42.04	45.14	48.37	49.67	34.99	42.03	45.13	48.36	49.66	34.99	42.03	45.14	48.37	49.66
rise (2010-2007 inches)	0.1	4.4	2.0	1.4	1.3	0.1	4.4	2.1	1.5	1.3	0.1	4.3	2.1	1.5	1.1
2017 (w.s. elev)	34.98	42.07	45.17	48.40	49.69	34.99	42.06	45.16	48.39	49.68	34.99	42.06	45.16	48.38	49.70
rise (2017-2010, inches)	-0.1	0.3	0.3	0.3	0.3	-0.1	0.3	0.3	0.3	0.3	-0.1	0.4	0.3	0.1	0.6
	point #7					point #8					point #9				
2007 (w.s. elev)	34.98	41.68	44.98	48.25	49.56	34.98	41.67	44.97	48.25	49.56	35.02	41.76	45.05	48.32	49.63
2010 (w.s. elev)	34.99	42.04	45.25	48.37	49.67	34.99	42.04	45.14	48.37	49.67	35.03	42.12	45.22	48.44	49.73
rise (2010-2007 inches)	0.1	4.4	3.2	1.5	1.3	0.1	4.4	2.1	1.5	1.3	0.1	4.2	2.0	1.4	1.3
2017 (w.s. elev)	34.99	42.07	45.17	48.40	49.69	34.99	42.07	45.17	48.40	49.70	35.02	42.14	45.24	48.46	49.76
rise (2017-2010, inches)	-0.1	0.3	-1.0	0.3	0.1	-0.1	0.3	0.4	0.3	0.3	-0.1	0.3	0.3	0.3	0.3
	point #10					point #11					point #12				
2007 (w.s. elev)	34.98	41.68	44.98	48.26	49.57	35.78	41.71	45.01	48.28	49.60	37.08	42.23	45.39	48.60	49.88
2010 (w.s. elev)	34.99	42.05	45.15	48.38	49.67	35.78	42.07	45.19	48.41	49.70	37.08	42.52	45.54	48.72	49.98
rise (2010-2007 inches)	0.1	4.4	2.1	1.4	1.3	0.0	4.4	2.1	1.5	1.3	0.0	3.5	1.8	1.3	1.2
2017 (w.s. elev)	34.99	42.08	45.18	48.40	49.70	35.78	42.10	45.21	48.43	49.73	37.08	42.54	45.57	48.74	50.00
rise (2017-2010, inches)	-0.1	0.3	0.4	0.3	0.3	0.0	0.3	0.3	0.3	0.3	0.0	0.3	0.3	0.3	0.3

3. Two-dimensional model details

Two-dimensional modeling requires a very accurate definition of the land surface, which can be accomplished with LiDAR. Using this available digital elevation information, an extensive grid was developed, to represent the entire floodplain from about rivermile 12.4, upstream of Duvall to rivermile 2.7 downstream of the Pearson Eddy outlet. (See Figures 2 and 3.) The nominal cell size of this grid was 200-foot square. The Snoqualmie River centerline and river mile delineation is shown in Figures 2 and 3 below, with background photo from 2016. Note that these river miles match the TetraTech study of 2007, but not those shown on the older USGS topo maps, because of natural changes in the river alignment, particularly near the confluence with the Skykomish River.

Figure 2. Snoqualmie River, with the Pearson Eddy outlet near river mile 3.0.

The previous HecRAS analysis performed by TetraTech for FEMA included the entire Snoqualmie-Skykomish-Snohomish river system. The model was 1D unsteady flow and took into account the fact that floods on the Skykomish will back water up the Snoqualmie to and above the Pearson Eddy outlet.

Figure 3. Snoqualmie River, with the bridge into Duvall between river miles 9 and 9.5.

TetraTech developed inflow hydrographs for the upstream ends of the Skykomish-Snoqualmie-Snohomish hydraulic model, along with smaller inflows from other small contributing watersheds, such as Cherry Creek (tributary of the Snoqualmie downstream of Duvall, entering at about river mile 6.5) and Riley Slough (tributary of the Skykomish). The analysis showed the backwater effect up the Snoqualmie from the Skykomish.

For the current 2D model, the upstream inflows were taken from the TetraTech model output, as the flow hydrographs at river mile 12.4, and downstream stage hydrographs were taken from the TetraTech output at river mile 2.73. (These coincided with cross-sections of that model and formed the upstream and downstream ends of the 2D grid of the current model.) The 1D TetraTech analysis included numerous bridges on these rivers and modeled the very wide Snoqualmie floodplain with cross-sections in the Pearson Eddy Slough area.

