Appendix B

Hydrology and Hydraulics

Puyallup River Basin
Flood Risk Management Feasibility Study

Department of the Army
Seattle District, US Army Corps of Engineers

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Hydrology and Hydraulics Appendix

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Hydrology and Hydraulics

Hydraulic Modeling of Existing Conditions

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1.0 Purpose and Overview

This appendix documents the work conducted for the Puyallup River Flood Risk Reduction Feasibility Study to develop the existing conditions hydraulic computational model. The immediate use of these models will be to establish existing without project hydraulic conditions. Future with- and without project conditions hydraulic modeling efforts are documented in a separate H&H appendix and built off of the existing conditions modeling described here. Additionally, support for the FDA analysis is described in a separate appendix.

This appendix documents the following:

- Description of the hydraulic analysis methodology
- Development of the HEC-RAS hydraulic computational model
- Calibration and validation of the hydraulic computational model
- Development of basin statistical hydrology
- Modeling of statistical hydrology in the hydraulic computational model

The hydraulic model was developed and calibrated to several recent flood events, and then run with statistical hydrology to model extreme flood events. The hydraulic model is discussed first, followed by statistical hydrology, and then lastly modeling of extreme flood events.

2.0 Puyallup River Basin and Scope of Study

The Puyallup River Basin covers about 948 sq. mi. of western-central Washington. The Puyallup River Basin drains the upland area surrounding Mount Rainier and lowland areas including the developed areas of Auburn, Benroy, Algona, Pacific, Dieringer, Sumner, Puyallup, Tacoma, Fife, Alderton, McMillin, Meeker, Orting, and Crocker into the Commencement Bay area of Puget Sound. The major rivers of this basin are: the Puyallup River, Carbon River (tributary to the Puyallup), South Prairie Creek (tributary to the Carbon), White River (tributary to the Puyallup), and the Greenwater River (tributary to the White). For purposes of routing floodwater between reaches and investigating existing conditions flood levels and frequencies in the GI Study area, the Puyallup River Basin was divided into the following reaches:

- Upper Puyallup: The portion of the Puyallup River starting 1 mile downstream of Electron Dam extending to the junction with the Carbon River, covering 11.4 miles
- Middle Puyallup: The portion of the Puyallup River between the Carbon River and the White River junctions, covering 7.2 miles
- Lower Puyallup: The portion of the Puyallup River downstream of the junction with the White River extending to Commencement Bay, covering 10.2 miles

- White River: The portion of the White River downstream of Auburn, covering 10.4 miles
- The Carbon River: The portion of the Carbon River starting 7 miles downstream of Fairfax, covering 8.3 miles

Reaches in the hydraulic model extend only as far upstream as necessary to capture the project area. The upstream boundary of the Puyallup, Carbon, and white River modeling terminates between existing streamgages. The synthetic hydrology as been developed to capture conditions at the model boundaries. South Prairie creek is not explicitly included, but the hydrology is accounted for as a Carbon River inflow.

The character of the rivers in the study area varies tremendously. The bed slope changes from greater than 1% at the steepest reaches of the Carbon and Upper Puyallup, to 0.5% on the upper reaches of the White, and terminates at a bed slope of 0. 06% at the Lower reach of the Puyallup. Bed elevations range from approximately 550 ft. at the Upper Puyallup to -10 ft. (NAVD 88) at the Lower Puyallup. Land uses vary throughout the modeled reaches including: forested, pasture, wetland, agricultural, prairie, industrial, commercial, and residential. Floodplain areas vary considerably in land slope and connectivity with the main channel. Each river reach is leveed to varying degrees with the Lower Puyallup completely leveed and only a few levees on the upper areas of each river.

A horizontal datum of North American 1983 Harn and a vertical datum of NAVD88 were used for all modeling. The spatial reference used in GIS modeling was NAD 1983 Harn Washington State Plane South. River stationing for the Puyallup begins at Commencement bay, for the Carbon begins at its junction with the Upper Puyallup, and for the White begins at its junction with the Middle Puyallup.

3.0 Hydraulic Analysis

Hydraulic modeling of the Puyallup River basin for this study began with the use of HEC-RAS version 4.1. As the study progressed modeling was transitioned to version 5.0.0 Beta. The developed model encompasses all river reaches described previously and their associated floodplains. A total of 428 cross sections are used throughout the system. For purposes of this phase of the study, the one dimensional nature of HEC-RAS is appropriate for in-channel flows of the system. All major bridges (34) are included in the model. Where appropriate, cross sections extend to cover adjacent floodplain areas. This is appropriate when overbank areas are well connected with, and can experience the same water surface elevation as, the main channel. This is however, seldom the case in the heavily urbanized areas of the model such as Tacoma, Orting, pacific, and Sumner. The complex geologic history of the region has created a river system and associated floodplain that vary considerably. To approximate the 2-dimensional character of the observed flooding in these damage centers, a cascading storage area approach was used, in conjunction with lateral weirs at levee locations, to route overbank flows through the floodplain. The goal of this approach is to more realistically estimate overbank flood elevations under conditions when the river may be at a higher stage than the floodplain. Storage areas were linked together at roads, berms, and natural high ground so that overbank flood levels would gradually step down the sloping valleys and spread out laterally across flatter areas of the floodplain.

A total of 185 storage areas were used throughout the system. Storage areas were created behind most levees (federal and non-federal). Storage areas are connected to the main riverine reaches of the model by lateral structures. Of the 113 lateral structures used, 47 are sections of existing levees. Adjacent storage areas are connected by storage area connections giving, in effect, cascading pools. This approach allows for varying water surfaces and better accounting for floodplain storage. It is not entirely accurate, however, where land slopes are high and water can flow with significant velocity across the terrain. Areas where this can occur in the model are the far upper reaches of the Puyallup, Carbon, and White Rivers. These areas have sparse development and agriculture, and have been left as cross sections. Dividing features between storage areas included highways, railroad grades, and other elevated areas that gave distinct separations between storage areas. The storage area approach, while reasonably quantifying the volume of water that may exist between significant features that divide the floodplain, does not accurately capture where the water travels as it moves to the lowest point within each storage area. This modeling approach was discussed extensively with supervision and HEC at the beginning of the GI study. It was decided that the multiple storage area approach will provide acceptable accuracy for determining existing and future with-out project damages and consequences and will better work in tandem with FDA than the 2-dimensional approach. If greater resolution in floodplain damages is needed the modeling approach can be re-evaluated at later phases of the study on a reach-by-reach basis.

3.1 HEC-RAS Model Development

Geometric features for the model (cross sections, storage areas, storage area connections, bridges, and lateral structures) were developed using HEC-GeoRAS. Initial creation of these features from LiDAR data served to geo-reference their location within the study area for later inundation mapping (note that lateral structures are not geo-referenced within HEC-RAS). Where available, survey data was merged into features to provide better resolution for the main channel, bridges, and levees. When all the geometry features were completed in the model, steady 50% AEP (Annual Exceedence Probability) flows for each reach were run to set the bank stations to the active channel width. Bank stations were further refined from Orthographic photos. Steady flows were used to get the model running and resolve errors and warnings. Because of the complexity of the system, the majority of work on the model was done with unsteady flows.

3.1.1 Geometry

The base model geometry was laid out in ArcGIS using HEC-GeoRAS to locate features, and later imported into HEC-RAS. A significant amount of survey data was used to improve the model over its base characteristics extracted from LiDAR terrain data. Surveyed cross section data was merged with LiDAR overbanks within GIS and Surveyed levee profiles were developed into lateral structure features in GIS; additional data manipulation was accomplished within HEC-RAS. Several sources were used to construct model features and are summarized in Table 1.

Table 1: Data Sources

Feature	Data Source
Cross section- floodplain	Pierce County 2009 LiDAR
Cross section- channel surveys	USGS 2010, King County 2009 (White R mi 5-10.5), FEMA 2007
Bridge surveys	FEMA 2007, NHC 2009
Lateral structures (levee survey)	USACE Seattle District
Storage areas	Pierce County 2009 LiDAR
Storage area connections	Pierce County 2009 LiDAR

All model features were incorporated and finalized within HEC-RAS and the model schematic is shown in Figure 1. Provided that flows are high enough, water is capable of overtopping lateral structures of the model and moving into the storage areas shown in the figure. A small pilot channel one foot wide by a few feet deep was created within the main channel throughout the riverbed to improve computational stability of the model. This feature was created with a specific tool in HEC-RAS, the pilot channel tool. The small channel does not affect results significantly but improves model stability considerably, and is located everywhere the river channel is shown in Figure 1.

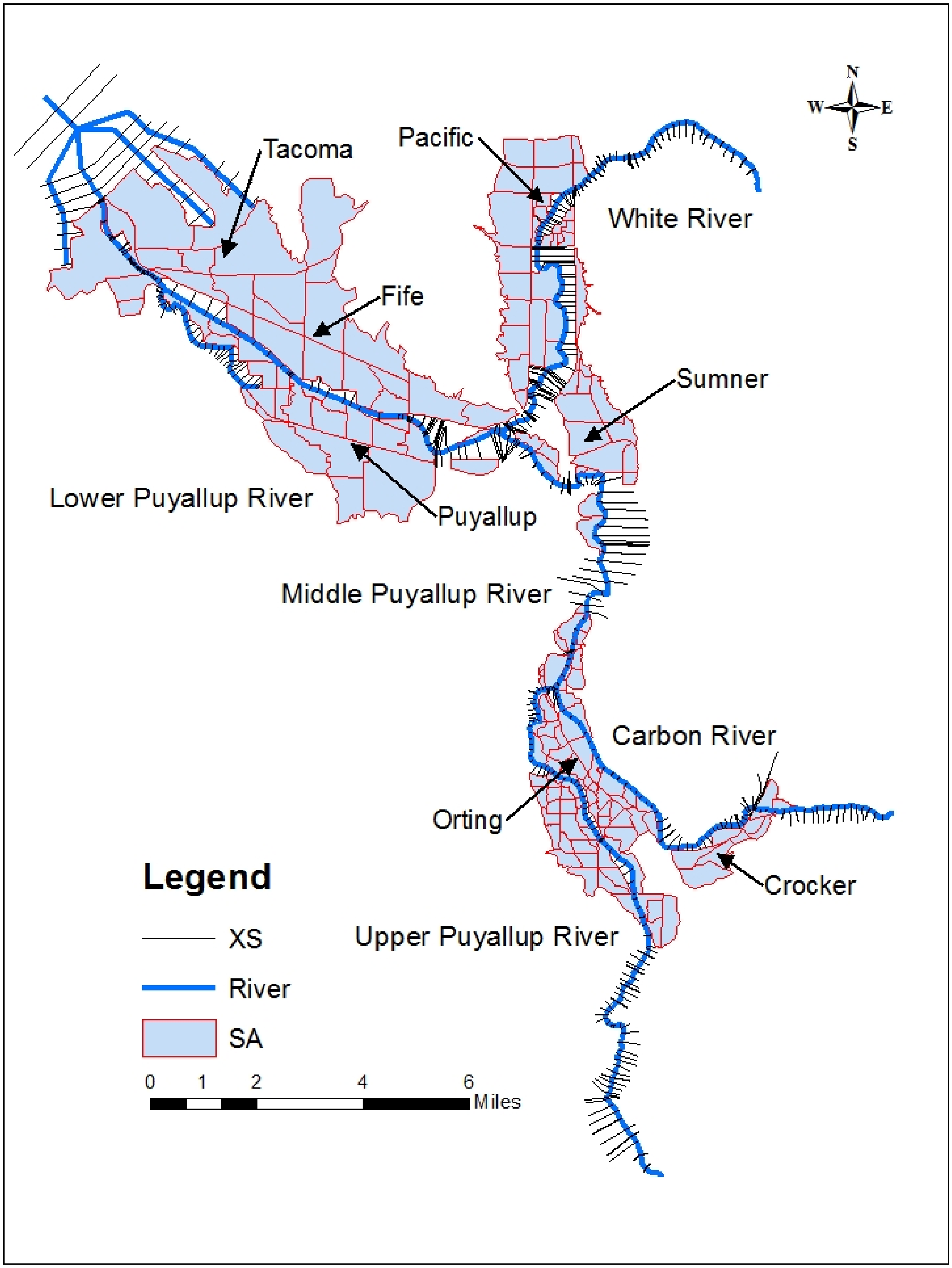


Figure 1: HEC-RAS model layout (XS= cross section, SA=storage area)

3.1.2 Cross Sections

Cross sections of the RAS model are comprised of surveyed channel, interpolated channel, and LiDAR overbanks cut from a 6x6 ft. resolution DEM. Survey channel data from the USGS (USGS, 2010) was used where available and additional channel cross sections were interpolated in RAS. Bridges and bounding cross sections (labeled NHC#A in the geometry), which had originated in the 2007 FEMA study, were found to be in the NGVD29 datum. These were converted to NAVD88 for the upper, middle, and lower reaches of the Puyallup River. In both cases, overbank terrain was created from the DEM and merged with the channel sections in RAS. This gave the best available cross section profiles from the data sources given previously in Table 1. LiDAR and USGS data were merged in GIS, while interpolated cross sections were merged with overbanks in RAS. Cross sections were located to capture changes in channel characteristics, overbank terrain, and other features that can significantly influence flow characteristics.

3.1.3 Roughness

Roughness values for overbank areas of the RAS model were determined by layering land use polygons with the cross sections in GIS and processing each cross section into segments of values. Land use polygons from the USDA Land Use Database were used and their suggested values were compared to accepted values (Chow, 1959). Where values did not agree well, the value from Chow was used. Roughness profiles were then imported into the HEC-RAS model for all cross sections. Mid-channel bars that were found to persist in the active channel over the intended calibration period (2006-2009) were located from Orthographic photos and assigned appropriate values directly in the RAS model. Land uses and associated roughness values are given in the Table 2 below. The river channels were given generalized values where open water occurred in the land use database. The steeper upper sections of the Carbon, Upper Puyallup, and White were given 0.055, while the Middle Puyallup and Lower White sections were given 0.05, and the Lower Puyallup 0.04. Values occurring in the main rivers were further calibrated, as discussed in section 5 below.

Table 2: Roughness Values and Land Uses

Land use/ cover type	Manning's "n" value
Open Water	0.04-0.055
Sand bar	0.040
Perennial Ice/Snow	0.000
Developed, Open Space	0.030
Developed, Low Intensity	0.045
Developed, Medium Intensity	0.040
Developed, High Intensity	0.035
Barren Land	0.010

Unconsolidated Shore	0.015
Deciduous Forest	0.080
Evergreen Forest	0.100
Mixed Forest	0.090
Dwarf Shrub	0.060
Scrub/Shrub	0.070
Grasslands/Herbaceous	0.035
Sedge Herbaceous	0.045
Lichens/moss	0.025
Pasture/Hay	0.035
Cultivated Crops	0.038
Woody Wetlands	0.110
Forested Wetland	0.100
Scrub/Shrub	0.070
Emergent/ Herbaceous Wetlands	0.100
Aquatic Bed	0.060

3.1.4 Junctions

The confluence between the Carbon and the Upper Puyallup Rivers is located on a steeper area of the basin just north of Orting and was best modeled by setting the computations to calculate energy losses through the junction. The confluence between the White and Middle Puyallup Rivers is considerably flatter and little if any water surface slope was predicted by the model, so the computations were set to force equal water surface to speed up the model runs. Modeling of the two junctions is somewhat complicated at very high flows, when water can take alternate paths through the floodplain. These are explained in further detail with the figures below. Cross sections on bounding sides of each junction were snapped together and ineffective flow used along the common edges to ensure wetted perimeter was calculated correctly.

The Lower Puyallup/White junction is shown in Figure 2 below. Overbank flooding at very high flows (i.e. 0.2% and less frequent events) is capable of moving into the Salmon Springs area to the East of Sumner under a railroad bridge off of the Lower White River and a highway 410 underpass off the Middle Puyallup River. Although these are not likely to be overtopped, a flowpath exists through each of them and is depicted by arrows in the figure. The portion of Sumner not covered by storage areas is located on high ground. Model flows less than a 0.2% AEP event were not seen to cause flooding of this area.

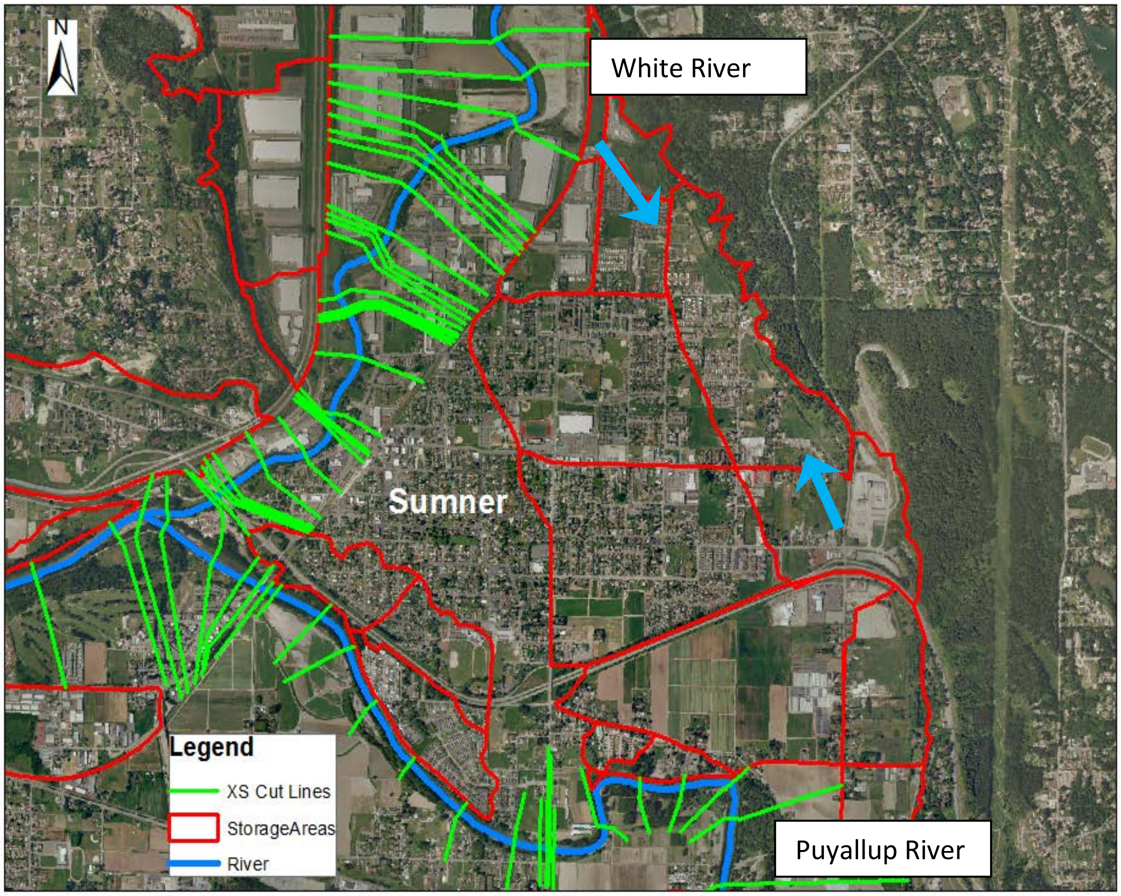


Figure 2: Junction at Puyallup/White Rivers

The Upper Puyallup/Carbon River junction is adjacent to a low agricultural area to the East shown in Figure 3 below. Flows larger than those of a 1% AEP event (depicted by an arrow) can cross through the floodplain here. The Riddell levee extends along the Carbon River up to the junction on the left bank. The depression located behind the levee is not appropriate for a storage area, but the levee was used as a lateral structure and flow over it sent to the right overbank of the long Puyallup River cross sections located in the depression. The right overbank area of these cross sections have been set to ineffective flow. This layout should provide an accurate portrayal of the junction.

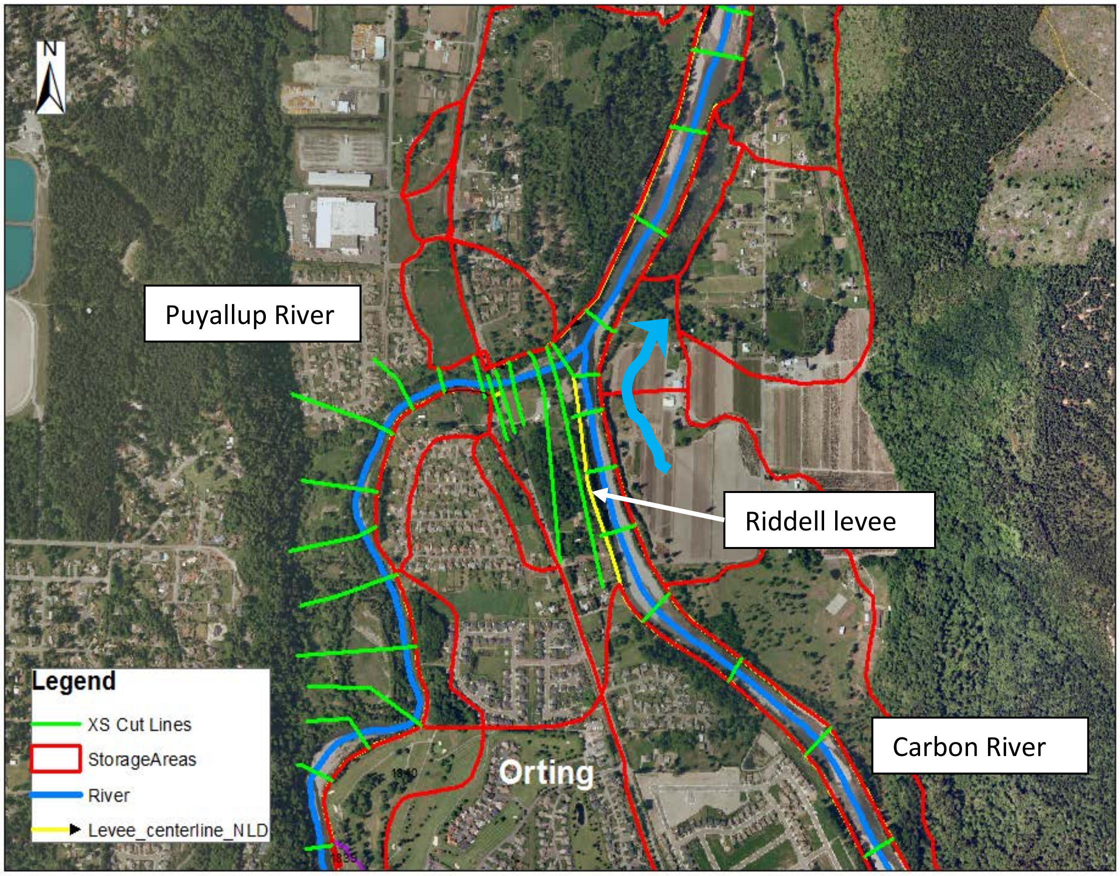


Figure 3: Junction at Upper Puyallup/Carbon Rivers

South Prairie Creek, at its confluence with the Carbon River, can be backwatered for some distance by the Carbon. A storage area has been placed adjacent to Highway 162 near the junction to represent Water Ski Lake. Initially, South Prairie Creek was incorporated into the model in three different ways to evaluate local effects on the Carbon and timing in the broader system: As a separate reach, as a storage area, and as a lateral inflow to the Carbon. Very little difference if any in downstream modeling results was seen between these approaches. No measures are expected to be evaluated on South Prairie Creek itself, so the approach that ran the fastest and with the least model stability issues was used (as a lateral inflow). Long cross sections were extended up South Prairie Creek. This is shown in Figure 4.

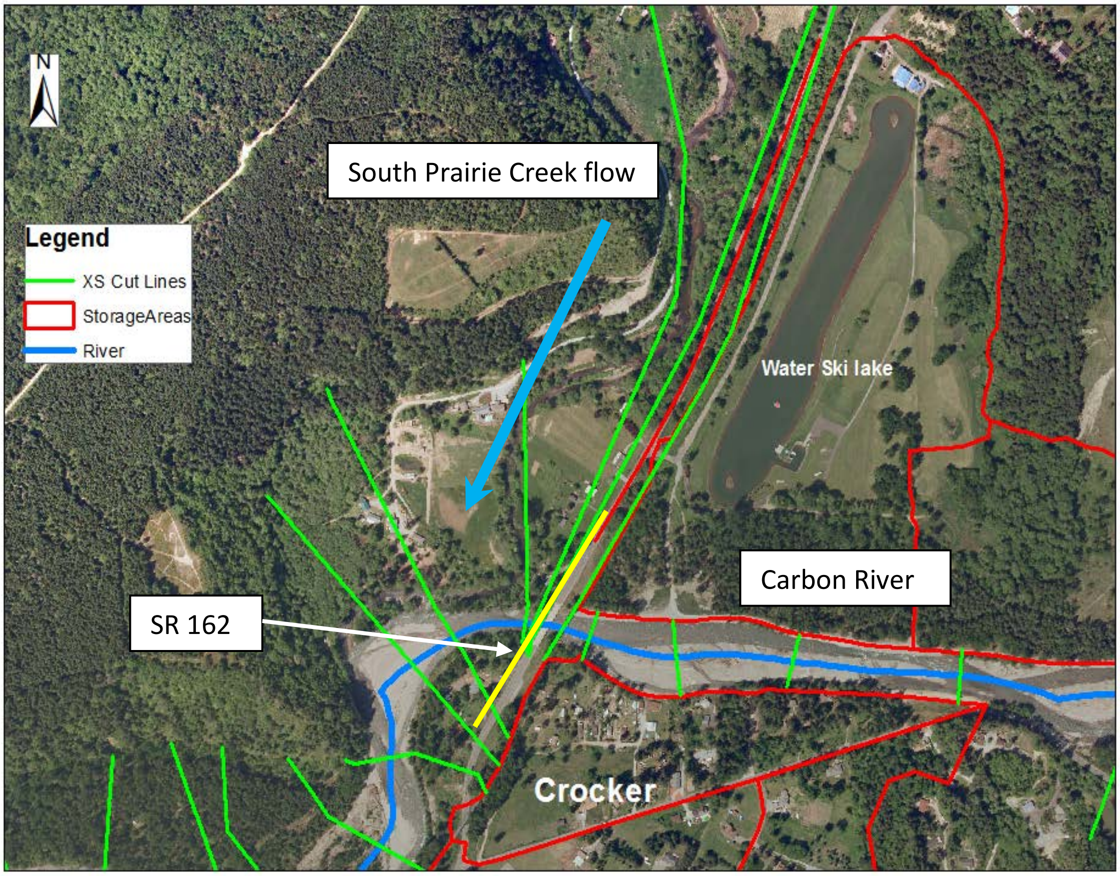


Figure 4: Junction at South Prairie Creek and Carbon River

3.1.5 Bridges and Culverts

A total of 34 bridges throughout the Puyallup River Basin were incorporated into the RAS model. Profiles and survey data were provided by the USGS for all bridges as a HEC-RAS model geometry with the surveyed cross sections. Bridges on the Puyallup River (all 3 reaches) were in the NGVD 29 datum and had to be converted to NAVD 88. Of the bridge locations, 6 were double bridges (railroad and vehicle or pedestrian and vehicle) too close together to need to model independently at high flows, so they were combined into single bridges. To combine them, the bridge deck which created the largest flow obstruction was used and given the total width of both bridges. This gave 27 bridges in the model which are given in the Table 3 below with their associated river reach and station. The Hwy 509 Bridge on the Lower Puyallup replaced the BNSF Railroad truss at that location, however survey data was not available so the BNSF Railroad truss was left in the model. This may be a later refinement to replace. At the flows used for calibration, none of the bridge decks were impacted so the flow is classified as low flow and is subcritical through all bridges. The Energy modeling approach is appropriate for these. Contraction and expansion coefficients were not a concern in the unsteady flow modeling approach for cross sections located at bridges in this study (the affect is captured in the momentum computations in HEC-RAS). At high flows when the water surface can contact the bridge deck, pressure and/or weir flow would be selected. Several bridges were impacted by the predicted 0.2% AEP water surface: Clark St, UP Railroad truss at RM 2.53, TRMD Railroad truss at Rm 2.23, and the BNSF Railroad truss at RM 1.98. Ineffective flow areas and cross sections around each bridge were configured for expansion and contraction distances as recommended in the HEC-RAS Hydraulic Reference Manual (Brunner, 2012). These distances are a function of how far bridge abutments project into the river at high flows, the bed slope, and a coefficient provided in the manual.

Table 3: Bridges within the Study Area

Bridge	Reach	RAS Station
SR 162/ foot bridge*	Carbon	31038
R St.	White	40151.53
A St./railroad*	White	33532.36
8th St.	White	26378.46
Tacoma Ave.	White	9340.251
142nd Ave.	White	7523.514
Railroad truss	White	5990.067
Main St.	White	3871.056
SR 410	White	1450.748
Orville Rd.	Upper Puyallup	136343
Calistoga Rd.	Upper Puyallup	113843.3
SR 162/railroad*	Upper Puyallup	93781
128th St./ foot bridge*	Middle Puyallup	88993
96th St.	Middle Puyallup	75848.71
SR 162	Middle Puyallup	63738.26
Main St./ Railroad truss*	Middle Puyallup	56718.82
SR 512/ Railroad truss*	Lower Puyallup	47954
Milwaukee St.	Lower Puyallup	45468.62
SR 167/ Meridian St.*	Lower Puyallup	43175.43
Clark St.	Lower Puyallup	30395.52
UP Railroad truss	Lower Puyallup	13616.56
Interstate 5	Lower Puyallup	12699.13
TRMD Railroad truss	Lower Puyallup	12027.2
Ells St. (Hwy 99)	Lower Puyallup	11206.3
BNSF Railroad truss[1]	Lower Puyallup	9847.444
Lincoln Ave./ foot bridge[2]	Lower Puyallup	7902.367

11th St.	Lower Puyallup	3863.223

* Combined bridges into a single structure
[1]Now Hwy 509 bridge (needs to be modified)
[2]Foot bridge not included

Throughout the floodplain, culverts are dispersed among road embankments and railroad grades. Where these features occurred through storage area connections they were incorporated into the model to pass flow between storage areas. A database of culverts was provided by Pierce County as a GIS shapefile containing attributes of shape, span, rise, length, elevation, and other characteristics. This shapefile was intersected with the storage area connection feature class (with a 100 ft. buffer to pick up those that didn't intersect exactly). The resulting shapefile was inspected and the attributes exported to Excel and organized by storage area connection number. The resulting 41 culverts were then manually entered into HEC-RAS.

3.1.6 Lateral Structures and Storage Areas

Levees from the National levee Database (NLD) were developed into lateral structures in the RAS model. Lateral structures allow flow to leave the main channel at high stages and enter storage areas (and vise-versa). NLD levees that spanned several storage areas were cut into segments using GIS so each did not span more than one storage area. The profile of each segment was then developed into a lateral Structure feature class in GIS for export to the RAS model. Levees are labeled in the model under the "Description" field for applicable lateral structures.

Levee setbacks at Potelco (County Line), Old soldiers Home, and Calistoga were incorporated. County Line and Calistoga setback levees have not been completed at this point, so profile data supplied by Pierce County was used to develop lateral structures. Calistoga is scheduled for completion summer of 2016 and County line for summer of 2017. At a few locations where the NLD levee Shapefile was found to contain flawed survey data (at Guy West, High Cedars, Bower/Parker, Bridge St, and Water Ski) the DEM was used to correct the levee profile.

Several NLD levees on the steepest reaches are not included in the model as lateral structures because no feature requiring it (i.e. storage area) was used. These floodplains are too steep to function as storage areas. These levees include: Ford (upstream half), Needham Rd, and Alward Segment 2. The levee tool in RAS was used at each cross section to keep flow in the channel. When the levees are overtopped in the model, overbank flow is conveyed behind the levee at the same water surface elevation as the main channel. This is appropriate considering that these areas are small and would be well connected with the main channel if a major levee failure were to occur. If these areas are deemed significant in future modeling of alternatives, such as for providing overbank storage, then additional storage areas can be added to improve them. NLD levees included in the model and their corresponding lateral structure stations are given in the

Table 4 below. The widespread use of storage areas in the basin necessitated additional lateral structures beyond the NLD levees. Where appropriate for storage area placement, lateral structures were created from the DEM and imported into the RAS model. Weir coefficients used for storage area connections were set at a value of 0.5, and for lateral structures the default value of 2. No floodplain high water marks exist for calibration of these coefficients, however values were found to be reasonable in similar studies (Brunner, 2012).

Table 4: NLD Levees

NLD Levee	Reach	RAS station
Water ski levee	Carbon	35517.38
Water ski levee	Carbon	32917.84
Alward segment 2	Carbon	33313.3
Guy west (DEM)	Carbon	29211.88
Bridge St.	Carbon	19993.74
Orting treatment plant	Carbon	16383.96
Orting treatment plant	Carbon	13957.1
Orting treatment plant	Carbon	12439.22
Riddell	Carbon	9678.314
Riddell	Carbon	7201.229
Riddell	Carbon	5995.899
Riddell	Carbon	2214.949
Lindsay	Carbon	6658.07
Lindsay	Carbon	2430.869
Ford	Upper Puyallup	126082.1
Ford	Upper Puyallup	123628.9
Jones	Upper Puyallup	120405.9
Jones	Upper Puyallup	116102.1
Calistoga	Upper Puyallup	108758.08
Calistoga	Upper Puyallup	114089
Calistoga	Upper Puyallup	113751.6
Calistoga	Upper Puyallup	112254.4
Calistoga	Upper Puyallup	109998
Old soldiers Home (DEM)	Upper Puyallup	120627.6
Leach Rd.	Upper Puyallup	113653.73
Leach Rd.	Upper Puyallup	109820.1
Leach Rd.	Upper Puyallup	105075.9
High Cedars (DEM)	Upper Puyallup	105593.6
High Cedars (DEM)	Upper Puyallup	103107.3
High Cedars (DEM)	Upper Puyallup	97863.06
High Cedars (DEM)	Upper Puyallup	96469.08
Bower/Parker	Upper Puyallup	93830
Lindsay	Middle Puyallup	92492.19
Bower/Parker	Middle Puyallup	92481.04
McMillin	Middle Puyallup	88915
Sportsman	Middle Puyallup	77113.79

Sportsman	Middle Puyallup	75764.61
Bowman/Hilton	Middle Puyallup	72368.56
Riverside	Middle Puyallup	67815.93
River Grove	Middle Puyallup	60700.5
County Line	White	33334
County Line	White	29951.03
County Line	White	28180
Old Cannery	Lower Puyallup	54088.08
North Levee Rd.	Lower Puyallup	42806.1
North Levee Rd.	Lower Puyallup	39026.05
North Levee Rd.	Lower Puyallup	36149.11
North Levee Rd.	Lower Puyallup	31771.94
North Levee Rd.	Lower Puyallup	30272.63
North Levee Rd.	Lower Puyallup	25302.5
North Levee Rd.	Lower Puyallup	22291.17
Puyallup Right Bank	Lower Puyallup	16196.75
Puyallup Right Bank	Lower Puyallup	13894.84
River Rd.	Lower Puyallup	40795.13
River Rd.	Lower Puyallup	38315.94
River Rd.	Lower Puyallup	36776.76
River Rd.	Lower Puyallup	34556.6
River Rd.	Lower Puyallup	31934.43
River Rd.	Lower Puyallup	30303.49
River Rd.	Lower Puyallup	28773.35
River Rd.	Lower Puyallup	24010.35
River Rd.	Lower Puyallup	20701.42
River Rd.	Lower Puyallup	17267.37
Puyallup Left Bank	Lower Puyallup	15215.65
Puyallup Left Bank	Lower Puyallup	13508.54
Puyallup Left Bank	Lower Puyallup	12500

Each lateral structure segment in the model is associated with a separate storage area. As previously described in the hydraulic analysis methodology, this was done to provide a way for flow to move laterally away from the main channel at a different water surface. This will also provide a better way to track inundation across the terrain than extending cross sections out away from the channel. Storage areas are numbered with river reach designations as: LL=lower left, LR=lower right, ML=middle left, MR=middle right, UL=upper left, UR=upper right, CL=Carbon left, CR=Carbon right, WL=White left, WR=White right. Numbering is downstream to upstream for each side of each reach, with numbering from each reach following the previous one for consecutive 1-185. Storage area numbering is shown in Table 5. Inundation for each storage area is shown in the mapping for existing and future conditions.

Table 5: Storage Area Designation

Reach	designation	numbering
Lower Puyallup left	LL	1-24
Lower Puyallup right	LR	25-50
Middle Puyallup left	ML	51-56
Middle Puyallup right	MR	57-70
Upper Puyallup left	UL	71-98
Upper Puyallup right	UR	99-129
Carbon left	CL	130-147
Carbon right	CR	148-152
White left	WL	153-168
White right	WR	169-185

3.2 Initial and Boundary Conditions

An upstream hydrograph boundary was used for each reach of the model, and a downstream stage time series was used to represent the Commencement Bay boundary. Flows used for calibration of the model are discussed here; frequency flow data is discussed in the statistical Hydrology section. However, model input locations are generally the same. The time periods for use in calibration cover the last three large Atmospheric River events that impacted the Puyallup River basin: Nov 5-12, 2006, Nov 10-17, 2008, and Jan 5-13, 2009. The lower reach of the Puyallup River is tidally influenced at the downstream boundary of the RAS model. A time series of tidal stages at Tacoma was obtained from NOAA for each of the time periods and the datum adjusted to NAVD88 from the station datum (by adding the 3.07 ft. conversion). Commencement Bay was represented as a single reach for each waterway at the Port of Tacoma, for a total of five reaches. Tidal effects do not cause significant coastal flooding for up to the 0.2% AEP event (the maximum tide stage is lower than the developed port facilities). The downstream-most storage areas were connected to these reaches with lateral structures cut from the DEM along the perimeter of the coast (mostly shipping docks and industrial areas). For the upstream boundary of each river reach a time series of flows was obtained from the USGS for the nearest streamgage and translated for travel time to the model area. Due to the very steep upper reaches of the model and proximity to nearest streamgages, routing was not necessary and test hydrographs did not show significant attenuation. Additionally, several USGS streamgages are located in the interior of the model. Streamgage data and sources are summarized in Table 6 below and all hydrograph data is 1 hr. time interval. The model was found to run stably with an initial flow of 3000 cfs for the upper reaches, summed to 6000 cfs for the Middle Puyallup and 9000 cfs for the Lower Puyallup. This is a little higher than what would typically be seen as baseflow in these rivers. Minimum flows were set at 1000 cfs for all reaches.

Table 6: Streamgage Data and Location

USGS Streamgage	RM	Location	Reach	RAS Cross Section
12093500	25.2	Puyallup River near Orting[1]	Upper Puyallup	3.3 miles US of 135681.9
12096500	12	Puyallup river at Alderton	Middle Puyallup	63717.53
12101500	6.5	Puyallup River at Puyallup	Lower Puyallup	34618.98
12100496	6.2	White River near Auburn[1]	White	4 miles DS of 55590.453
12094000		Carbon River near Fairfax[1]	Carbon	7 miles US of 44337.578
12095000		South Prairie Creek at South Prairie	South Prairie	30990.38
12102190		Swan Ck. (Tacoma)	Lower Puyallup	14519.32
12102075		Clarks Ck. (Tacoma)	Lower Puyallup	30631.16
9446484		(NOAA) Tacoma[1]	Lower Puyallup	457.05

[1]External boundary condition

Flows from major gauged tributaries (South Prairie Creek, Kapowsin Creek, Clear Creek/Swan Creek, and Clarks Creek) and ungaged tributaries (Voights creek, Fiske Creek, and Fox Creek) were included as point lateral inflows (NHC, 2012). Ungaged local inflows were included as uniform lateral inflows (NHC, 2012). Data sources for all model inflows are summarized in Table 7 below. Ungaged model inflows were determined from gauged reference basins based upon several criteria (NHC, 2012): Streamgage proximity, drainage area, topography, rainfall regime, soil type, and land use. Flows from suitable reference basins were scaled to ungaged tributaries in the model on a flow-per-square-mile basis. Local ungaged inflows that could not be attributed to specific tributaries were added to the model as uniform lateral inflows over each model reach. Local inflows over the entire basin have a contribution of around 5-10,000 cfs for recent flood events (typically less than 25% of total flow at the Puyallup streamgage). Model flow inputs for frequency runs will differ in that they are determined from many events throughout the record to give the best possible estimation of what a given frequency event will actually be. This is further discussed in the hydrology section.

Table 7: Local Model Inflows

Model Inflow	Data Source	Area (sq mi)	RAS XS/ SA
Swan Ck.	Swan Ck.	3.5	SA 2280
Clear Ck.	Swan Ck.	3.1	SA 2003
Clarks Ck.	Clarks Ck.	13.0	SA 1068
Wapato Ck.	Clarks Ck.	6.0	SA 1062
Hylebos Ck.	Clarks Ck.	4.7	SA 1026
ungaged local	Clarks Ck.	17.8	53575-34618
ungaged local	Clarks Ck.	9.0	34618-1408
Fennel Ck./ local	Newaukum Creek	27.0	85543.27

Orting local	South Prairie Ck.	16.0	135271.7-93153.99
Fiske Ck./ Fox Ck./ upper local	South Prairie Ck.	12.4	149828.2-137650.4
Kapowsin Ck.	Machel River	29.6	141117.20
local above SP	South Prairie Ck.	20.7	42764.22-35164.91
Voight Ck.	South Prairie Ck.	32.6	SA 1132
Orting local below SP	South Prairie Ck.	17.0	29739.44-3530.09
South Prairie Ck. (SP)	South Prairie Ck.	79.5	30904.60
Lower ungaged local	Newaukum Creek	16	34618-32847

4.0 Hydraulic Model Calibration and Validation

4.1 Data Sources

The objective of calibration was to adjust parameters in the hydraulic model such that the water surface output represents observed conditions with minimal error. This meant adjusting sources of energy loss such as roughness and other coefficients. There is a significant amount of uncertainty in estimation of local inflows between the streamgages in the system for high flow events. The location, timing, and magnitude if each local inflow source is not very well defined between streamgages. The approach used in development of the hydrologic data, discussed in the hydrology section, was used to synthesize local inflows for the recent flood events used for calibration and validation. Because of the significant variability in timing throughout the system, possible channel storage, and the need for variation in roughness with flow, significant time was not spent trying to calibrate the model with steady flows. Unsteady flows were run to capture these sources of uncertainty better and to attempt to match rating curves at streamgages. Information on flows and stages was collected for the November 2006, November 2008, and January 2009 flood events from the USGS. Several observed high water marks were used for these events. High water marks are very limited throughout the basin with some on the White River and the Lower Puyallup River. The 2006 (10% AEP) and 2009 (5% AEP) events were used for calibration, and the 2008 (20% AEP) event used as a validation check on the model. Section 5.8 discusses development of the downstream boundary condition.

4.2 HEC-RAS Model Calibration

A combination of high water marks, streamgage records, and rating curves were used to calibrate the model. The ultimate goal was matching the observed streamgage data at the Lower Puyallup. Available streamgage data differed slightly between events due to establishment of new streamgages on the White and Middle Puyallup Rivers. Streamgage data used for calibration consisted of published rating curves and stage/flow records at Puyallup and Orting streamgages, and stage records at the Alderton and White River (A St.) streamgages. The rating at the Alderton streamgage is known to be unreliable for high flows over the 2006-2009 time period so was not used, but the stage data was still useful for calibration. The rating at the White River A St streamgage is known to have changed so drastically between 2006 and 2009 that it was replaced with the R St Streamgage after the Jan 2009 event. The stage data at A St streamgage was useful for calibration. The rating curve at Orting is known to be inaccurate for the upper end (above 10,000 cfs). The datum and published rating were confirmed to be incorrect for the streamgage, and the correct datum and rating were obtained directly from the USGS. To match the rating curves, flow-roughness variation in Manning's "n" value was necessary for the main channel. Main channel Manning's "n" values were the primary variable adjusted to calibrate the model. Roughness was varied, within reasonable limits, to essentially adjust the produced model rating curve to match the streamgage rating curve. Necessary factors varied by as much as +60% and -15% from base values. Base values used were: 0.04-0.05 for the Upper Puyallup and Carbon, 0.038-0.028 for the Middle Puyallup, 0.035-0.028 for the Lower Puyallup, and 0.03-0.04 for the White. Peak stages and rating curves used are shown in Table 8. USGS streamgages are labeled in the model under the "Node Name" field for applicable cross sections.

Table 8: Calibration data (elevations in NAVD88)

USGS Streamgage	Streamgage #	RAS model XS	2006 (ft.)	2009 (ft.)	2008 (ft.)	rating curve
Puyallup at Puyallup	12101500	34618.98	32.18	33.04	30.36	all events
Puyallup at Alderton	12096500	63717.53	64.44	64.76	62.47	unreliable
Puyallup near Orting	12093500	135681.9	368.09	367.80	367.46	all events
White R. at A street	12100496	33593.38	89.28	90.19	88.49	unreliable
White R. at R street	12100490	40234.84	n/a	n/a	n/a	n/a

Floodplain values (where cross sections covered the floodplain) were not adjusted beyond what was determined from the land use database because little or no floodplain flow was observed for these events. Floodplain areas that were inundated for these events are covered with storage areas. After adjusting the model based on streamgage data and high water marks, each event was mapped to compare flood extents with Ariel photos. Minor adjustments to levees, lateral structures, and storage areas were made to better capture observed flooding for each event.

4.2.1 Calibration Results: November 2006 Flood Event

This event transpired from Nov 6-8, 2006, with high flows on the White River lasting several more days as Mud Mountain Dam was drawn down. Cross sections were surveyed in 2009 and are not reflective of the channel conditions on the White River between RM 5-6.4 (Pacific area) in 2006. Therefore, comparison to 2006 stage data at this location is not realistic. The 2009 event was used for calibration instead for the White River. Table 9 shows the computed peak stages, observed peak stages, and differences in stage and timing (computed minus observed). Calibration is discussed further in section 4.3.

Table 9: Model Calibration 2006 results

Location	Computed Peak Stage (ft. NAVD)	Observed Peak Stage (ft. NAVD)	Stage Difference (ft.)	Timing Difference (hr)
Puyallup at Puyallup	32.05	32.18	-0.13	-2
Puyallup at Alderton	64.27	64.44[1]	-0.17	-1
Puyallup near Orting	369.87	368.09[1]	1.78	0
White at A street	91.41	89.28[1]	2.13	0
Location	**Computed Peak Flow (cfs)**	**Observed Peak Flow (cfs)**	**Flow Difference (cfs)**	**Timing Difference (hr)**
Puyallup at Puyallup	44616	39700	4916	-2
Puyallup at Alderton	42282	51600[1]	-9318	-1
Puyallup near Orting	16876	17400[1]	-524	0
White at A street	13547	14700[1]	-1153	0

[1]Known rating curve issues at streamgage

Shown in the figures below are rating curve and hydrograph plots for each location.