4. Ground conditions modeled

A 2005 aerial photo shows the ground conditions with no added vegetation. (See Figure 4.) The Mannings roughness values for this condition were determined from the National Land Cover Dataset (NLCD) layer for this area, by assigning conventional roughness values to land cover and vegetation types. Table 3 shows vegetation types in the Pearson Eddy Slough area and associated Mannings roughness values used in the 2D model.

Table 3. NLCD vegetation types and Mannings roughness at Pearson Eddy Slough

land cover	Mannings roughness
forest	0.12
hay/pasture	0.05
grassland/herbaceous	0.05
woody wetlands	0.12
emergent herbaceous wetlands	0.08

The 2007 ground conditions assume that the wetland mitigation bank (also called Habitat Bank) property has been fully vegetated with a majority of wetland, non-woody plants, and that vegetation on the NRCS easements has not been altered from previous years. (See Figure 4.) The 2010 ground conditions assume that NRCS flood plain easement vegetation has been planted with a majority of woody species, such as cottonwood and alder. (See figure 5.) The 2017 ground conditions assume further NRCS vegetation plantings, in the wetland reserve program easement, with that parcel receiving mostly non-woody vegetation. (See figure 5.)

For all model runs the vegetation is assumed to have fully grown to maturity.

Figure 4. Vegetation conditions in 2005 (from Google Earth Pro at left) and conditions modeled as 2007 (over 2017 image, at right). Mannings roughness of 0.08 assumed for the Habitat Bank.

Figure 5. Below, at left, conditions modeled as 2010 (over 2017 image), with Mannings roughness assumed at 0.12 for woody areas and 0.08 for non-woody areas. At right, proposed vegetation, modeled as 2017 conditions.

5. Interpretation of output

Three modeling details should be understood for proper interpretation of the 2D results:

1) The larger floods, such as the 100-year, are high enough to overtop highway 203 on the right overbank(east side), inundate further into the Duvall area and up Cherry Valley. Lack of LiDAR terrain for this area precluded the 2D analysis from modeling the full extent of this area for the larger floods. (Cherry Creek enters the Snoqualmie at about river mile 6.5.)

2) The bridge decks for the various openings along Woodinville-Duvall road, including the bridge over the Snoqualmie at Duvall were not included in the 2D analysis. Any higher flood event which would contact these bridge decks will not be precisely represented in the 2D analysis.

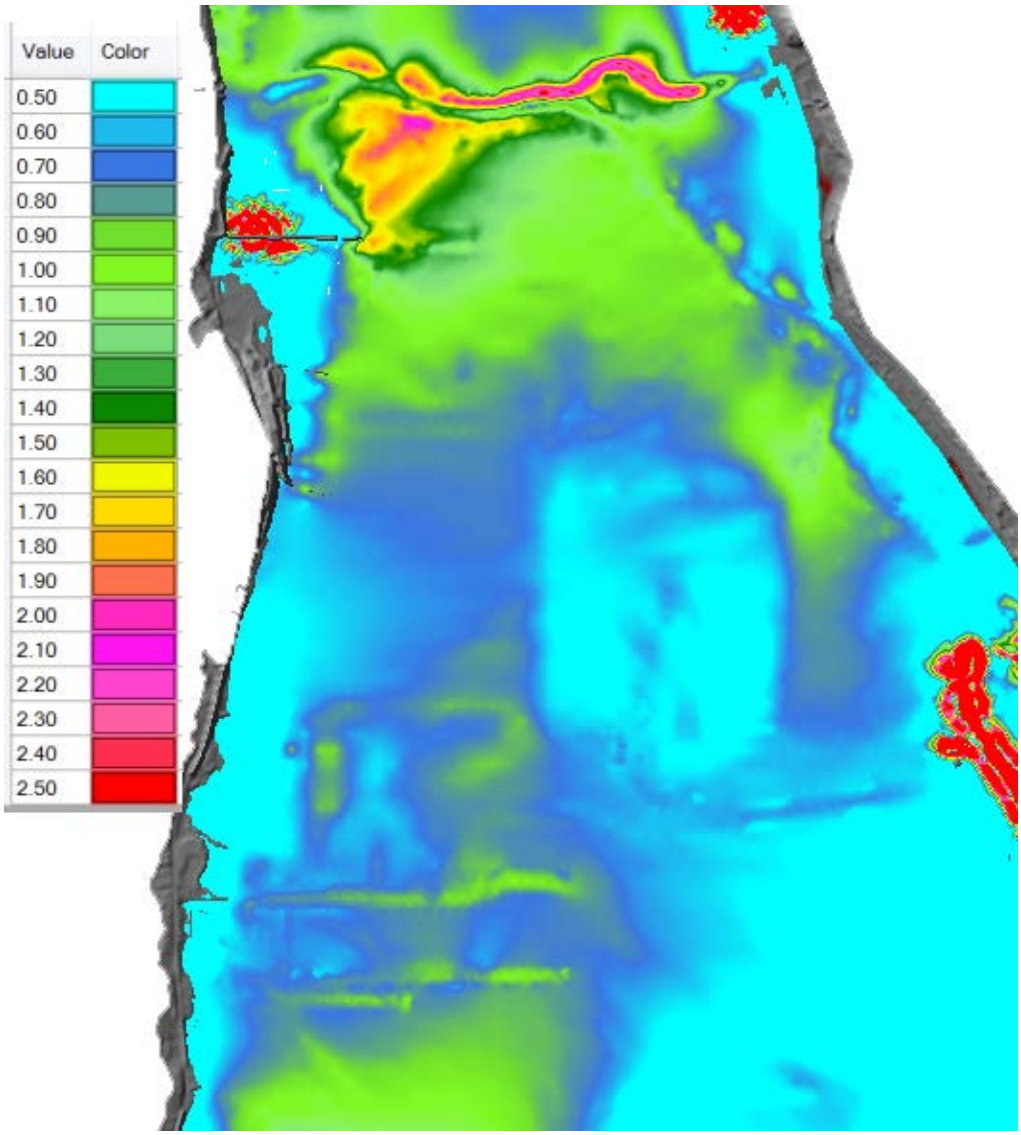
3) Various locations along the Snoqualmie banks between river miles 12.4 and 2.7 may have drainage culverts or side creeks that drain the floodplain or allow the river to flow into it. These locations were reprented in the model by the LiDAR terrain, without attempting to represent culverts, with or without flap gates.