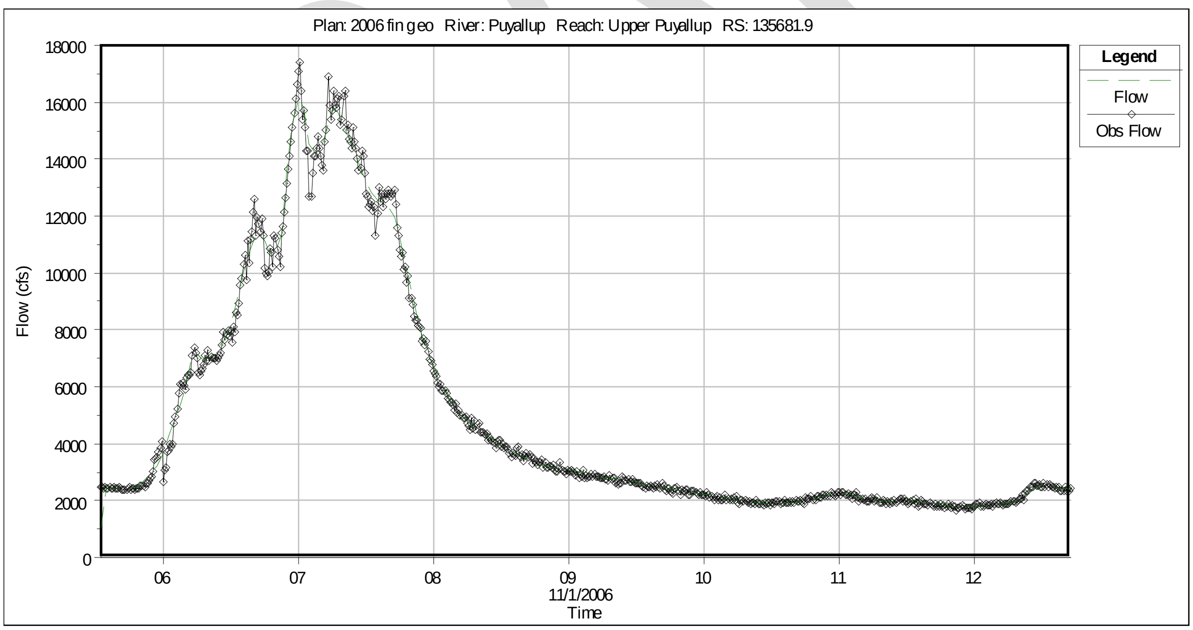


Figure 5: Computed and Observed Orting hydrograph at RS 135681.9- 2006

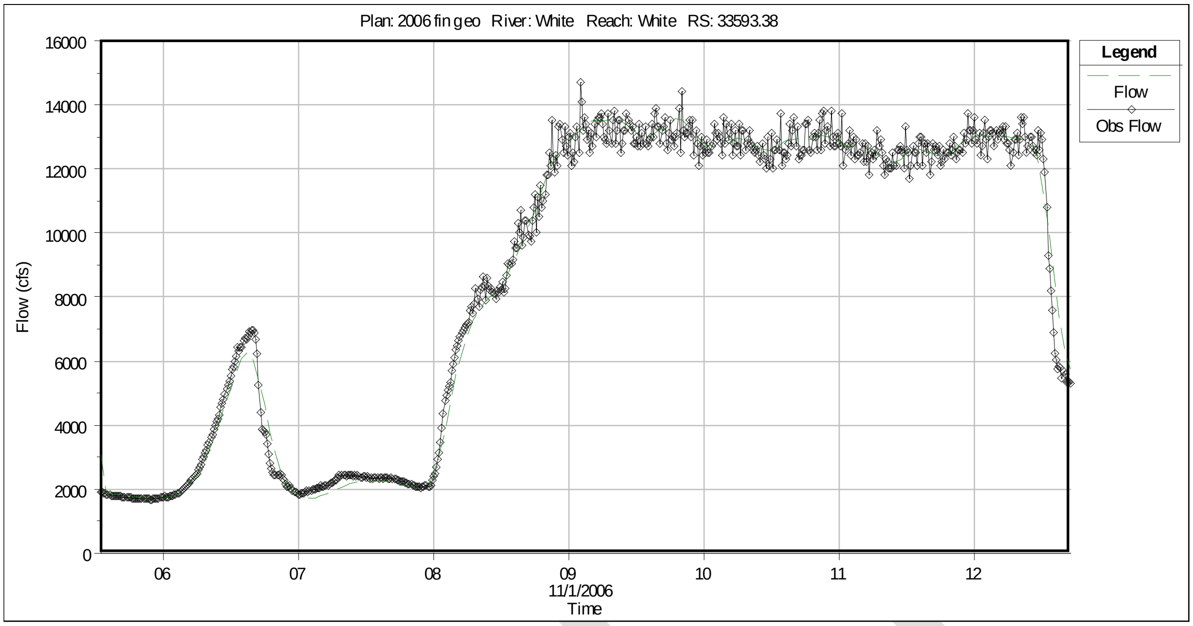


Figure 6: Computed and Observed A Street hydrograph at RS 33593.38- 2006

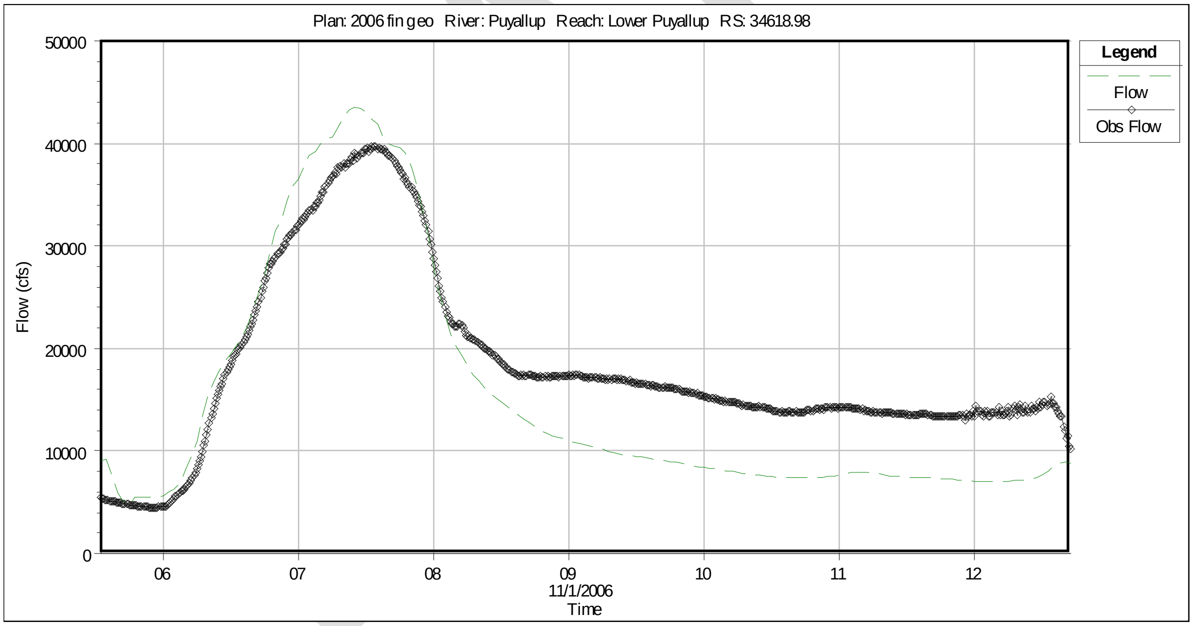


Figure 7: Computed and Observed Puyallup hydrograph at RS 34618.98- 2006

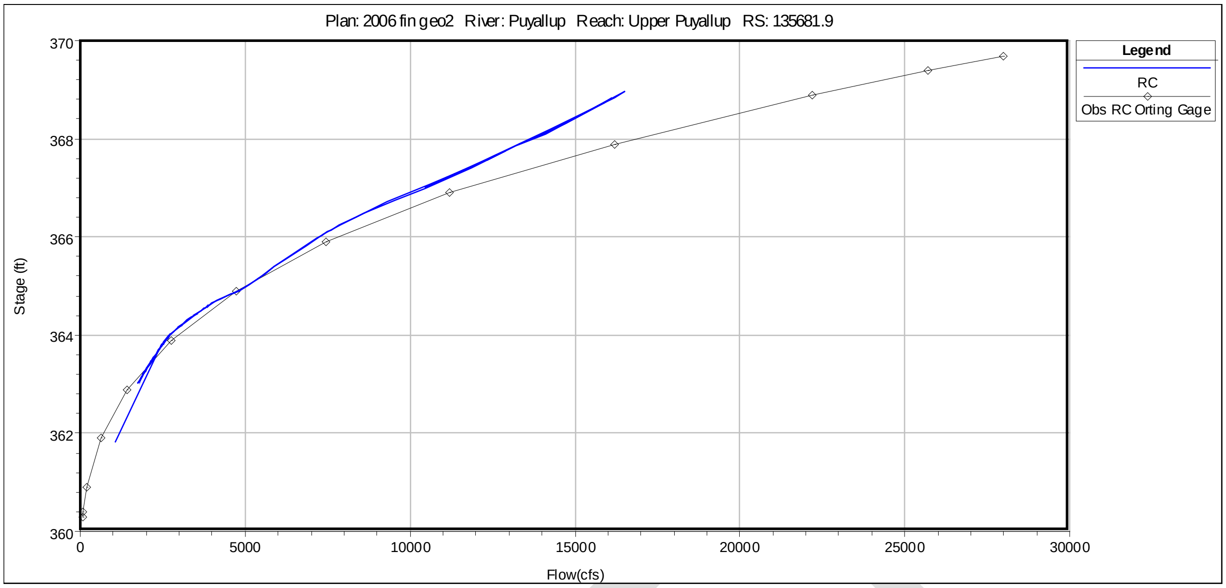


Figure 8: Orting rating curve at RS 135681.9- 2006

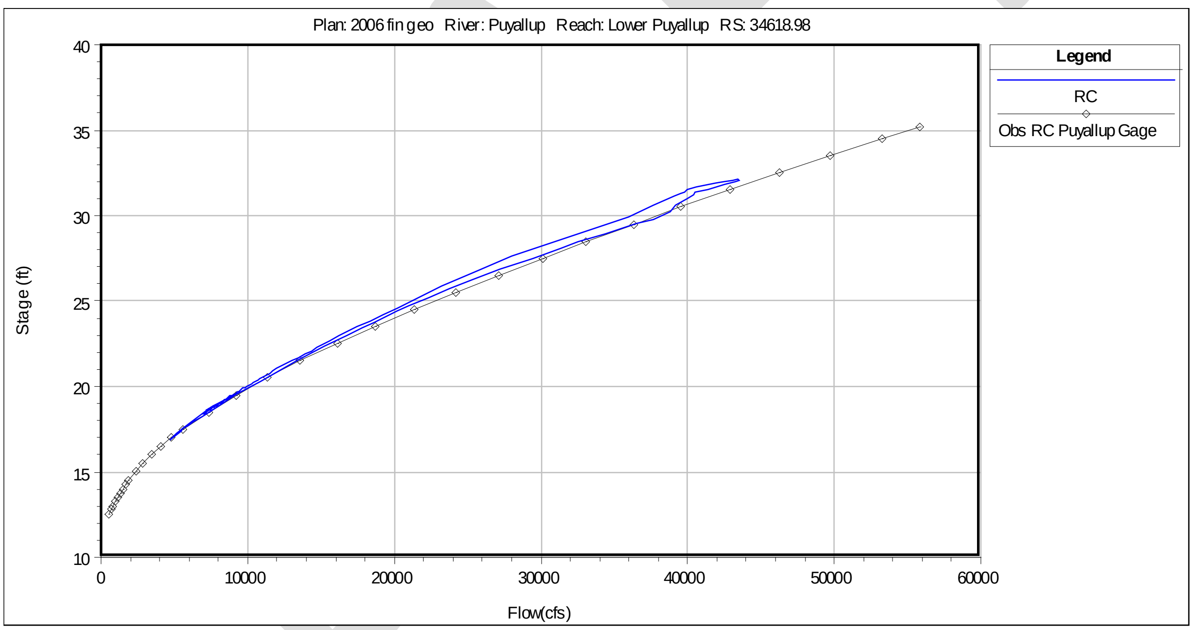


Figure 9: Puyallup rating curve at RS 34618.98- 2006

4.2.2 Calibration results: January 2009 Flood Event

This event transpired from Jan 7-9, 2009, with high flows on the White River lasting several more days as Mud Mountain dam was drawn down. Antecedent conditions for this event were substantially different than the other two events used. Jan 2009 saw high snowfall throughout the lower basin, which served to dampen local runoff. While it was possible to match the rating curve and peak stage at Puyallup, the peak flow timing was 7 hours later than computed by the model. Calibration is discussed further in section 4.3.

Table 10: Model Calibration 2009 Results

Location	Computed Peak stage (ft. NAVD)	Observed Peak stage (ft. NAVD)	Stage Difference (ft.)	Timing Difference (hr)
Puyallup at Puyallup	33.48	33.04	0.44	-6
Puyallup at Alderton	65.46	64.76[1]	0.7	-5
Puyallup near Orting	369.6	367.80[1]	1.8	-1
White at A street	90.65	90.19[1]	0.46	0
Location	**Computed Peak Flow (cfs)**	**Observed Peak Flow (cfs)**	**Flow Difference (cfs)**	**Timing Difference (hr)**
Puyallup at Puyallup	47764	48200	-436	-6
Puyallup at Alderton	44488	53600[1]	-9112	-5
Puyallup near Orting	15840	16900[1]	-1060	-1
White at A street	11720	12000[1]	-280	0

[1]Known rating curve issues at streamgage

Shown in the figures below are rating curve and hydrograph plots for each location.

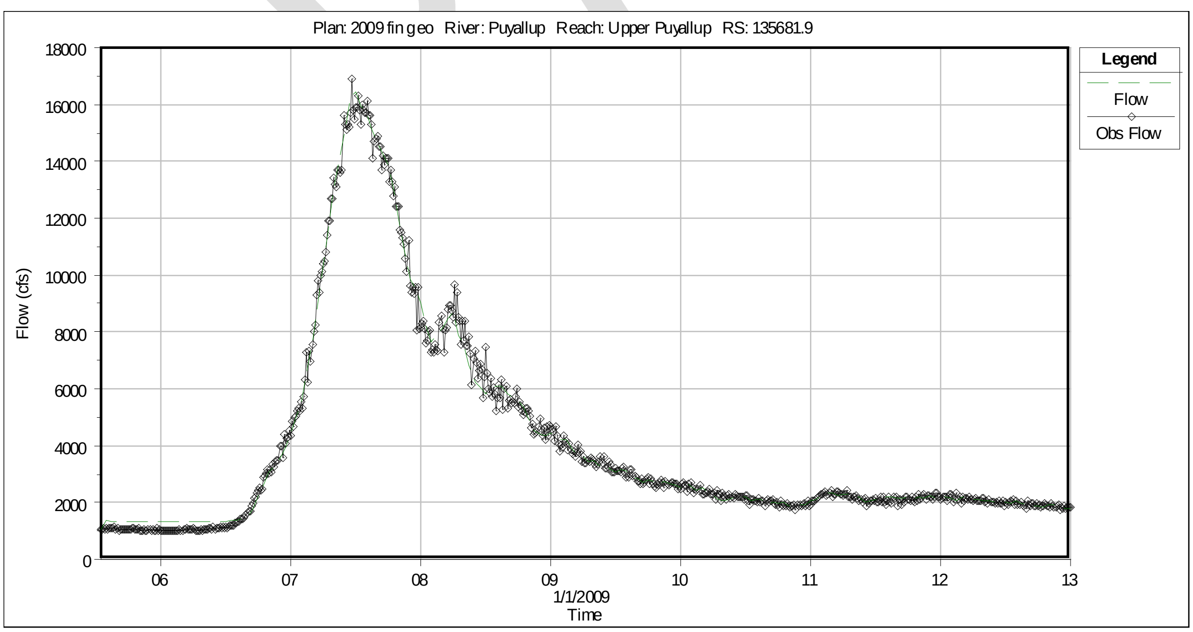


Figure 10: Computed and Observed Orting hydrograph at RS 135681.9- 2009

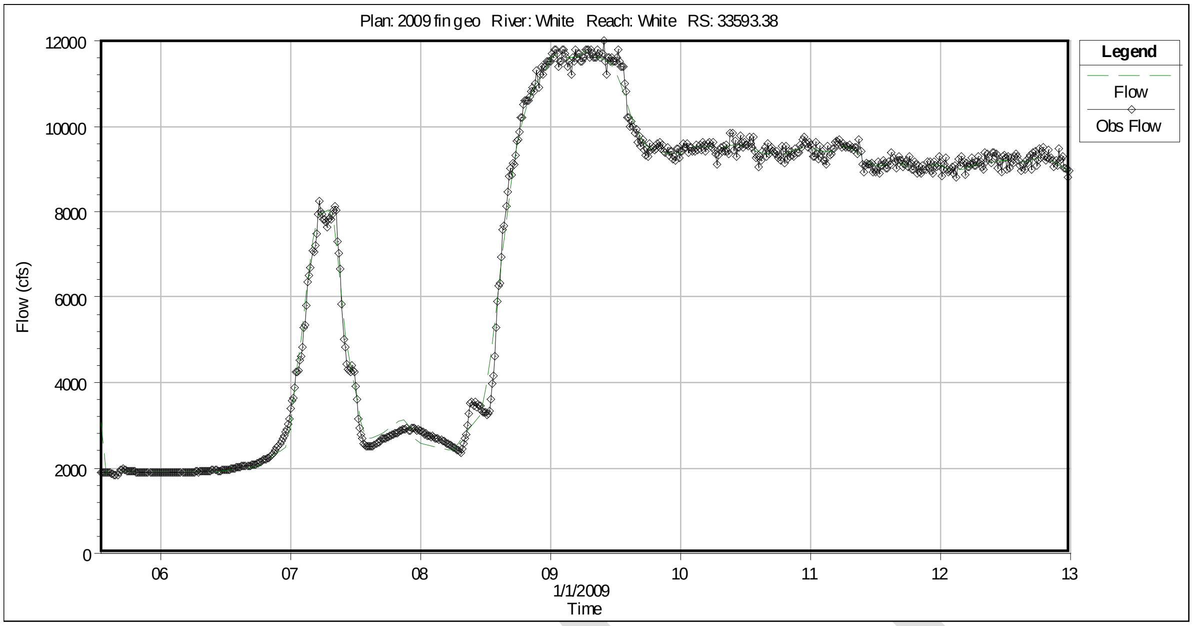


Figure 11: Computed and Observed A Street hydrograph at RS 33593.38- 2009

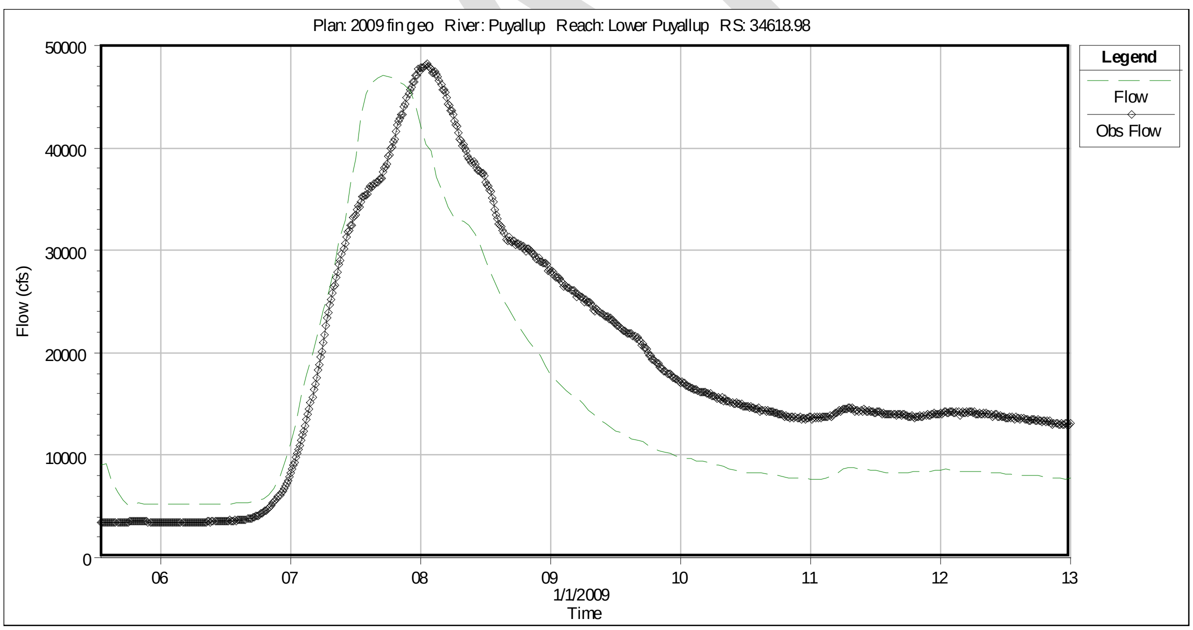


Figure 12: Computed and Observed Puyallup hydrograph at RS 34618.98- 2009

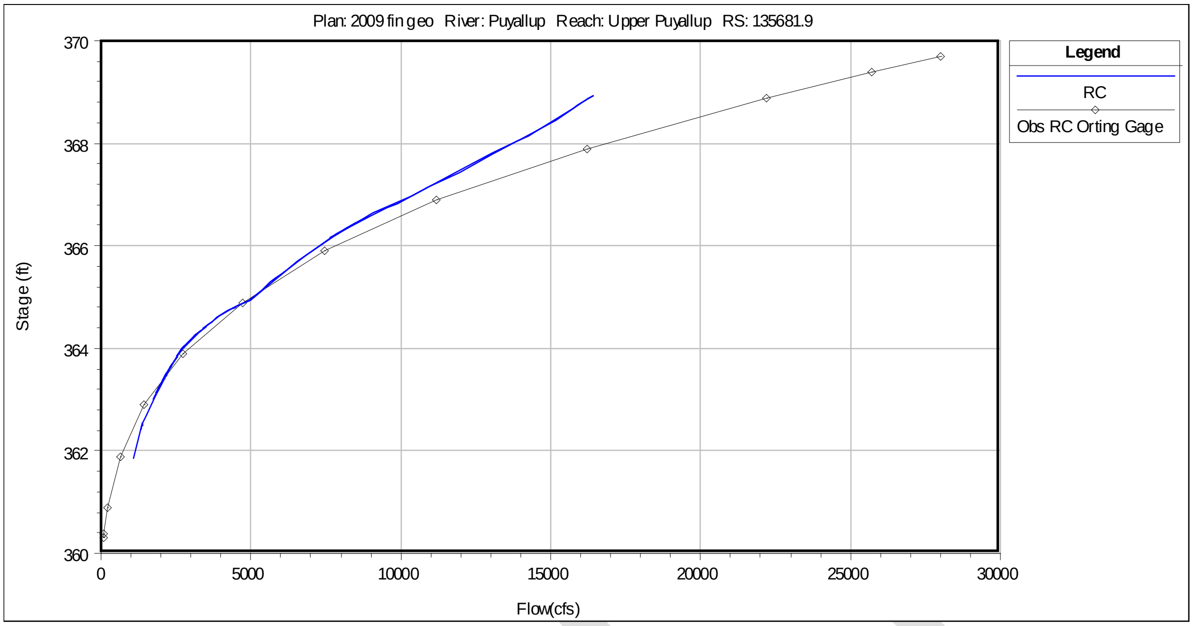


Figure 13: Orting rating curve at RS 135681.9- 2009

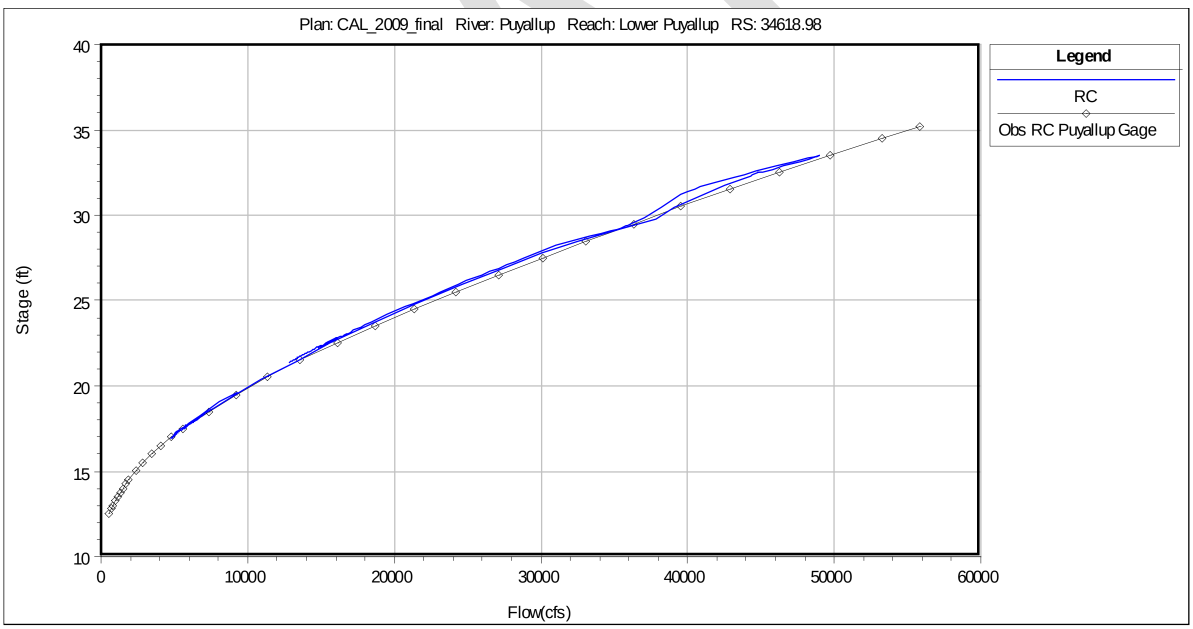


Figure 14: Puyallup rating curve at RS 34618.98- 2009

4.2.3 Validation Results: November 2008 Flood Event

This event transpired from Nov 12-13, 2008 with high flows on the White River lasting several more days as Mud Mountain dam was drawn down. Cross sections were surveyed in 2009 and are not reflective of the channel conditions on the White River between RM 5-6.4 (Pacific area) in 2008. Therefore, comparison to 2008 stage data at this location is not realistic. The 2009 event was used for calibration for the White River. Table 11 shows the computed peak stages, observed peak stages, and differences in stage and timing (computed minus observed). Validation is discussed further in section 4.3.

Table 11: Model Validation 2008 Results

Location	Computed Peak Stage (ft. NAVD)	Observed Peak Stage (ft. NAVD)	Stage Difference (ft.)	Timing Difference (hr)
Puyallup at Puyallup	29.53	30.36	-0.83	-0.5
Puyallup at Alderton	62.21	62.47[1]	-0.26	-0.25
Puyallup near Orting	368.96	367.46[1]	1.5	0
White at A street	90.46	88.49[1]	1.97	0
Location	**Computed Peak Flow (cfs)**	**Observed Peak Flow (cfs)**	**Flow Difference (cfs)**	**Timing Difference (hr)**
Puyallup at Puyallup	36952	34600	2352	-0.5
Puyallup at Alderton	34300	40200[1]	-5900	-0.25
Puyallup near Orting	14485	15200[1]	-715	0
White at A street	10550	10900[1]	-350	0

[1]Known rating curve issues at streamgage

Shown in the figures below are rating curve and hydrograph plots for each location.

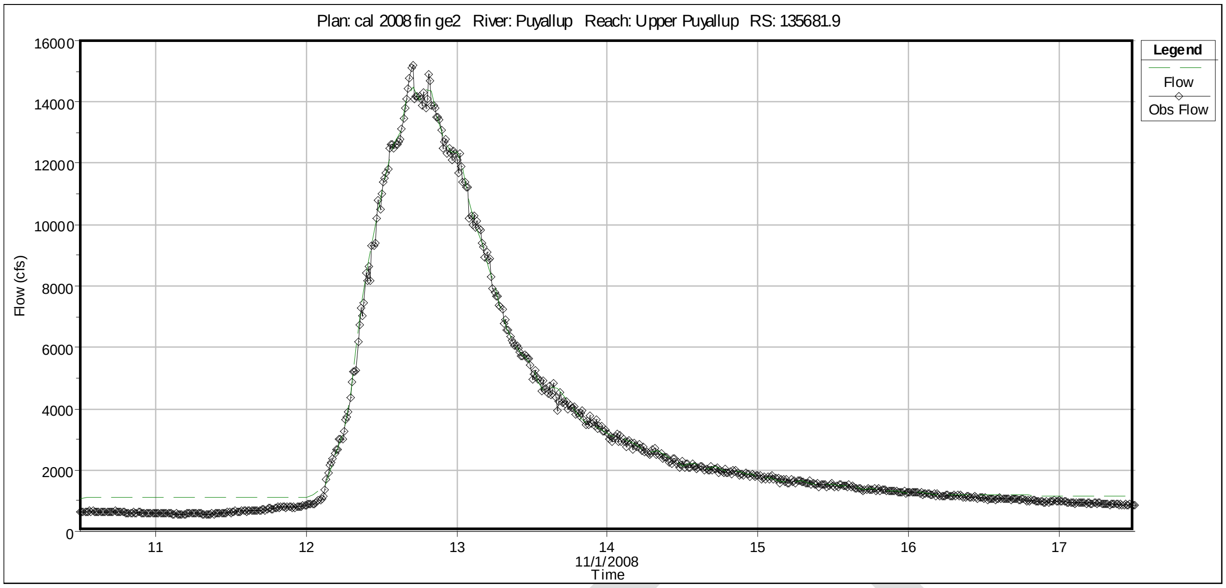


Figure 15: Computed and Observed Orting hydrograph at RS 135681.9- 2008

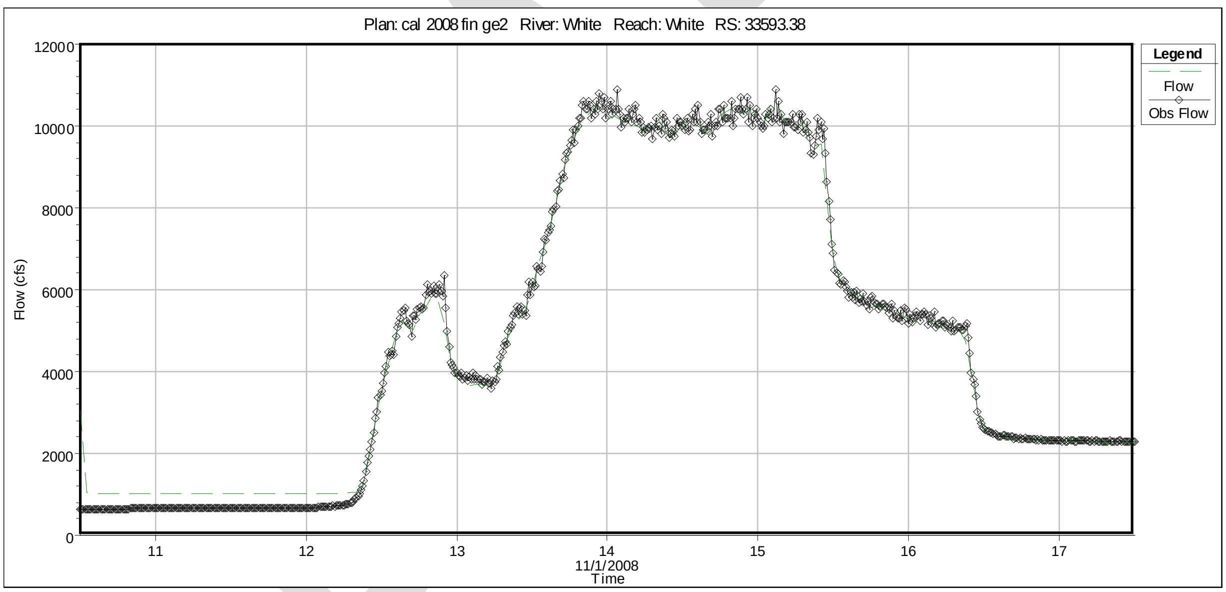


Figure 16: Computed and Observed A Street hydrograph at RS 33593.38- 2008

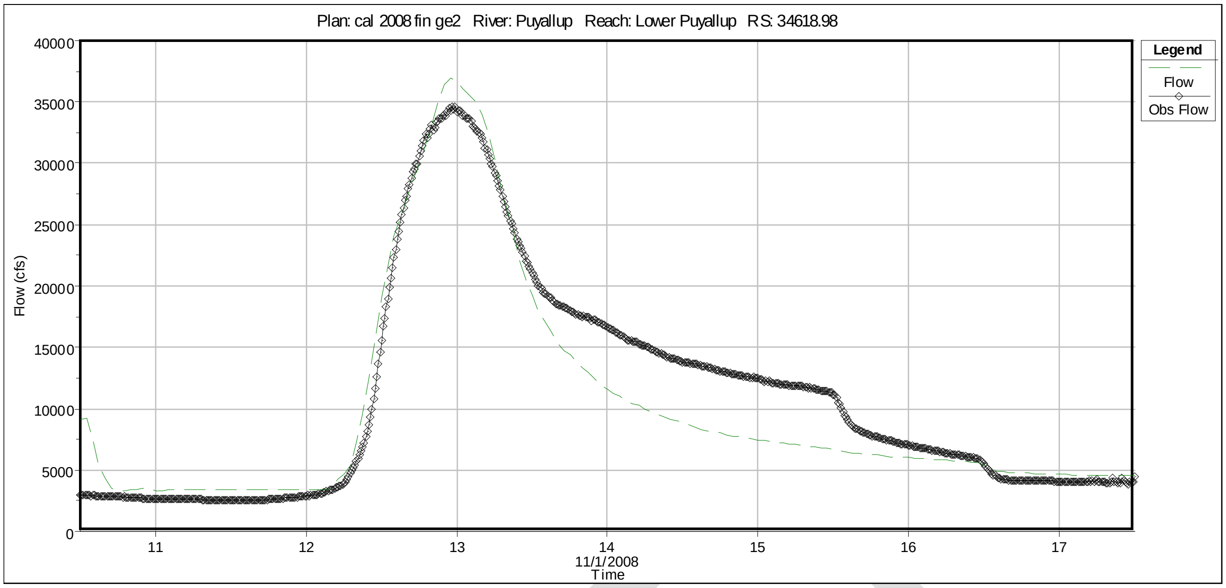


Figure 17: Computed and Observed Puyallup hydrograph at RS 34618.98- 2008

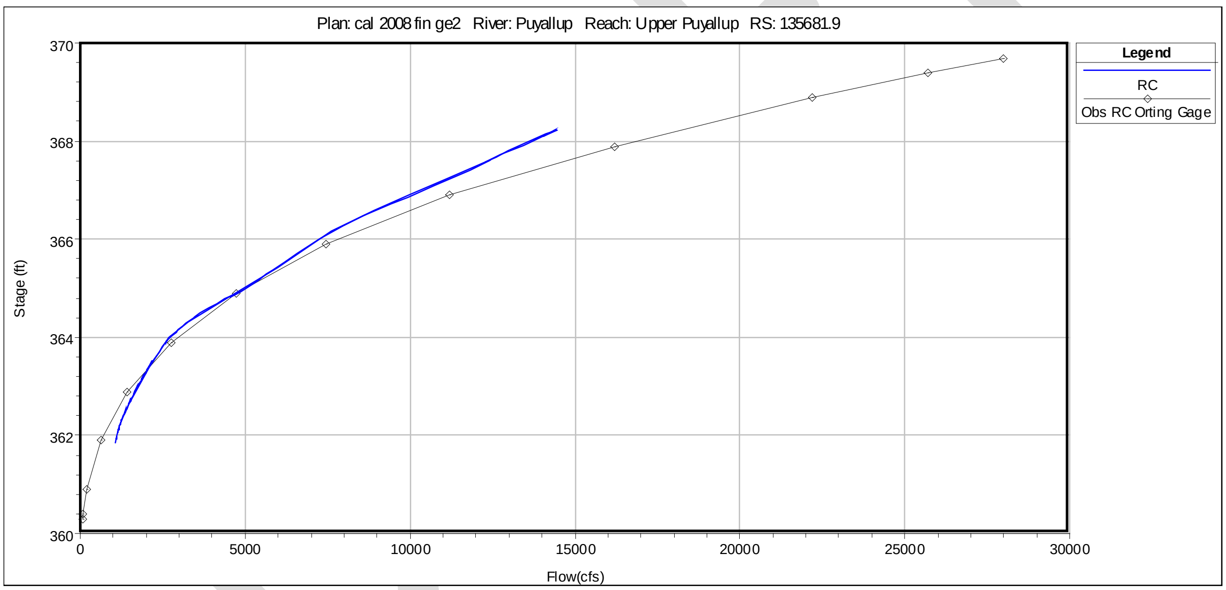


Figure 18: Orting rating curve at RS 135681.9- 2008

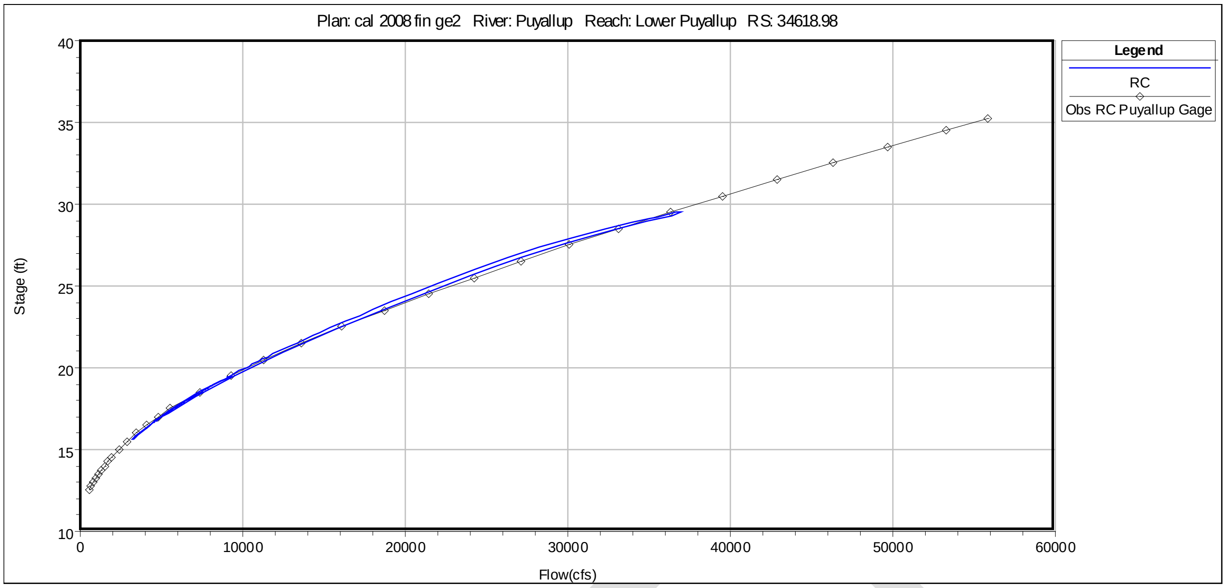


Figure 19: Puyallup rating curve at RS 34618.98- 2008

4.3 Discussion of Calibration and Validation Results

As shown in the tables and plots above, the model calibration was good at locations of high confidence where computed stages were typically within 1 foot of observed data. The Orting streamgage location was problematic and matching the observed peak stages would have resulted in unrealistic roughness values. Locations of known problems with streamgage ratings were not given unreasonable roughness values to compensate, but instead roughness values commonly given in literature were used with professional judgment. Overall the model is a good representation of water movement and flooding throughout the Lower Puyallup basin, and is considered adequate to access potential flooding due to changes in levees, dam regulation, and hydrology. The approach taken to model the floodplain (cascading storage areas) has captured the final location of pooled water, but does not predict the exact path taken through the floodplain or the timing to get there. The maximum resulting stage for each storage area will be used to estimate economic damages. Flooding not picked up in this will largely be due to either sheet flow over rural areas or concentrated flow through streets and other urban features that will convey water quickly to low areas. The intended use of the model, for FDA analysis, depends upon peak flow and stage rather than volume at the streamgages. With the large uncertainty in timing and magnitude of locals throughout the system, reproducing hydrograph volumes proved to be difficult. Significant variability exists in local inflows that proved difficult to capture. This is likely the cause of volume differences. Less problematic was reproducing peak stages. Model parameters (such as roughness) were kept to within accepted ranges rather than adjusted to unreasonable values.

The calibrated model geometry will be used at high flows (i.e. 50% and events of less frequent AEP). A 0.2% AEP event will see a significant amount of water leave the channel along each reach. Areas of uncertainty include Manning's "n" values, estimation of local ungaged inflows, errors in levee alignment and elevation profile data, errors in terrain (LiDAR) data, errors in bridge survey data, errors in flow data, and errors in cross section survey data. As with any hydraulic model there is always room for improving upon what has been accomplished so far. Each of these areas of uncertainty could be improved upon with additional survey data and streamgage data. However the value of better data comes at the expense of time and resources. The data available for this study was quite extensive and it is doubtful that significant expense could be justified to improve it when uncertainty is well accounted for in many of the probabilistic methods used in analysis of the model results. One large source of uncertainty for modeling hypothetical events that was not problematic in calibrating the model to known events, was prediction of timing of flood flows on the different reaches of the model. This is discussed further in the hydrology section. The period of record has been analyzed and the most likely coincidental occurrence of flows on each reach used in development of flow frequency estimates.

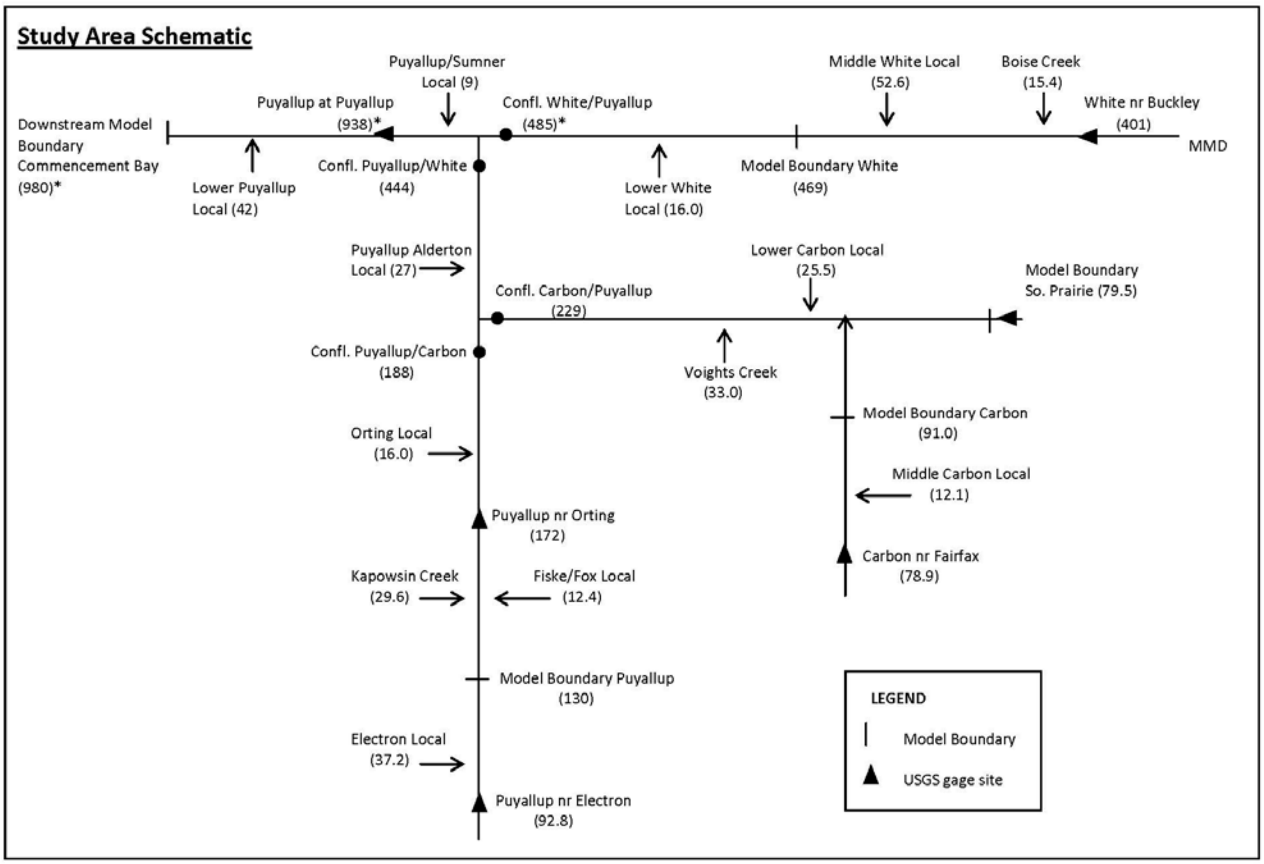
5.0 Statistical Hydrology

5.1 Approach

River flows throughout the study area can be estimated in many ways. Historic streamgage data was used directly for calibration of the model. For analysis of flood events, historic stream streamgage data was analyzed to develop statistical estimates of flood hydrographs at the upstream ends of the hydraulic model reaches and to estimate the coincident contribution of ungaged areas of local runoff that occur within the modeled reaches. Statistical estimates are representative of flood event runoff which has a certain probability of being equaled or exceeded during any given year. When these inputs are simulated with the hydraulic model, the resulting water surface elevations and inundation extents are representative of those that would occur with the corresponding hydrologic probabilities for a given physical condition, in this case the existing without project condition. An elevation time series representing Commencement Bay elevations is used for the downstream boundary of the model while flow time series data sets are used for upstream boundaries as well as internal boundaries where flow contributions occur.

5.2 Hydrologic Reaches

The five distinct reaches of the model were given in section 2 and are described further below. Five distinct reaches within the larger study area are: the lower portion of the White River, the lower portion of the Carbon River, the Puyallup River upstream of the confluence with the Carbon River, the Middle Puyallup River between the confluence with the Carbon and White Rivers and the Lower Puyallup River below the confluence with the White River. See Figure 20, below, for a schematic of the study area.



Notes:
Drainage area (sq. mi.) in parentheses
*Excludes 10 sq. mi. Lake Tapps assumed non-contributing

Figure 20: Basin flow Schematic; Arrows indicate direction and local inflows

5.2.1 Carbon River Reach

The Carbon River study area reach extends from the confluence to approximately RM 8.4. The drainage area at the confluence is 229 square miles and 91 square miles at the upstream end. The major tributary within the reach is South Prairie Creek with a drainage area of 90.3 square miles. South Prairie Creek enters the Carbon River at RM 5.9. Upstream of the reach, at RM 16.1, the USGS streamgage Carbon River near Fairfax (1209400) is located. There is also a streamgage located on South Prairie Creek (South Prairie Creek at South Prairie USGS #12095000). The drainage area upstream of this streamgage is 79.5 square miles. Another significant tributary in this reach is Voight Creek. This tributary enters the left bank of the Carbon River at RM 3.7. This tributary is ungaged.

From a comparison of annual peak flow from events where the annual peak at the Fairfax and South Prairie streamgages are attributed to the same event, in general, the peak flow values at the Carbon River location were found to be higher. Given this, it is assumed that the Carbon is more of a driver of high flows than South Prairie Creek. As such, flood statistics for this reach are based on the Carbon River with coincident values estimated for tributaries. Figure 20, below, shows a comparison of Carbon River and South Prairie Creek annual peak flow values for events where the annual peak resulted from the same event.

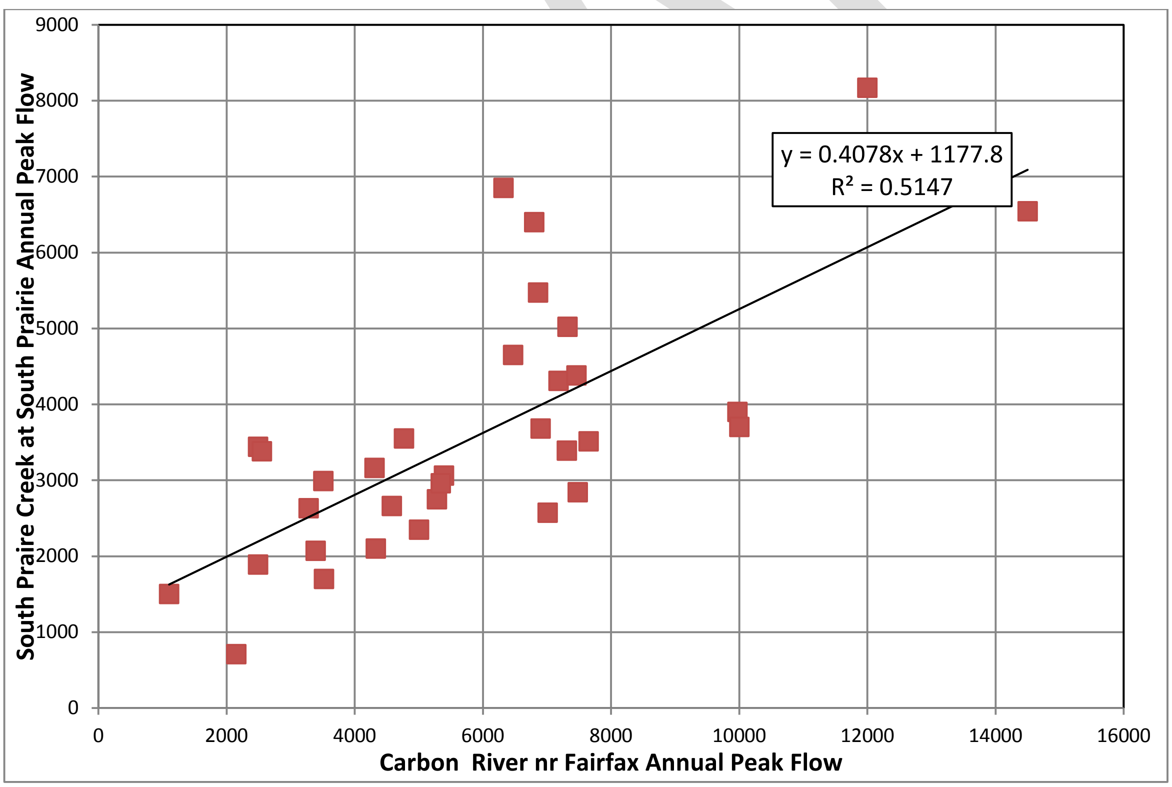


Figure 21: Comparison of South Prairie Creek and Carbon River Annual Peak Flow

The data indicates that in general, the Carbon River generates more flow during flood events than does South Prairie Creek (higher elevation basin with more orthographic precipitation). Given this, flood flow statistics for this reach are based on the Carbon River with estimated coincident flow for tributaries including Voight and South Prairie Creeks.