One exception to item 3) is a drainage ditch entering the Snoqualmie River in the vicinity of river mile 5.5 from the left bank (west side). At this location, a 30-inch diameter culvert with a flap gate located on private land was assumed to be operational. In addition, the NRCS project conditions include upgrades to the culverts at the outlet of Pearson Eddy Slough, with operational flap gates.

Due to the modeling issues discussed above, the analysis is not intended to represent the precise flood levels of these Snoqualmie River flood events in the Pearson Eddy Slough drainage, but only to show the effect of NRCS project vegetation for slough drainage flows of these magnitudes.

Flow velocities across the Pearson Eddy Slough floodplain of the Snoqualmie River are relatively low at all flood levels, and in many locations under one foot per second. As a measure of the highest flow velocities experienced, Figure 7 shows the 100-year flood, 2007 ground conditions, and velocities near the hydrograph peak. (The higher velocities, in red, are over two feet per second, but are confined to the Snoqualmie River channel or areas of very shallow flow depth.) Note that the entire valley is inundated.

Figure 7. Flow velocities (feet/second) during the 100-year peak flow, 2007 ground conditions.



To examine the possibility that the woody vegetation planted on the flood plain easement might redirect the flow southerly, from the easement in Snohomish County to King County lands further south, Figure 8 shows that flow directions remain northerly. The figure is a zoomed in view of HecRAS velocity output for the area east and south of the hunt club, with the club access road shown horizontally in the lower third (2016 photo). In HecRAS, the particle tracing lines move in the flow direction and leave diminishing trails to their rear. The figure is a screenshot showing flow directions steadily to the north. The output is from the 34,000 cfs peak flood, 2017 ground conditions, (including plantings planned for 2017) and all velocity magnitutes in this view are under 0.5 feet per second.

Figure 8. Flow velocity particle tracing lines in thearea south and east of the hunt club, showing northerly flow during the 34,000 cfs peak, 2017 ground conditions. All velocity maginitudes are under 0.5 feet per second. The background photo is from 2016.

The output tables show that the flood level most effected by vegetation plantings is the 34,000 cfs flood, rather than the larger 50-year and 100-year floods. The best way to interpret this result is to visualize that deeper flows, even at a low velocity, have more momentum due to their greater mass, and can more easily overcome the resistance of ground roughness.

To verify that the flood water surfaces are relatively flat, note in table 2 the difference in water surface elevation at points 3 and 10 (about 1.5 miles apart) from Figure 1. For all Snoqualmie River overbank floods, they differ by less than 2 tenths of a foot (about 2.5 inches).

Conclusion

The two dimensional hydraulic analysis confirms the previous studies. The Pearson Eddy drainage area behaves much like a bathtub. The area is mildly sloped, and the downstream end of the slough is like the bathtub drain. During local flood events and storms when the Snoqualmie River does not exceed its banks, the vegetation, existing on floodplain easements

and proposed on the WRP easement, has no effect on flood levels. During larger floods when the Snoqualmie River does exceed its banks, the vegetation can cause a very small rise in peak flood levels, as shown in the output tables, adding generally less than two inches to floodplain water depths of many feet. For example, in the middle of the floodplain, at location 5 in Figure 1, the peak depths shown in Table 1 are at or above 13 feet for all overbank floods.