5.2.2 Upper Puyallup Reach

The Upper Puyallup Reach drains 188 square miles at the confluence with the Carbon and 130 square miles at the location of the upstream hydraulic model boundary (see Figure 21). Unlike the Carbon River reach, there is not a major tributary within the Upper Puyallup reach that significantly increases the drainage area. The largest tributary is Kapowsin Creek. This tributary has a drainage area of 29.6 square miles and is located towards the upstream end of the reach.

An active USGS streamgage, Puyallup River near Orting (12093500), is located within the reach at RM 25. The drainage area at the streamgage location is 172 square miles. Flood statistics for this reach are based on historical flow values from the Orting streamgage. The computed values are adjusted to the upstream boundary of the hydraulic model based on drainage area ratio (NHC, 2012). The ungaged local hydrologic contributions are estimated based on coincident analysis of peak flow timing and drainage area ratios.

5.2.3 White River Reach

The White River study area reach extends from the confluence with the Puyallup River upstream to approximately RM 10.5. The drainage area of the White River basin at the confluence with the Puyallup is approximately 495 square miles. At the upstream boundary the drainage area is approximately 470 square miles.

Mud Mountain Dam, a Corps flood risk reduction project, is located on the White River at approximately RM 29. The drainage area above Mud Mountain Dam is approximately 400 square miles. During flood events, Mud Mountain is operated to keep flows in the Lower Puyallup River below 50,000 cfs, if possible, and to keep discharge in the White River at or below 12,000 cfs if possible. The operation of Mud Mountain Dam is the largest driver of hydrologic conditions on the White River. For this effort, flood flow statistics on the White are based on Mud Mountain operation as the primary driver with estimates of coincident flows from the local contributing area between the dam and the confluence with the Puyallup.

Because of the heavy regulation on this river, peak flow below Mud Mountain does not necessarily follow a pattern similar to other parts of the Puyallup River basin. For larger floods, peak flows on the Lower White River generally occur during the evacuation of Mud Mountain Dam flood storage. During these larger floods Mud Mountain discharge is lowered to provide flood risk reduction to the Lower Puyallup reach.

5.2.4 Lower Puyallup Reach

The Lower Puyallup reach extends from Commencement Bay on Puget Sound upstream to the confluence with the White River at approximately RM 10.3. The area along this reach is heavily developed. Levees are located along most of both river banks. At the downstream end of the reach the drainage area is approximately 990 square miles and at the upstream end the drainage area is approximately 940 square miles.

Operation of the Mud Mountain Dam Project has a large influence on flood flows in this reach. The project is operated to keep flows below 50,000 cfs, if possible, along this portion of the system. A flow of 50,000 cfs is roughly the channel capacity of this reach. Given that the Project only regulates 40% of the basin, it is not possible to regulate the flows of all flood events to 50,000 cfs. The project is operated in a manner where during a flood event, discharge is reduced to maintain flows in the Lower Puyallup to below 50,000 cfs. To accomplish this, discharge can be reduced to zero. At this point, all of the flow in the Lower Puyallup is generated by the unregulated portion of the basin, which excludes the drainage area above Mud Mountain Dam. The unregulated portion of the Puyallup basin can generate peak flows above 50,000 cfs.

The hydrology for this reach is based on the unregulated flow generated by the basin excluding the portion on the White River above Mud Mountain Dam. The Mud Mountain Dam outflow is added to the unregulated flows according to the following scheme:

- For return intervals where the local and unregulated flow is 38,000 cfs or less, 12,000 cfs is added as the Mud Mountain outflow.
- For return intervals where the local and unregulated flow is between 38,001 and 49,999 cfs, the difference between 50,000 cfs and the local unregulated flow is added.
- For return intervals where the local and unregulated flow is greater than 50,000 cfs, no Mud Mountain flow is added. Presumably Mud Mountain discharge would be zero during the peak.

5.2.5 Middle Puyallup Reach

The Middle Puyallup Reach extends from the confluence with the White River (RM 10.3), upstream to the confluence with the Carbon River (RM 17.5). Over this 7.2 mile reach, the drainage area increase is 27 square miles. There are no major tributaries entering the Puyallup River within this reach.

5.3 Flow Frequency Analysis

Flow frequency analysis is the process by which historic streamgage data is analyzed to develop statistical estimates for various exceedence probabilities. Flow frequency curves were constructed for all gauged locations within the Puyallup Basin. A frequency analysis was also conducted for some locations outside the basin that were thought to be of use in developing hydrographs of ungaged sub-basins within the study area. In addition to a peak flow analysis, analyses for durations of one, three and seven days were also conducted to allow for the construction of balanced hydrographs for each event modeled. The frequency analysis was conducted by Northwest Hydraulic Consultants (NHC) during the spring of 2012 under contract to provide statistical model inflows (NHC, 2012). It should be noted that the flood statistics from this effort generated for the Lower Puyallup River Reach were not used. A frequency curve for the Lower Puyallup was generated considering Mud Mountain Dam (MMD) contributions and is described below.

For risk-based analyses, Corps of Engineers Publication ER 1110-2-1450 (Corps, 1994) instructs that frequency curves should not use flow values with the expected probability adjustment. Since this hydrology is part of the input to the Puyallup Basin G.I. which incorporates a risk-based analysis, the flood flow values are based on the 'computed' values as opposed to the 'expected' values. The expected probability is an adjustment made in the FDA computation of expected damages. FDA makes this adjustment as part of its Monte Carlo computation procedure. The record length goes into the uncertainty band about the frequency curve computation. In general, the shorter the record length, the higher the uncertainty band. Use of this data is discussed further in the FDA support appendix.

5.3.1 Unregulated Frequency Curves

The frequency analyses for the unregulated locations were conducted using traditional procedures documented in Bulletin 17B, "Guidelines for Determining Flood Flow Frequency", and EM 1110-2-1415 (Corps, 1993). Frequency curves were constructed for instantaneous peak, 1-day, 3-day and 7-day average flows to facilitate the construction of balanced hydrographs for use in the hydraulic model. Return intervals used are the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP events. Years where the annual peak flow could be attributed to a spring snowmelt event were excluded from the data set to ensure the statistics represented winter rain dominated flood events, which is the primary driver of floods in this basin.

Table 12 lists flow values for the Puyallup River near Orting (USGS streamgage No. 12093500) and the Carbon River near Fairfax (USGS Streamgage No. 12094000). Statistics from these two locations are the main drivers on the Upper Puyallup and Carbon River reaches. Note, in addition to the 1-day duration a '24-hour' duration is included as well. Since the 1-day daily average flow data represents the average of flow values from midnight to midnight, it is likely if moving 24-hour average flow values were used instead, the values could be higher. To reflect this (where possible), an analysis was performed (NHC, 2012) to determine scaling factors to adjust the 1-day values to 24-hour values. The 24-hour values are used in the construction of the balanced hydrographs instead of the listed 1-day flow values shown in Table 12.

Table 12: Tabulated Flood Flow Values by Return Interval

Station	Duration	Computed Winter Flood Flow Values by Return Interval (AEP)							
		0.5	0.2	0.1	0.05	0.02	0.01	0.005	0.002
Puyallup River near Orting	Peak	6920	10300	12600	14900	17900	20200	22500	25700
	24-hr	4910	7420	9200	11000	13400	15300	17200	19900
	1-Day	4390	6640	8230	9820	12000	13700	15400	17800
	3-Day	3290	4820	5890	6960	8400	9530	10700	12300
	7-Day	2370	3340	4000	4640	5480	6120	6770	7660
Carbon River near Fairfax	Peak	4730	7290	9140	11000	13600	15600	17800	20800
	24-hr	3190	4940	6200	7490	9260	10700	12100	14200
	1-Day	2920	4530	5690	6870	8490	9780	11100	13000
	3-Day	2030	3080	3860	4680	5850	6810	7850	9340
	7-Day	1390	2000	2440	2880	3490	3980	4500	5220

5.3.2 Lower Puyallup Regulated Frequency Curve

Developing flood flow statistics for the Lower Puyallup reach was not as straight forward as for other locations. Flood flows in this reach are significantly influenced by the operation of Mud Mountain Dam. Per the Water Control Manual, during a flood event the project is operated to keep flow in the Lower Puyallup below 50,000 cfs, if possible, and limit discharges to 12,000 cfs or less from the dam. As discussed earlier, for extreme events the unregulated local inflow by itself can reach levels greater than 50,000 cfs. 50,000 cfs is approximately the channel capacity of the Lower Puyallup reach.

An analytical frequency curve of the unregulated local (everything except Mud Mountain Dam) was constructed. At each return interval, if the unregulated local was less than 50,000 cfs, the difference between 50,000 cfs and the unregulated local flow or 12,000 cfs, whichever is less, was added to the unregulated local value to obtain the regulated flow. For the 2% AEP and more frequent events it is assumed that MMD could reduce discharge to zero cfs if needed to keep the Lower Puyallup flow below 50,000 cfs. For 1% AEP less frequent events, it is assumed that at the peak of the local inflow, MMD would not be able to reduce outflow to zero, as the project would be on the special gate regulation schedule which specifies a minimum outflow. Table 13 shows the regulated Lower Puyallup frequency curve. A description of how these estimates were developed follows below.

Table 13: Lower Puyallup regulated frequency Curve

AEP %	Unregulated Local flow (cfs)	MMD Flow Contribution to Peak (cfs)	Regulated Lower Puyallup Frequency Curve Peak Flow Values (cfs)
0.2	77713	13,900	91613
0.5	65117	10000	75117
1	56283	5000	61283
1.25	53560	0	53560
1.5	51375	0	51375
1.75	49557	444	50000
2	48001	1999	50000
3	43394	6606	50000
5	37816	12000	49816
10	30603	12000	42603
20	23692	12000	35692
50	14534	12000	26534

Peak Unregulated Local Inflow Frequency Analysis

As a starting point towards constructing a regulated frequency curve, a frequency curve was constructed for the peak unregulated local flow at Puyallup. This is based on subtracting recorded Mud Mountain Dam discharge from the peak flow at the Puyallup at Puyallup USGS streamgage (No. 12101500) to compute the unregulated local flow at Puyallup. Prior to 2003 Mud Mountain discharge was measured via USGS streamgage No. 12098500 (White River near Buckley) dating back to Water Year 1929. The river cross section at this location has become very unstable and as such, it was difficult to maintain an accurate rating curve resulting. Since this time, measurement of Mud Mountain discharge has been based on the outlet works rating curve(s).

The travel time from Mud Mountain Dam to the Puyallup streamgage location is approximately six hours, so ideally the Mud Mountain discharge value used in the local inflow calculation is six hours earlier in time than that of the peak flow value at Puyallup. Instantaneous annual peak flow values for the Puyallup streamgage are available from 1915 to present, unfortunately corresponding (shifted six hours) flow values representing Mud Mountain Dam outflow (or White River flow prior to 1942 when the dam was built) are not widely available. For the most part, we were limited to using daily average flow values from 1929 to 2003 as a starting point for constructing this peak unregulated local flow data set. This required developing a daily-to-peak scale factor to make the conversion.

Unregulated daily flow values were computed by subtracting the daily flow values at streamgage 12098500 (Mud Mountain Dam discharge) from daily flow values at the Puyallup at Puyallup streamgage. From the results, the peak daily values were determined. An estimate of the peak daily flow was made by looking at the project operation log book for a known instantaneous flood peak at the Puyallup streamgage, to see if the Mud Mountain discharge could be ascertained approximately six hours earlier. Also required for this calculation was the time of peak at the Puyallup streamgage. Generally the USGS annual instantaneous peak data set does not include this. The regulation logbooks as well as other USGS and Corps records contained the time of peak at Puyallup for some floods. In all there were eight events where it was possible to make a direct estimate of the instantaneous local unregulated peak flow. Table 14 shows the events, the computed 1-day average flow, the computed instantaneous peak flow and the adopted factor to scale the one-day value to the instantaneous peak value for each event.

The scale factor data was modified slightly to adjust the daily flow values to peak flow values for the frequency analysis. Based on the eight years shown in Table 14, there is a lot of variability from flood to flood in these between the daily average values and the peak values. The average of the factors for these eight events is 1.49. Interestingly, the highest daily average value actually had one of the lower instantaneous peak values. Conversely, the event with the lowest daily average local inflow value had one of the higher instantaneous peak flow values. Table 14 shows two different scaling factors. One is the average of 1.49 and another is 1.35. Looking at the eight years where we have a directly calculated instantaneous peak flow value, when a factor of 1.49 is applied to the corresponding daily values six out of the eight were higher than the directly calculated value—three of them higher by more than 5,000 cfs. A smaller factor of 1.35 brought down the average difference between the adjusted values and brought the highest flow values in the record (1996 and 2009) more in line with the eight directly calculated values. Given this, a factor of 1.35 was adopted to adjust the daily local inflow values to peak values. This was essentially weighting the process towards the largest and most recent events.

Table 14: Scaling Factor Selection for Local Inflow

Year	Computed 1-Day Average Local Inflow	Comp Inst. Peak Local Flow	1-Day to Peak Scale Factor	Global Factor 1.49	Adopted Factor 1.35
1978	25200	33200	1.32	37548	34020
1984	20100	28500	1.42	29949	27135
1987	24400	35150	1.44	36356	32940
1990	19850	42150	2.12	29577	26798
1991	21880	40610	1.86	32601	29538
1996	35060	46700	1.33	52239	47331

2007	36000	39700	1.10	53640	48600
2009	35900	48200	1.34	53491	48465

The local inflow annual peak flow record consists of values from water years 1929 through 2009. All but eight of the years consist of computed daily average flow values adjusted by a factor of 1.35 to give peak flow values. Table 15 lists the top 10 peak flow estimates in the systematic record for the local inflow. Note that two of the values are based on the daily average value and the adjustment factor while the remainder are the eight years where a direct calculation of instantaneous peak was made.

Table 15: Top Ten Annual Local Inflow Peak Flow Estimates

Rank	Water Year	Peak Flow
1	2009	48200
2	1996	46700
3	1934[1]	46271
4	1990	42150
5	1991	40610
6	2007	39700
7	1987	35150
8	1965[1]	34654
9	1978	33200
10	1984	28500

[1]Flow estimate based on 1-day average times a factor of 1.35

A Bulletin 17B frequency analysis was conducted using this record using the Corps of Engineers software package HEC-SSP. The weighted skew option was selected with a regional skew value of zero and a regional skew mean square error of 0.302. Table 16 lists the computed and expected flow values for various return intervals at the Puyallup River at Puyallup Streamgage.

Table 16: Tabulated Peak Unregulated Local Inflow Flow Statistics at the Puyallup at Puyallup Streamgage

AEP %	Computed Flow	Expected Flow
0.2	77713	82865
0.5	65117	68370
1	56283	58478
1.25	53560	55472
1.5	51375	53077
1.75	49557	51094
2	48001	49405
3	43394	44440
5	37816	38507
10	30603	30954

20	23692	23834
50	14534	14534

Adjustment for Mud Mountain Dam Regulation

The contribution from Mud Mountain Dam was added to the unregulated local estimates shown previously in Table 16. The Water Control Plan for Mud Mountain Dam allows the project to be operated to keep flow in the Lower Puyallup River below 50,000 cfs and limit discharge to 12,000 cfs or less (USACE, 2004), if possible. During the peak of a large flood event, the bulk of the flow in the Lower Puyallup is generated by the unregulated local inflow, as evidenced by the local inflow frequency curve (tabulated in Table 16).

During a flood event, the anticipated magnitude of the peak local inflow has the greatest influence on decisions regarding releases from Mud Mountain. Clearly, if the local inflow is anticipated to be greater than 50,000 cfs, then if at all possible, releases would be held to zero during the peak of the local inflow hydrograph. If the local inflow is anticipated to be lower than 50,000 cfs, then, based on the Water Control Plan the operator technically (probably assuming perfect information) could decide to release some amount of water, up to 12,000 cfs, as long as this does not push the flow in the Lower Puyallup above 50,000 cfs. This basic decision process for Mud Mountain releases is the basis for the regulated frequency curve for the Lower Puyallup shown previously in Table 13, where the regulated frequency curve for the Lower Puyallup is the sum of the local unregulated flow and the Mud Mountain contribution values for each return interval. The MMD contribution is 12,000 cfs until the flow at Puyallup reaches 50,000 cfs. When local inflow becomes greater than 50,000 cfs, zero is entered since the Water Control Plan intends to keep the flow at Puyallup below 50,000 cfs, if possible. Once the special discharge curve is reached flows exceed 50,000 cfs.

Discussion

Due to the complexities of the regulated Puyallup Basin system, development of the regulated frequency curve for the Lower Puyallup was not as straight forward as the other unregulated locations and posed some challenges and required some assumptions. One notable assumption inherent to the regulated frequency curve tabulated in Table 13 is that for at least events up to a 0.2% AEP exceedence probability, Mud Mountain Dam could be operated such that discharge would be reduced to zero during the local inflow hydrograph peak on the Lower Puyallup. This assumption is based on work done by Seattle District Corps of Engineers (USACE, 2011).

Another item worth noting is the Mud Mountain contribution to the regulated frequency curve, shown previously in Table 13. For the return intervals where the peak local inflow is below 50,000 cfs, the regulated frequency curve assumes that Mud Mountain would discharge an amount of water that is either no more than 12,000 cfs or when added to the local inflow equates to 50,000 cfs. Based on the Water Control Plan, in theory this is the operation that could occur if project operators had perfect information regarding the inflow in the basin. However, the project may be operated more conservatively during a real-world flood due to uncertainty in weather forecasts. Of the eight floods presented in Table 15, Mud Mountain releases could have been higher than those that actually occurred. This likely reflects the uncertainties, such as forecasts, that go into decision making during a flood event and are ultimately reflected in the operation. At this point, for the purposes of communicating flood risk in the G.I. study, the conservative approach with respect to Mud Mountain operation is used for the regulated frequency curve.

Another limitation is the data used in the analysis. The local inflow flow values are calculated as opposed to measured at a streamgage. Furthermore, all of the values in the record except for the eight years shown in Table 15 were adjusted to peak values by a one size fits all factor.

Based on an inspection of previous flood events the developed curves may be a bit conservative. For some past flood events where the local inflow ended up being less than 50,000 cfs, outflow from the project was actually reduced to zero when in hindsight it did not have to be to keep the flow in the lower Puyallup below 50,000 cfs. The hydraulic modeling used for the Lower Puyallup looked at the computed flow at the first few upstream cross sections in the model as a streamgage as to whether we were getting the conditions we wanted. For example, if we were simulating the 2% AEP event, we ran our best attempt at the 2% AEP hydrographs and looked at the computed flow at the first few upper cross sections in the Lower Puyallup reach. The goal was to have computed flows roughly equal 50,000 cfs. This was more difficult to check for infrequent return intervals as the flow values associated with these events exceeds the channel capacity of the river and the model computes a significant amount of floodplain flow. It should be noted that the flow values compute here for the Lower Puyallup are higher than those used in previous studies.

5.4 Coincident Flows

The coincident flow analysis provides a means to determine flow values at interior study area locations which are coincident to the statistical flow values of the main reach flow (NHC, 2012). Relationships for flows coincident to annual peak flows at the Puyallup near Orting and Carbon near Fairfax were determined at areas of interest for use on these reaches. Relationships for flows coincident to the Puyallup at Puyallup unregulated flow were also determined. The coincident flow relationships to the Puyallup at Puyallup location were conducted as part of the NHC hydrology work (NHC, 2012). The coincident flow relationships for the Puyallup River near Orting and Carbon River near Fairfax were done by Corps staff and are documented below. For the purposes of quantifying flood flow statistics for the Middle Puyallup reach, it is assumed the return interval of a flood on the Lower Puyallup Reach is the same as on the Middle Puyallup Reach.

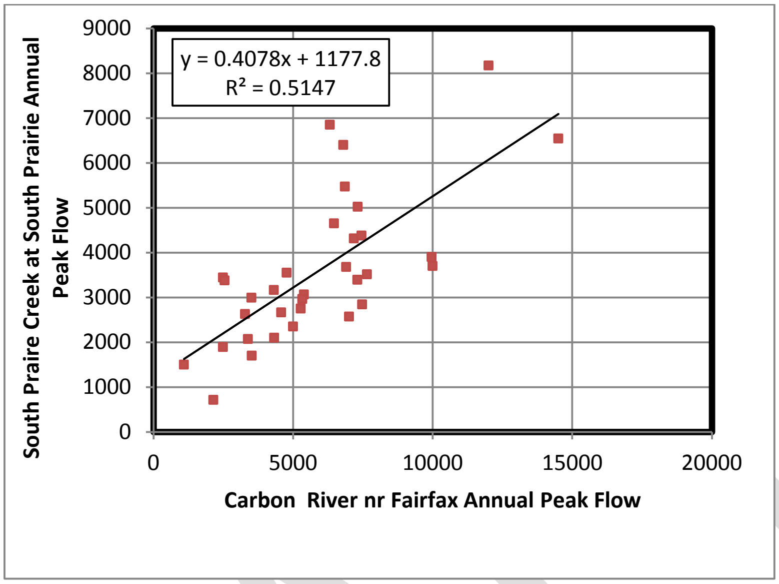


Figure 22: Carbon River nr Fairfax versus South Prairie Creek Peak Flow

Figure 22 compares annual peak flow at the Fairfax streamgage on the Carbon River (USGS No. 12094000), which is just upstream of this study's hydraulic model Carbon River upstream boundary location, with the annual peak flow on South Prairie Creek (USGS No. 12095000). Figure 23 compares the annual peak flow on the Puyallup River near Orting (USGS No. 12093500) with that of South Prairie Creek and Figure 24 compares the Puyallup River near Orting with the Mashel River near La Grande (USGS No. 12087000). The Mashel River is actually in the Nisqually River basin but its proximity to the Puyallup may make it useful as a pattern hydrograph for sub-basins above Orting.

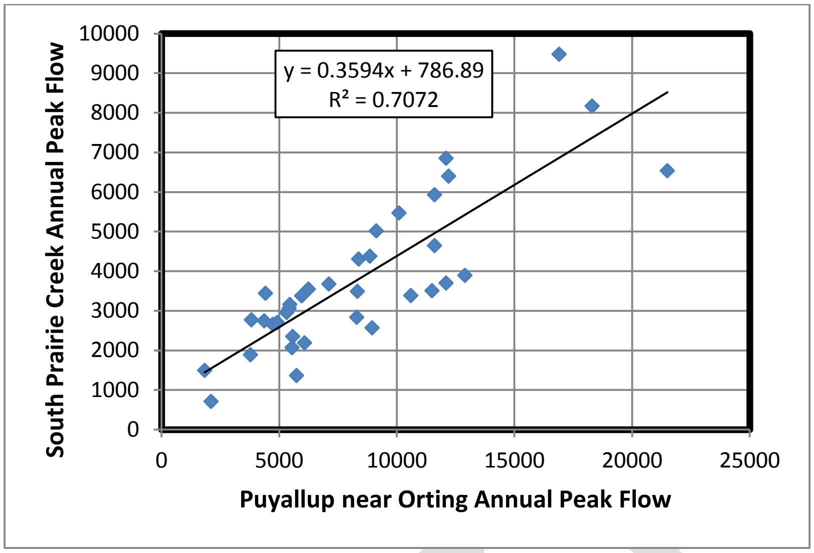


Figure 23: Puyallup River at Orting versus South Prairie Creek peak Flow

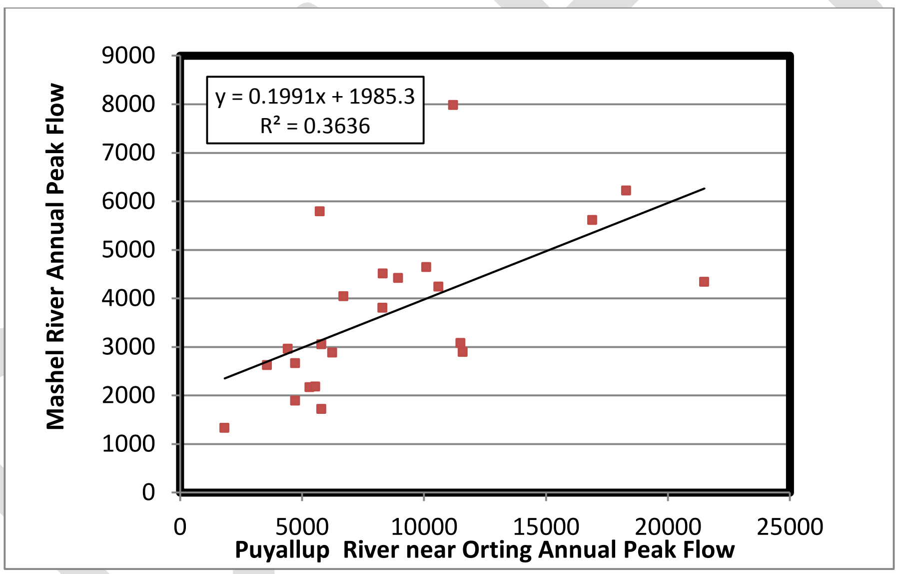


Figure 24: Puyallup River near Orting versus Mashel River near La Grande Annual Peak Flow

This system is highly variable in terms of the hydrologic contributions of various sub-basins from flood event to flood event. This is evidenced in the low R^2 values shown on the regression plots in Figure 22, Figure 23, and Figure 24. In reality, the coincident flow values only served as a starting point for the statistical flow hydraulic model simulations. As it turned out, some adjustments had to be made to the hydrographs to produce flood flow statistics that better matched the computed values at some locations. This is discussed in the Hydraulic Simulations section of this document.

5.5 Balanced Hydrographs

A balanced flood is one of equal exceedence probability for specified possible durations of the flood event. Balanced hydrographs are produced by selecting a pattern hydrograph based upon historic flood events and then scaling that pattern to maintain target flows for specified durations. Balanced hydrographs which reflect the flow values of a given return interval computed for the peak, 24-hr, 3-day, and 7-day durations were constructed for most of the locations where frequency curves were developed (NHC 2012). Most of the hydrographs were constructed based on flow values which are coincident to the unregulated flow at the Puyallup at Puyallup streamgage. As the approach to the statistical simulations was thought through in more detail, adjustments to some of the hydrographs associated with the Upper Puyallup, Lower Carbon and White River reaches were made to ensure the resulting hydraulic conditions were representative of true 50% through 0.2% AEP events.

In an effort to save time over the trial and error method of constructing the hydrographs using an Excel spreadsheet, most of the balanced hydrographs were constructed using the HyBart computer program (USACE, 2012). This program had been successfully used by Sacramento District and essentially performs the methods given in IDH-5 (HEC, 1975) to produce balanced flood hydrographs. The program is given a pattern hydrograph and statistical peak flows and produces hydrographs for statistical events of specified duration. Results for this study were mixed possibility due to regional differences in nesting of flood quintiles (i.e. events of varying magnitude may not follow the same genera pattern). The program seemed to have trouble maintaining a specified pattern hydrograph shape while maintaining the nesting of flood quintiles specified by the user. As a result the balanced hydrograph shape did not resemble the shape of observed hydrographs at a few locations, where it was necessary to manually correct the results.

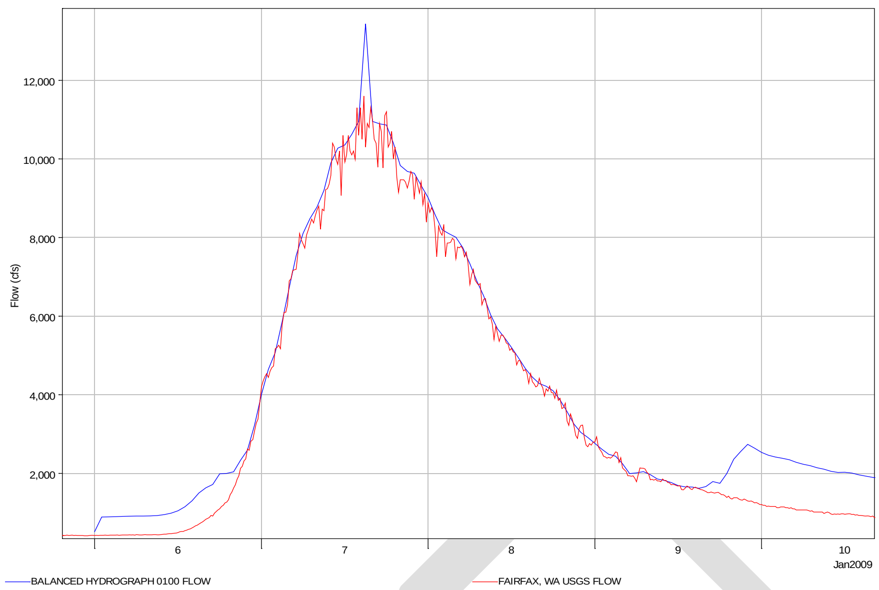


Figure 25: Carbon River Balanced and Observed Hydrograph

Figure 25, shows the 0.02 annual probability exceedence balanced hydrograph for the Carbon River near Fairfax location (blue) and the pattern observed hydrograph from the 2009 flood. Note, to balance all the durations and achieve the correct peak flows, the unnatural looking spike at the peak was required. Figure 26 shows the balanced and observed hydrographs for South Prairie Creek (USGS No. 12095000). Here the balanced hydrograph has a more natural appearance than the balanced one for the Carbon River. The goal was to match the shape of an observed pattern hydrograph, while keeping a statistical peak flow. So the peaks will not match in these figures. More discussion on the balanced hydrographs is given by NHC (NHC 2012).

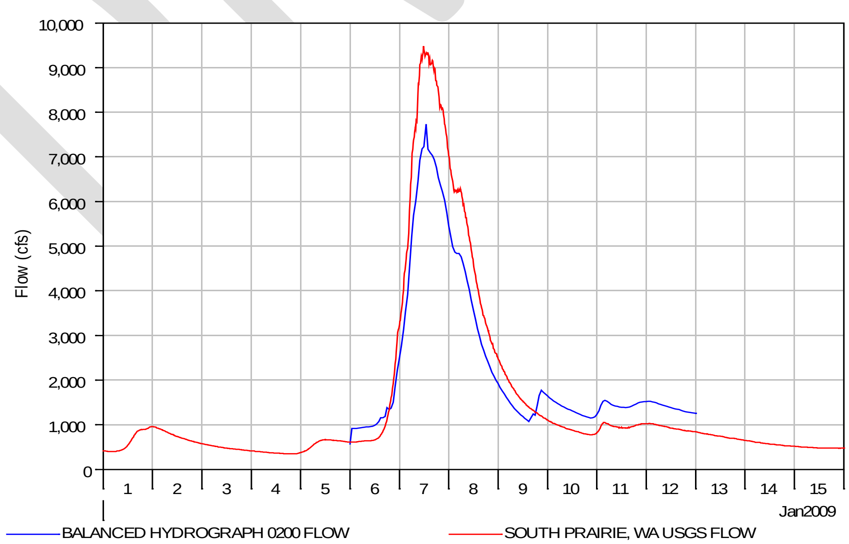


Figure 26: South Prairie Creek Balanced and Observed Hydrographs

5.6 Ungaged Sub-basin Hydrology

There are numerous locations within the hydraulic model domain where input hydrographs are required. Most of these locations are ungaged sub-basins. These internal inputs to the hydraulic model were estimated based on a pattern hydrograph from a gaged location. Details on selecting and scaling the streamgage location hydrographs to serve as patterns for various ungaged sub-basins within the model are documented by NHC (NHC 2012). Tables 9 and 10 of the NHC reference list the model location and the pattern used. During the hydraulic model simulations, some adjustments were made to some of these ungaged hydrographs to better match locations within the model domain where it was possible to compute (such as the Puyallup River near Orting and the Puyallup at Puyallup streamgages) flood flow statistics.

5.7 Hydrograph Timing

An analysis was conducted to determine the timing of input hydrographs in the model relative to observed hydrographs at the Puyallup @ Puyallup streamgage. This analysis is documented in the NHC reference (NHC 2012). Due to variability between the different locations from event to event during the simulations of the statistical flood events, some adjustments of hydrograph timing was required.

5.8 Downstream Boundary Condition

An analysis of Puyallup River peak flow versus Commencement Bay tidal elevation was conducted to determine an appropriate downstream boundary to use with flow-frequency estimates. Table 17 shows the resulting joint probability and corresponding tidal elevation and Puyallup River Flow.

Table 17: Joint Puyallup River Flow-Commencement Bay Elevation Probabilities

Scenario	TWL (ft. MLLW)	TWL (ft. NAVD88)	Puyallup River Flow	Joint Probability
1	13.83	11.22	7000	Joint TWL-Q 50% AEP (RP = 2)
2	13.84	11.23	22000	Joint TWL-Q 20% AEP (RP = 5)
3	13.76	11.15	33100	Joint TWL-Q 10% AEP (RP = 10)
4	13.8	11.19	39300	Joint TWL-Q 5% AEP (RP = 20)
5	13.85	11.24	43900	Joint TWL-Q 2% AEP (RP = 50)
6	13.87	11.26	48000	Joint TWL-Q 1% AEP (RP = 100)
7	13.97	11.36	50200	Joint TWL-Q 0.2% AEP (RP = 500)

TWL: Total Water Level Recorded at Tacoma Station 9446484

Figure 27 graphically shows the relationships between Puyallup River peak flow and Commencement Bay elevation from which the data in the table was determined. The figure is read from flow on the vertical axis, over to a return period contour line, and down to the corresponding water level on the horizontal axis (or vice versa). Contours were determined from the observed data plotted on the figure by the probability function.

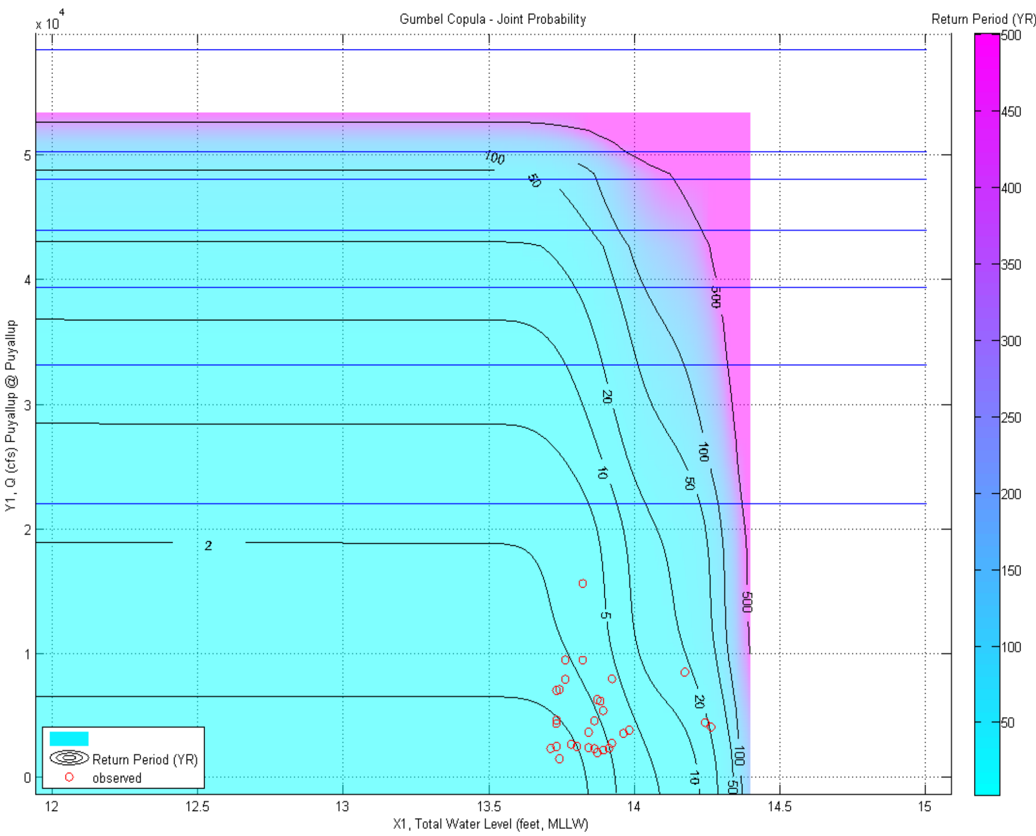


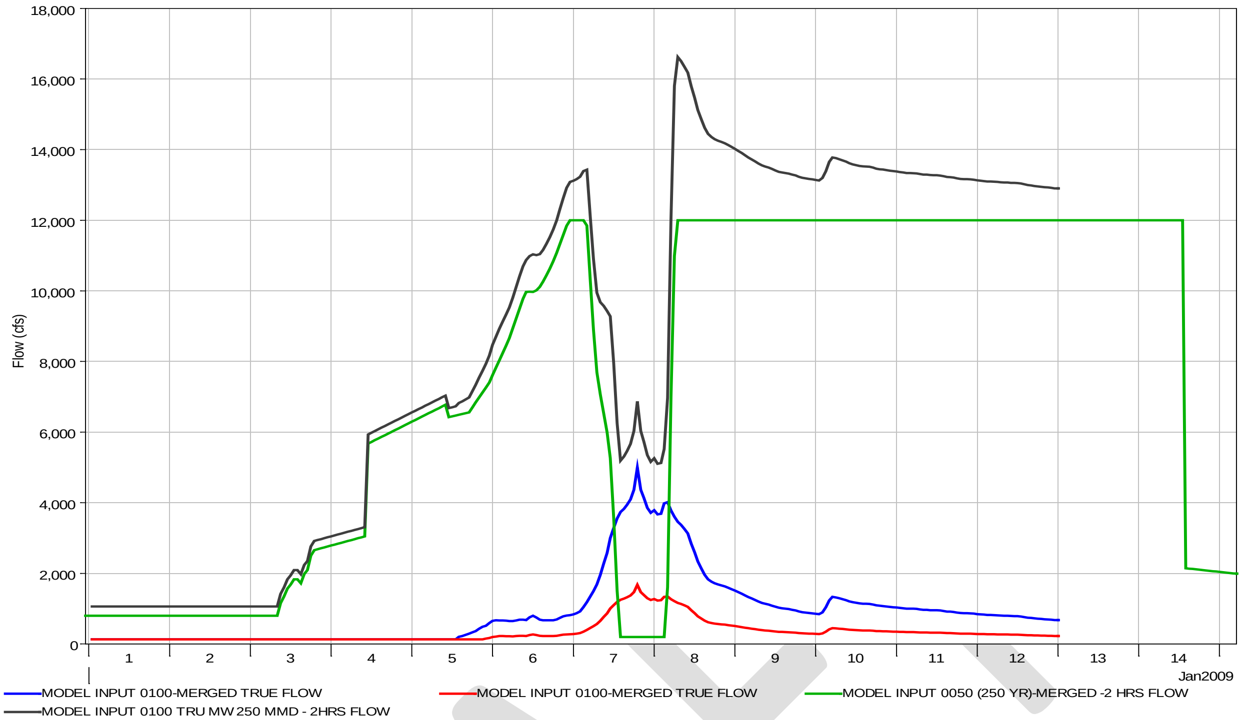
Figure 27: Puyallup River Peak Flow versus Commencement Bay Elevation

Given that the range of possible Commencement Bay elevations does not vary much outside a narrow range of 0.14 feet for 50%-0.2% AEP coincident events, a downstream elevation of 11.36 feet NAVD88 (13.97 ft. MLLW) was used for all the simulations. This is reflective of the lack of correlation between high riverine flow events and high tidal events (beyond the typical seasonal winter highest tides occurring with winter flood season). This value is similar to the base flood elevation (1% AEP event) of 11.1 ft. determined for the preliminary 2014 Flood insurance Study (FEMA, 2014). The slight difference may be due to the probability function used in analysis (Gumbel-Copula for this effort versus Log Pearson III for the FEMA effort). Commencement bay (and the lower reach of the Puyallup River) is sheltered from Puget Sound and wave run-up is expected to be minimal. Additionally, wave run-up was not included in the estimate because no coastal protection measures are in the design. Sea level change was incorporated into the future conditions analysis and the reader is referred to the H&H Future Conditions Modeling Appendix for detail.

5.9 Mud Mountain Dam Discharge Hydrographs

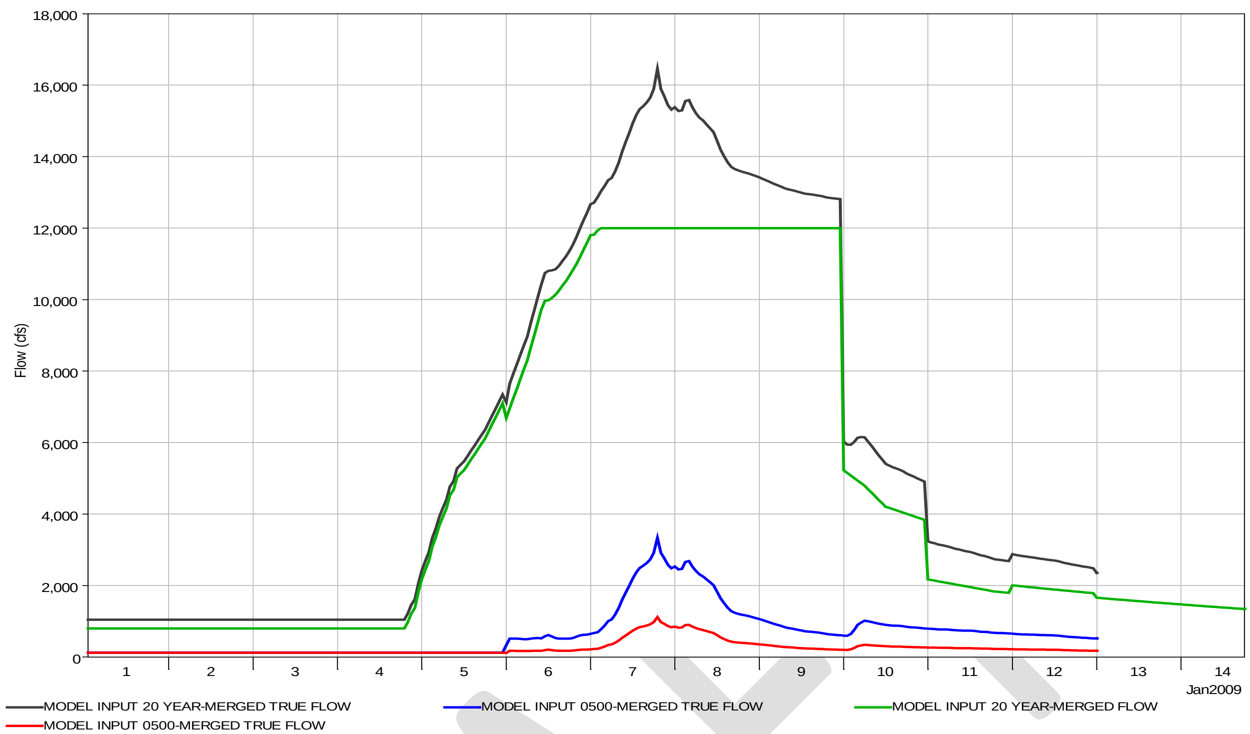
While the upstream hydraulic model boundary of the White River reach is not at Mud Mountain Dam, the Mud Mountain discharge is an important component of this hydrograph. The starting point for these hydrographs is the Corps 2011 work that looks at Mud Mountain operation under different release scenarios (Corps 2011). This effort developed outflow hydrographs for various return interval flood events. The outflow hydrographs from Mud Mountain were added to the local inflow hydrograph to obtain an upstream model boundary hydrograph for the White River.

Figure 28 shows the components of the 1% AEP upstream boundary hydrograph for the White River. The components that make up this hydrograph are the hydrograph for Boise Creek, a hydrograph representing the ungaged local inflow for the portion of the basin between the hydraulic model upstream boundary and the dam, and the outflow hydrograph for Mud Mountain Dam. The upper gray line represents the sum of these three inputs. Figure 29 contains the same inputs for the 5% AEP event. Note that for this event Mud Mountain Dam discharge is maintained at 12,000 cfs instead of being reduced as occurs in Figure 28 (the 0.01 event). This is because the 5% AEP peak local inflow (see Table 13) is low enough that 12,000 cfs can be discharged from Mud Mountain without the flow in the Lower Puyallup exceeding 50,000 cfs. Also note that the peak flows for these two events is nearly identical but occur at different points in the event. For the 0.05 probability event, the White River peak occurs with the peak of the White River local inflow hydrograph. For the 0.01 event the White River peak occurs when Mud Mountain discharge is ramped up after the local peak has passed.



Red-Boise Creek Blue-Middle White Local Green-MMD Outflow Gray-Composite Hydrograph

Figure 28: Hydrologic Components of White River Upstream Boundary Hydrograph (0.01 Prob)



Red-Bosie Creek Blue-Middle White Local Green-MMD Outflow Gray-Composite Hydrograph

Figure 29: Hydrologic Components of White River Upstream Boundary Hydrograph (0.05 Prob)

6.0 Hydraulic Simulations of Statistical Hydrology

6.1 Approach

The production hydraulic model simulations were run for the 50% through 0.2% AEP flood events for the existing conditions.. Hydrologic data shows that there is a significant amount of variability from flood event to flood event spatially within the basin in terms of flood return interval. To address this, the statistical flood model runs were done in two groups. One group simulated 50% through 0.2% AEP floods on the Lower Carbon, Upper Puyallup, White, and Middle Puyallup reaches. While the other group simulated 50% through 0.2% AEP floods on the Lower Puyallup reach. Future conditions are discussed in a separate H&H Appendix, which incorporates sediment predictions, project measures, and climate change/ Sea level rise predictions.

For the without project condition (existing and future-Lower Puyallup reach only), levee breaches were incorporated in the simulations at three locations on the Lower Puyallup River. Breach locations are listed below in Table 18 and are based on FDA index points. Flood extents resulting from these failures are shown at the end of the report in Figure 45 through Figure 48.

Table 18: Levee Breach locations

Levee	Lateral Structure	XS	Bank	RM	Time (hrs.)	Dimensions (feet)
Puyallup Left Bank	12500	9944.706	Left	1.9	2.33	153x10
North Levee Road	42806.1	42525.43	Right	8	2.48	170x10
Puyallup Right Bank	13894.84	10752.54	Right	2	2.33	153x10

Parameters required to simulate a levee failure, such as trigger elevation, breach dimensions and formation time of breach, were provided by geotechnical engineers and incorporated in the hydraulic model. Each breach was simulated in isolation (one at a time) for the existing and future conditions. Breaches at the three locations were assumed to occur anytime river water surfaces were modeled as higher than the pre-determined breach trigger elevations. At the three locations, under conditions where a levee would have been overtopped absent a breach, a breach was assumed to occur and simulated as such by the model. Everywhere else it was assumed water could only leave the river via overtopping of structures adjacent to the river. Water surface elevations in the river are based on the no-breach condition to remove any breach drawdown influence elsewhere in the river. This is conservative and neglects the joint probability or breaches occurring.

This modeling approach resulted in the computation of floodplain water surface elevations for each of the four conditions (base without project, future without project, base with-project, and future with-project) and for each of the eight FDA return intervals. The results were "composited" by selecting the highest computed floodplain (storage area) water surface for each scenario and return interval. For example, if storage area 600025, the model computed for the 0.5% AEP existing condition event 20 feet for Breach Scenario A, 21 feet for Breach scenario B, 19 feet for Breach Scenario C and 18 feet for Breach Scenario D (all hypothetical values for illustrative purposes), the water surface elevation value used in FDA for the station 600025 corresponding to the 0.5% AEP existing condition is 21 feet.

6.1.1 Upper Puyallup and Carbon Reaches

The upstream boundary condition hydrograph for the Carbon River reach is the balanced hydrograph developed for the Carbon River near Fairfax stream streamgage scaled to the upstream boundary location based on the composite drainage area/rainfall ratio factor (NHC, 2012-Table 9). The internal boundaries on the Carbon shown previously in Figure 21 (i.e. local inflows, Voight Creek inflow, and South Prairie Creek inflow) are based on South Prairie Creek as a pattern. The balanced South Prairie Creek hydrographs (50% through 0.2% AEP) are scaled based on the regression equation from Figure 22, using the return interval of interest peak flow for the

Carbon River near Fairfax frequency curve. Since South Prairie Creek is an interior boundary condition, the resulting hydrograph from the regression scaling is directly used here. For the other internal boundaries within this reach, the South Prairie hydrograph is used as the pattern (NHC, 2012 table 10). For these locations, the coincident South Prairie Creek hydrograph was then scaled based on the composite drainage area/rainfall factor (NHC, 2012 table 10).

The Upper Puyallup reach is treated in a similar manner. One key difference is that the Puyallup near Orting USGS streamgage is located mid-reach. This allowed for a comparison of the modeled flow values with those from the frequency curves. Some adjusting of the input hydrographs was done to make sure the modeled flow values at the Orting streamgage location matched the computed statistical flow values for the return interval of interest.

The balanced hydrograph developed for the Orting streamgage was adjusted to the upstream model boundary using the drainage area ratio. The ungaged local inflow was based on the South Prairie Creek coincident hydrograph scaled with the composite drainage area/rainfall factor (NHC, 2012 table 10).

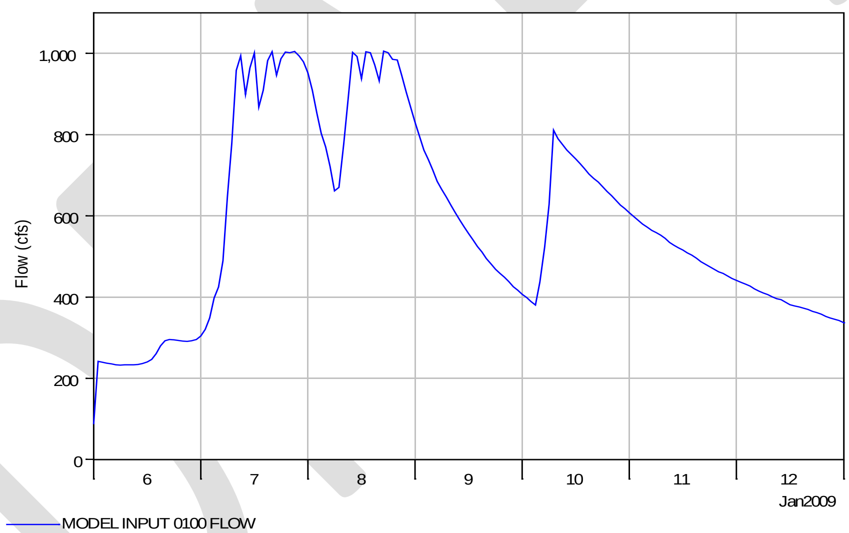


Figure 30: Kapowsin Creek Computed 0.01 Balanced Hydrograph

Initial model runs for most return intervals yielded peak flow values at the Orting streamgage location which were generally lower than that which was computed in the peak flow frequency analysis. The largest tributary in this reach is Kapowsin Creek which has a drainage area of 25.9 square miles. While Table 10 in the NHC report (NHC, 2012) shows the hydrological input for Kapowsin Creek as being the coincident balanced hydrograph for Kapowsin Creek, this was changed for a couple of reasons. One reason was that the balanced hydrograph computed for this location using the HyBart program looked nothing like a typical hydrograph from streamgage locations within the basin. Figure 30 shows the Kapowsin hydrograph computed using the HyBart program. Beyond the shape another concern was with the flow data period of record. The Kapowsin Creek period of record for peak flow values is from Water Year 1928 through 1970. This period does not capture the large floods from the 1990's and 2000's. Based on the available data for Kapowsin Creek, the computed 1% AEP peak flow value is 899 cfs (see Table 3 in NHC, 2012) or 34.7 cfs per square mile. As a comparison, the Mashel River, which lies close by in the Nisqually River basin, has a computed 1% AEP peak flow value of 102 cfs per square mile. It seems reasonable to assume that had the Kapowsin Creek streamgage (USGS 12093000) continued to operate up to the present, the peak flood flow values for given return intervals would be higher. As such, the balanced hydrograph constructed for the Mashel River was used as a pattern hydrograph for Kapowsin Creek. This hydrograph was scaled based on the coincident flow analysis with the Orting streamgage (Figure 24) and the drainage area ratio between Kapowsin Creek and the Mashel River. Even with this approach the upstream boundary hydrograph had to be increased by 10% to 20% (note in the hydraulic model flow files a value for 'multiplier' is sometimes added to address this) to match the peak flow values from the frequency curve.

This reach had the benefit of a streamgage mid-reach to allow for fine tuning of the coincident analysis flows to better produce hydraulic conditions representative of the return intervals of interest. Since a mid-reach streamgage does not exist (for the purpose of quantifying flood statistics) in the modeled reach of the Carbon River, no adjustments to these boundary hydrographs were made based on Upper Puyallup adjustments. While there is no way of knowing for sure, if the Carbon River hydrographs do follow a similar pattern of being too high, the water surface elevations used in this study may be too high. Given the variability of hydrographs at various streamgage locations within the basin (as evidenced by poor coincident correlations for flow and timing) it was decided not to extrapolate the Puyallup adjustments to the coincident analysis to the Carbon. Figure 31 through Figure 38 show the maximum water surface profiles for the 50% through 0.2% AEP model simulations.

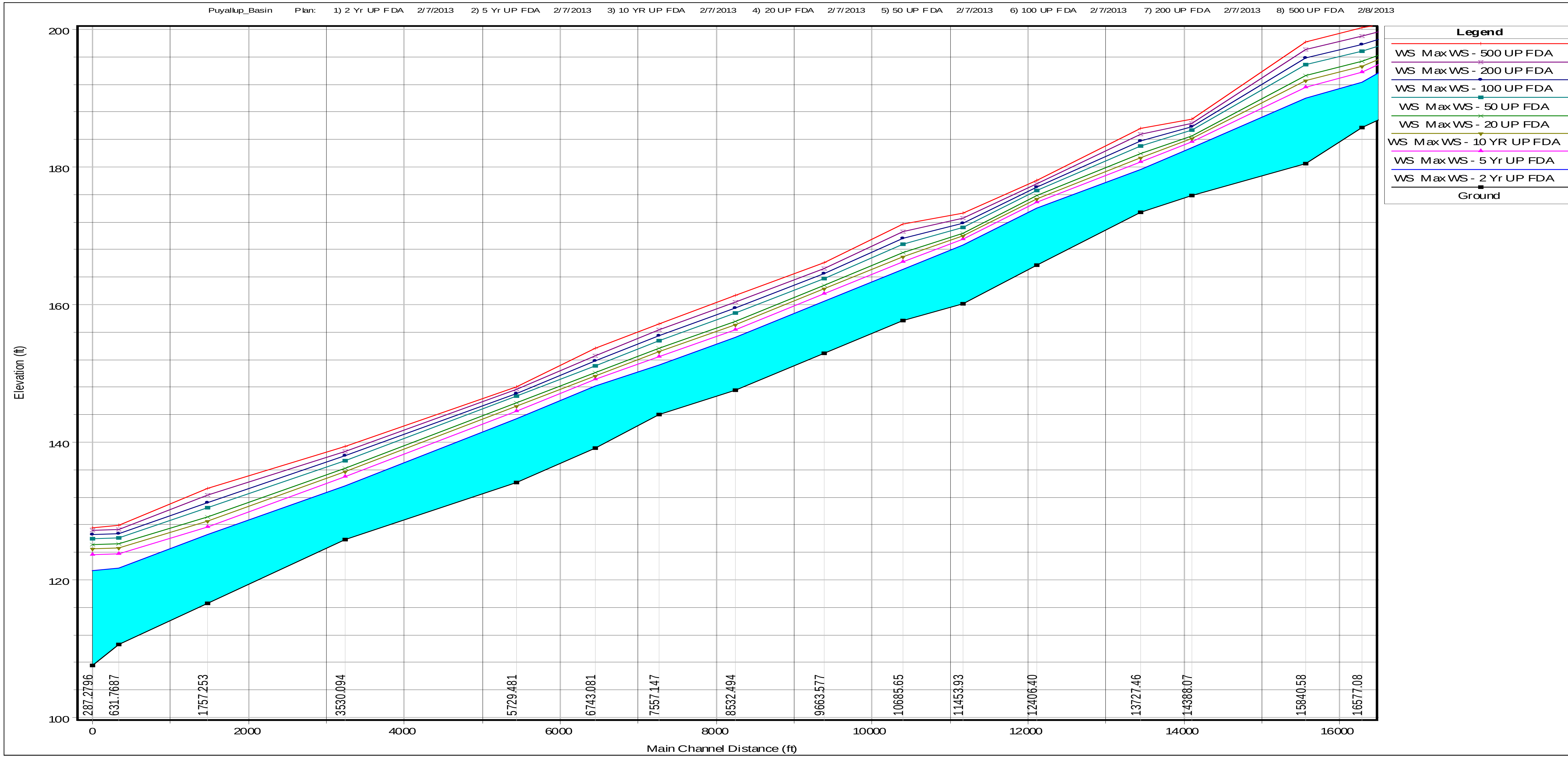


Figure 31: Water Surface Profiles Upper Carbon River

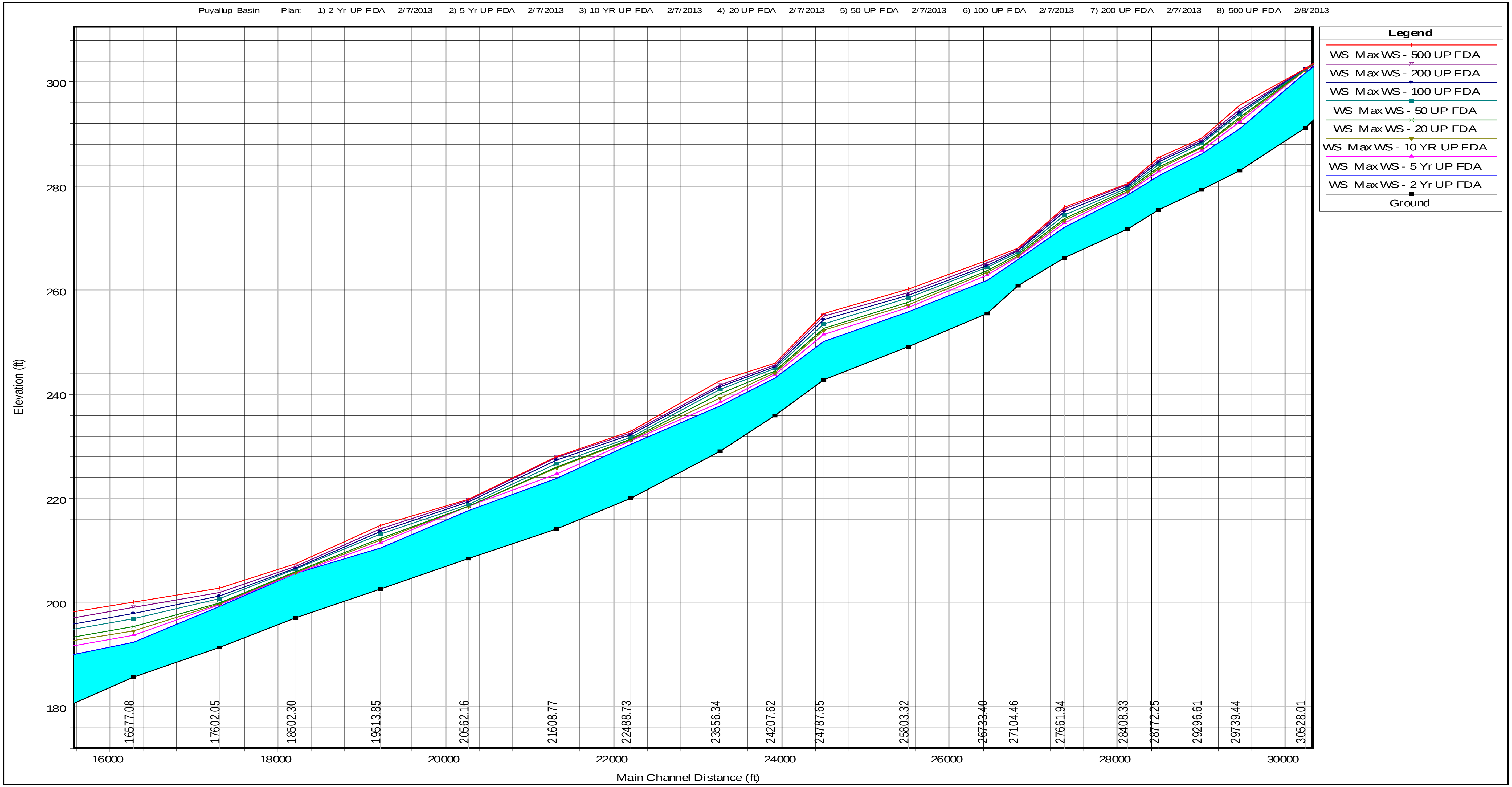


Figure 32: Carbon River Water Surface Profiles (con't)

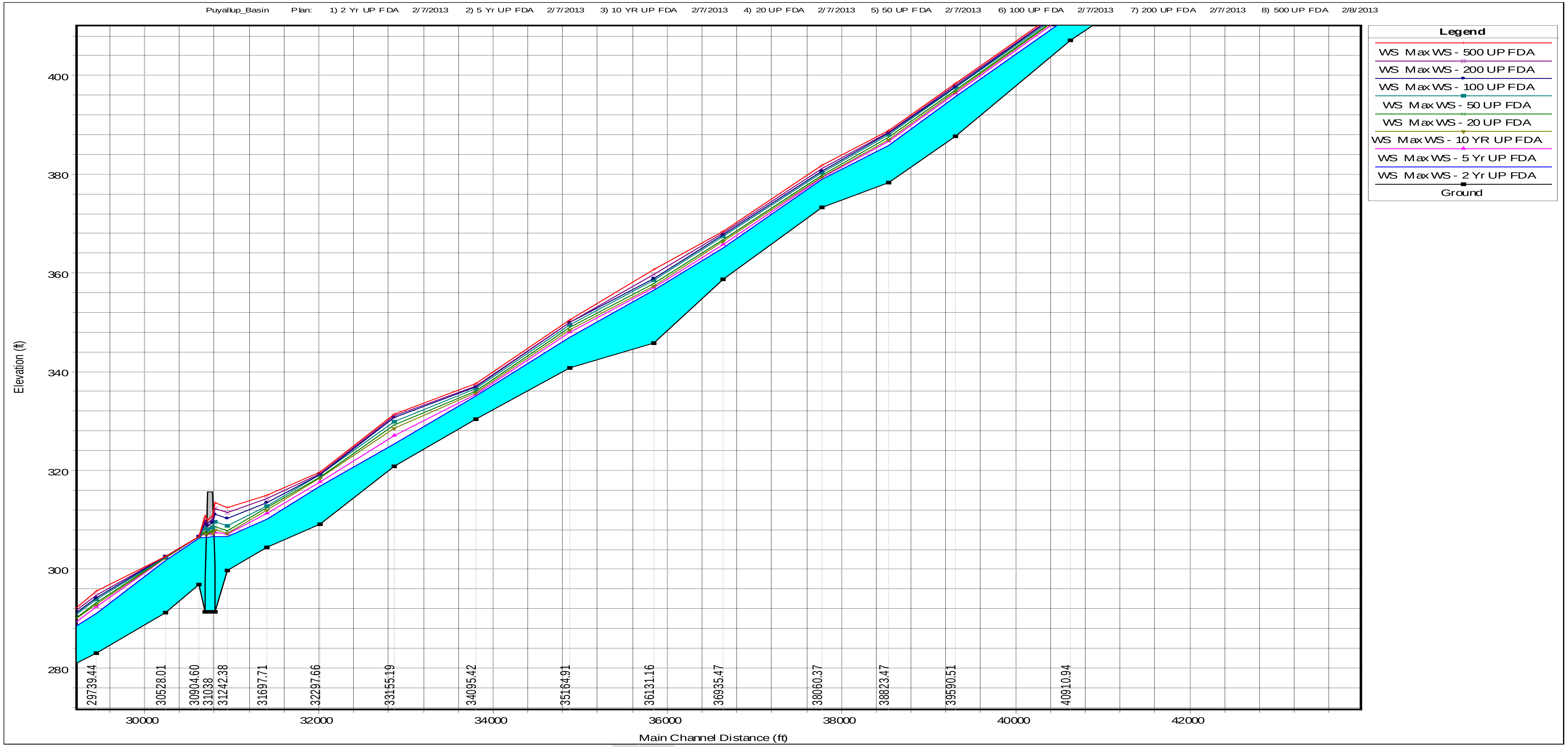


Figure 33: Carbon River Water Surface Profiles (con't)

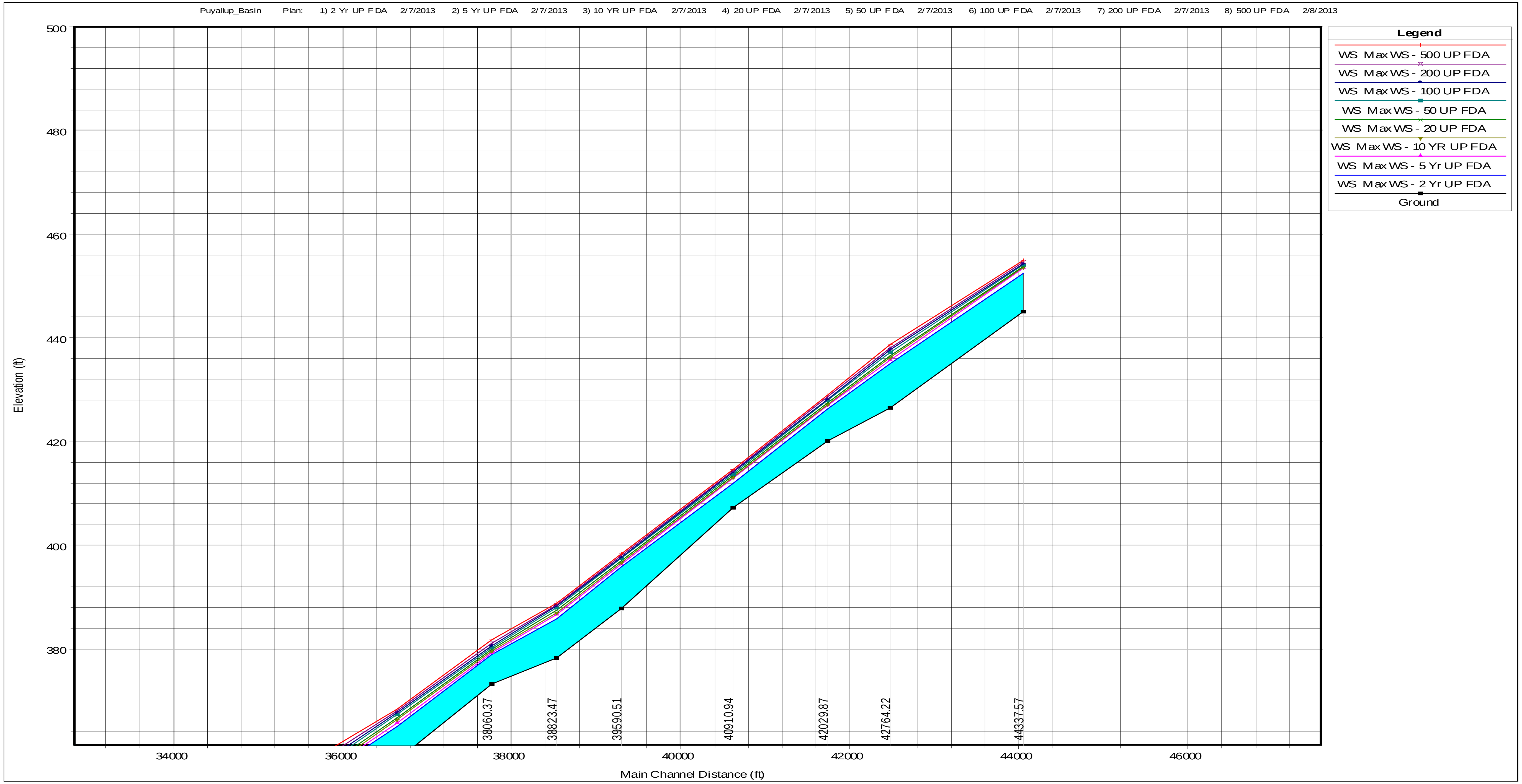


Figure 34: Carbon River Water Surface Profiles (con't)

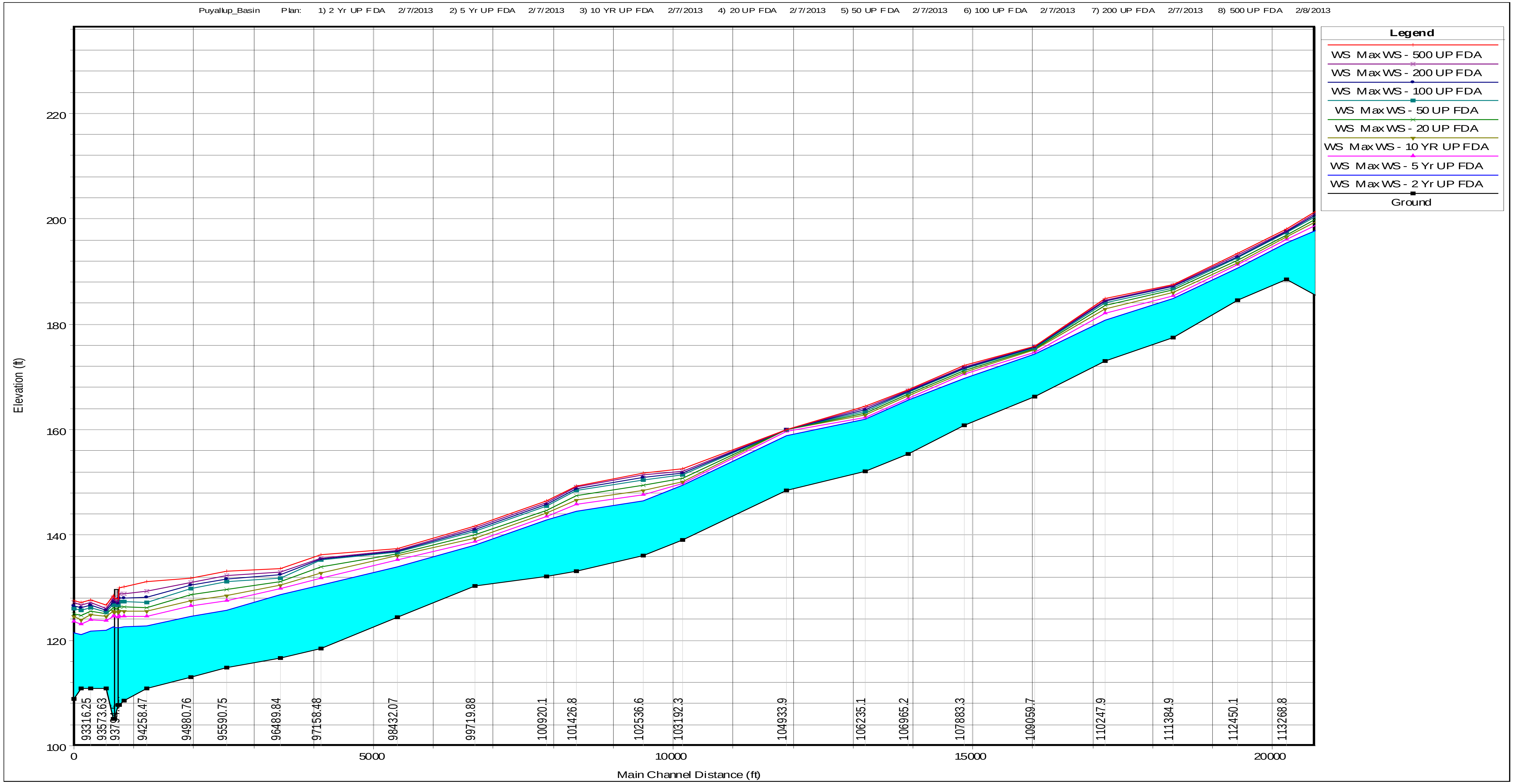


Figure 35: Upper Puyallup River Water Surface Profiles

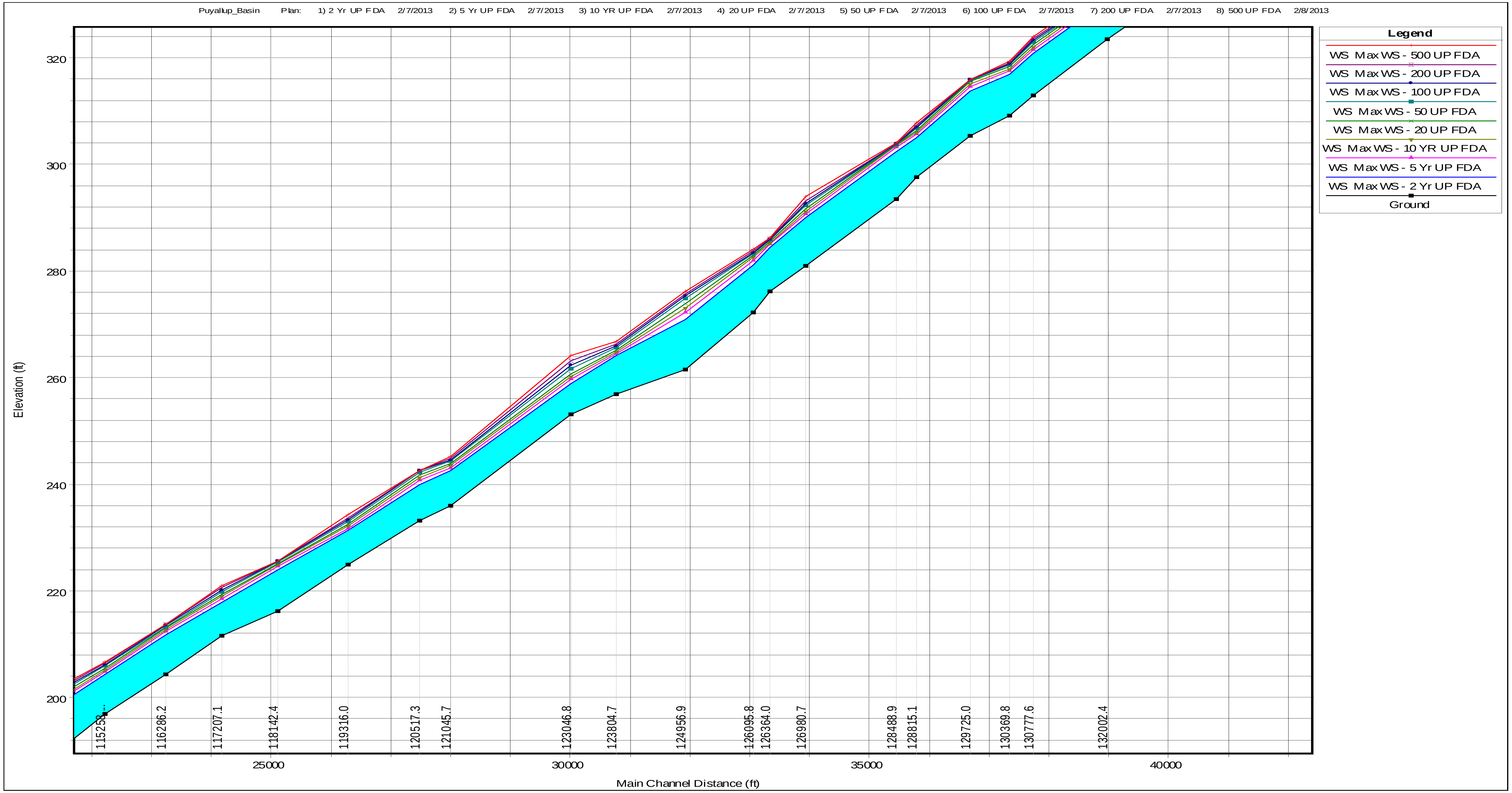


Figure 36: Upper Puyallup River Water Surface Profiles (con't)

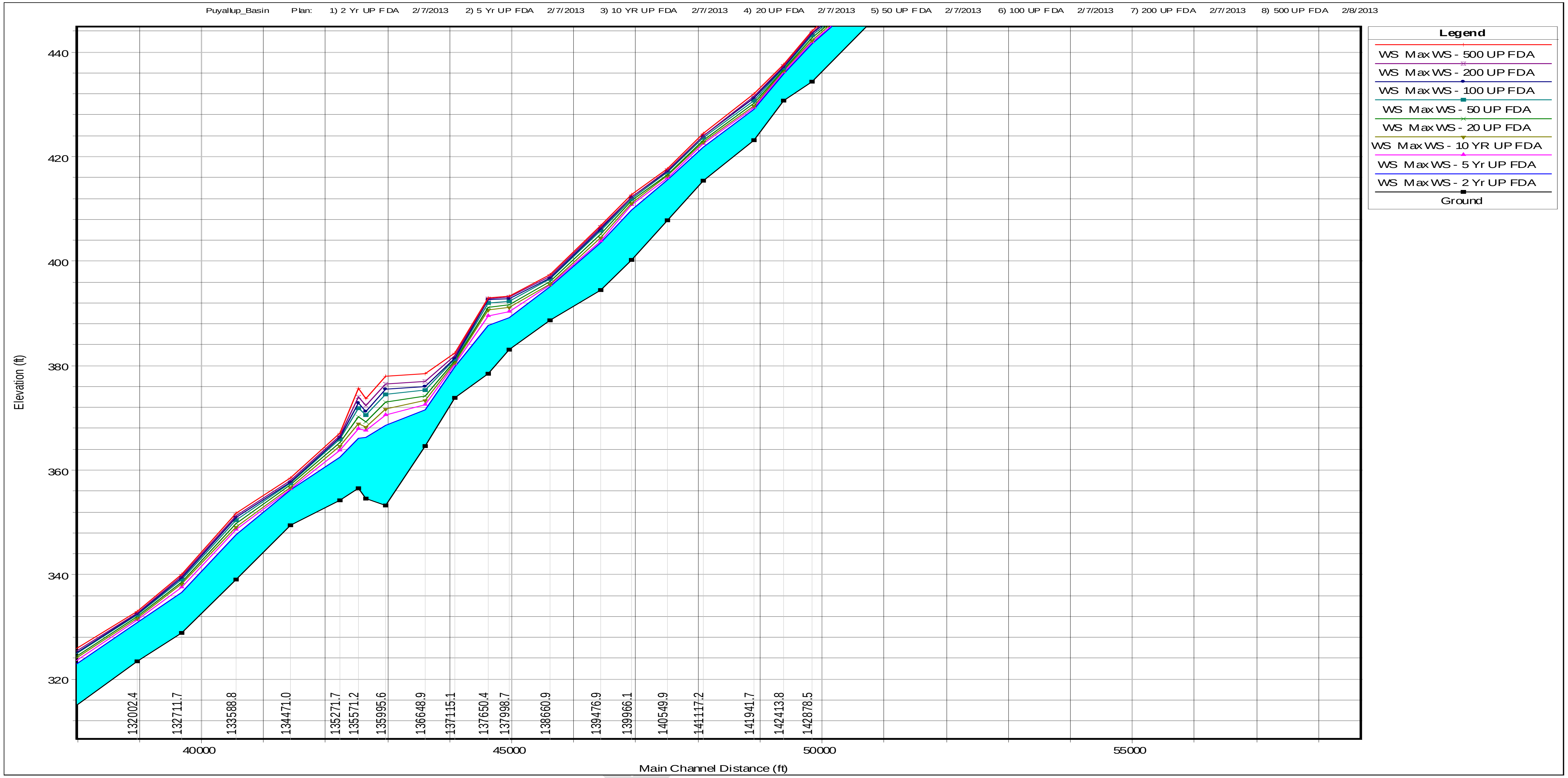


Figure 37: Upper Puyallup River Water Surface Profiles (con't)

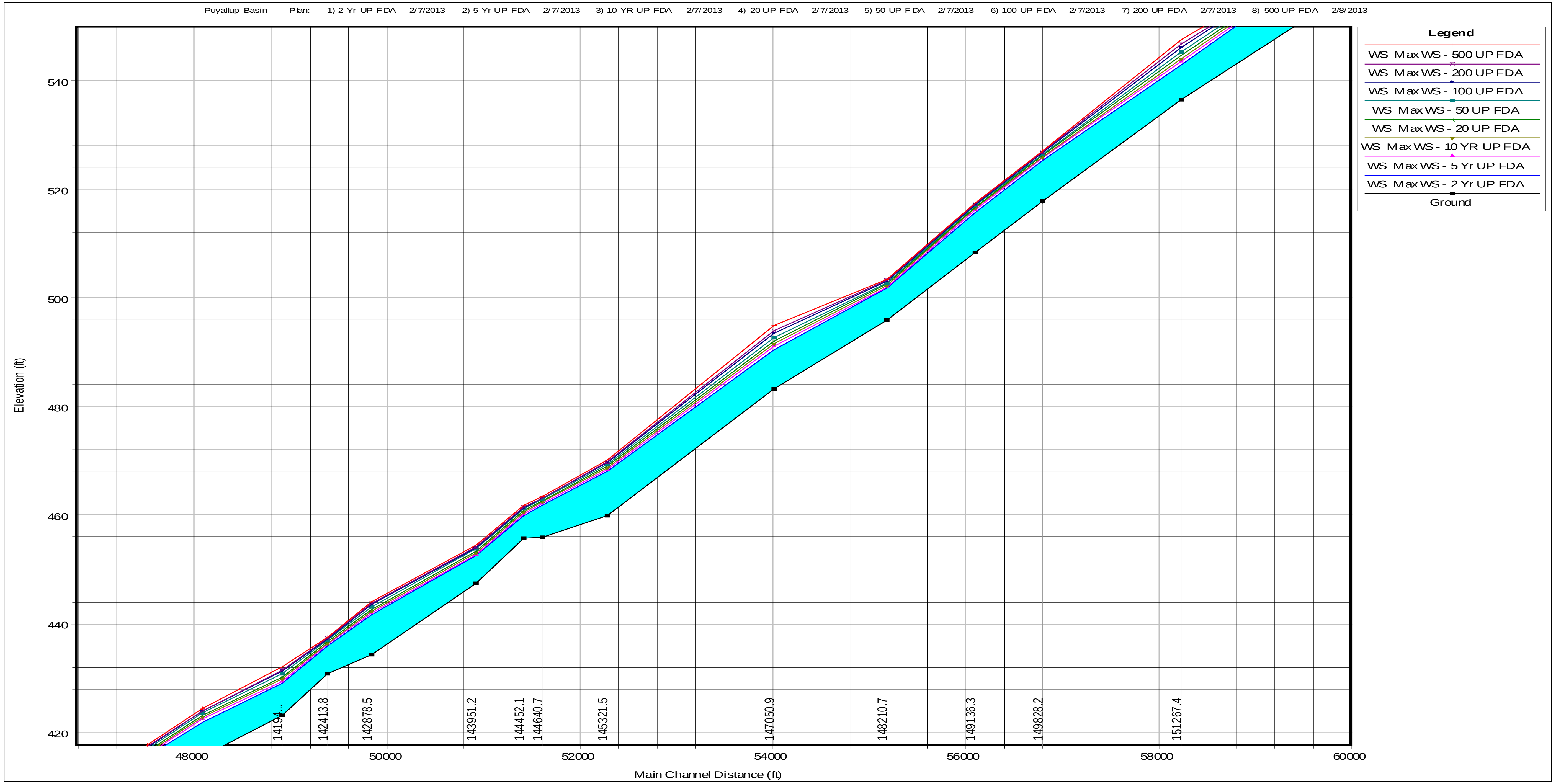


Figure 38: Upper Puyallup River Water Surface Profiles (cont)

6.1.2 White River

As discussed earlier the White River is challenging from a hydrological standpoint due to the influence of Mud Mountain Dam regulation. The upstream boundary hydrograph is the composite Mud Mountain outflow/ungaged local estimate (based on the balanced Boise Creek pattern hydrograph) between the dam and the upstream White River model boundary. The unregulated local inflow at the upstream model boundary (Lower White local on Figure 21) is based on the Newaukum Creek balanced hydrograph as a pattern. Newaukum Creek is located on the adjacent Green River however the pattern was selected (NHC, 2012). The coincident flow adjustment was developed from a relationship between the Newaukum Creek and an estimate of the Lower Puyallup River natural flow and the scaling of the Newaukum balanced hydrograph is based on the composite drainage area ratio/rainfall factor. These are both detailed in the NHC report (NHC, 2012).

Figure 39 through Figure 44 show the plots of the water surface profiles. The results of the model simulations for this reach yielded some interesting results due to Mud Mountain Dam flow regulation. Water surface profiles for the 5% AEP event were actually higher than for the2% AEP) event at locations upstream of the extreme backwater influence of the Puyallup River (RM 0-3 on the White River). This is due to the regulation of Mud Mountain Dam. For the 0.05 event, discharge from Mud Mountain can be maintained at 12,000 cfs while still keeping the Lower Puyallup below 50,000 cfs. This is reflected in higher flows on the White River.

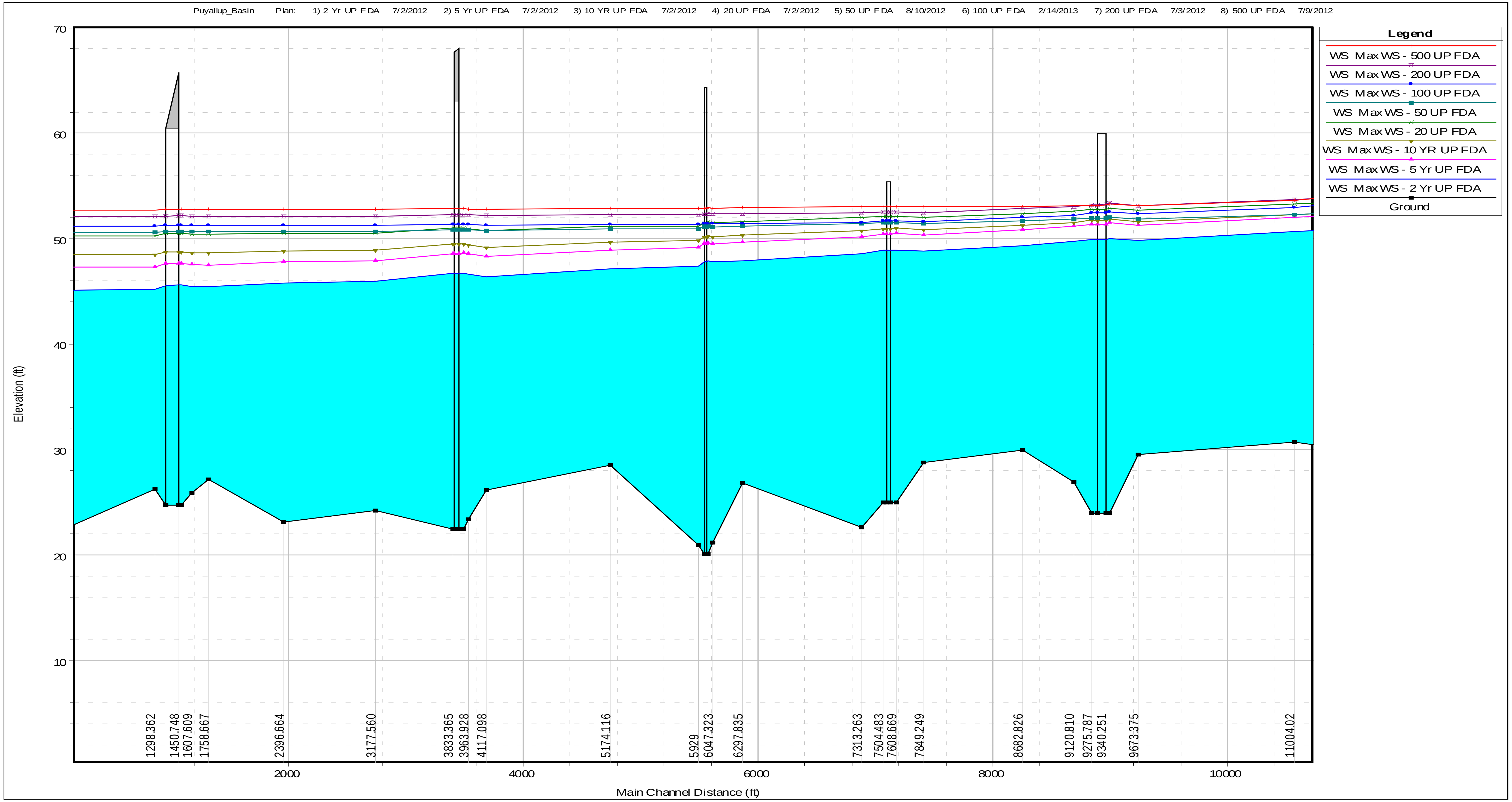


Figure 39: White River Water Surface Profiles

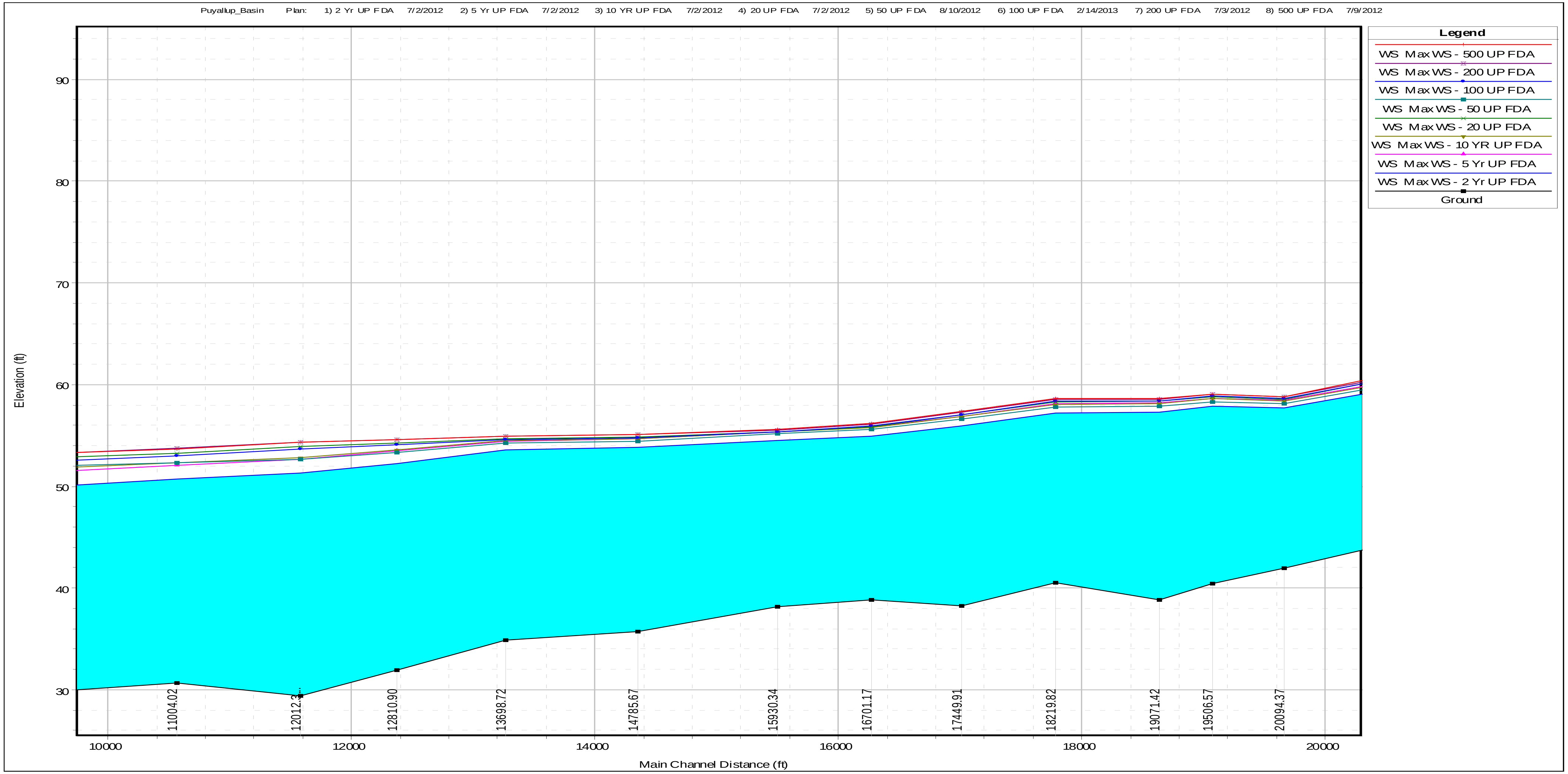


Figure 40: White River Water Surface Profiles (cont)

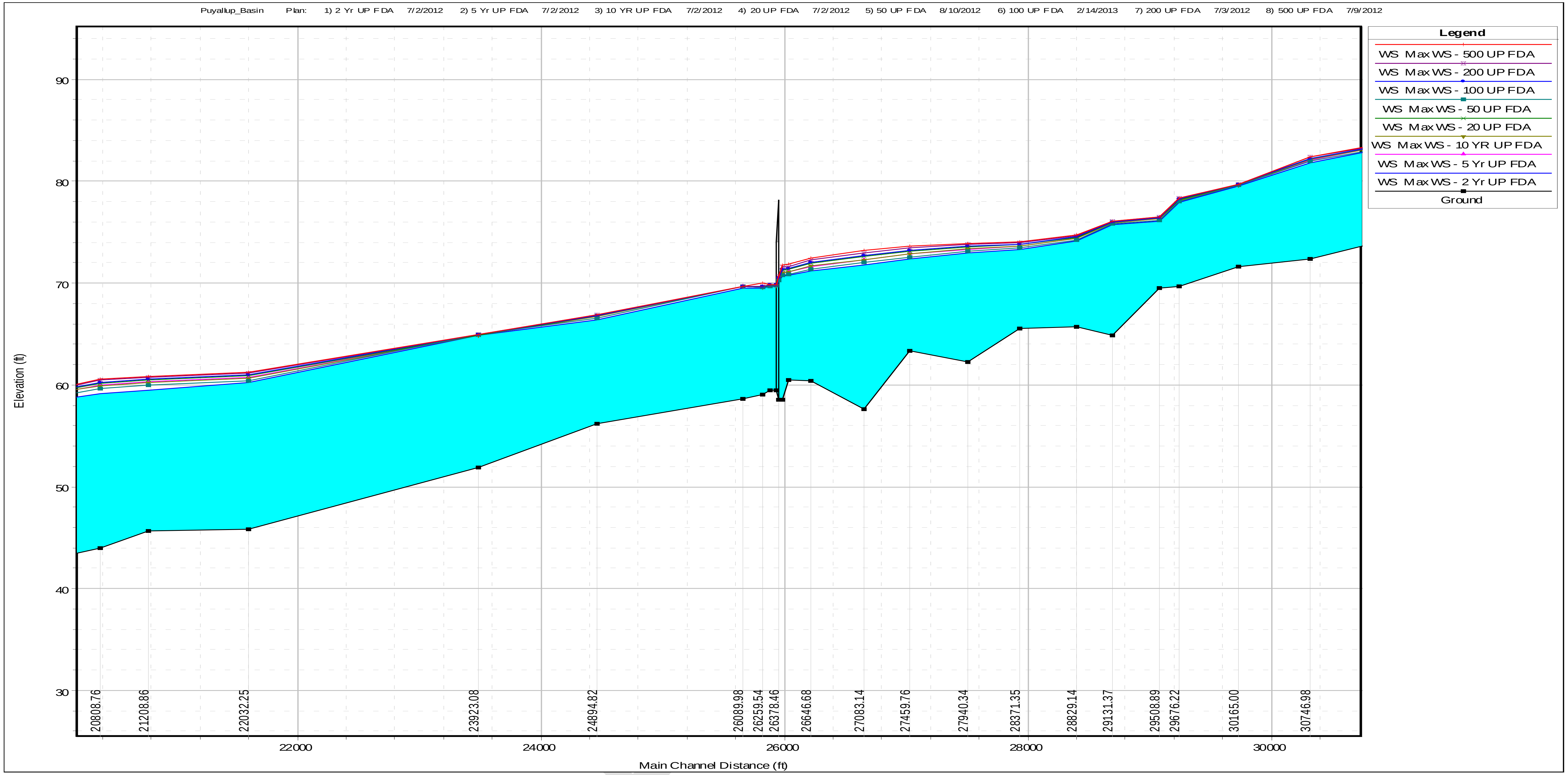


Figure 41: White River Water Surface Profiles (cont)

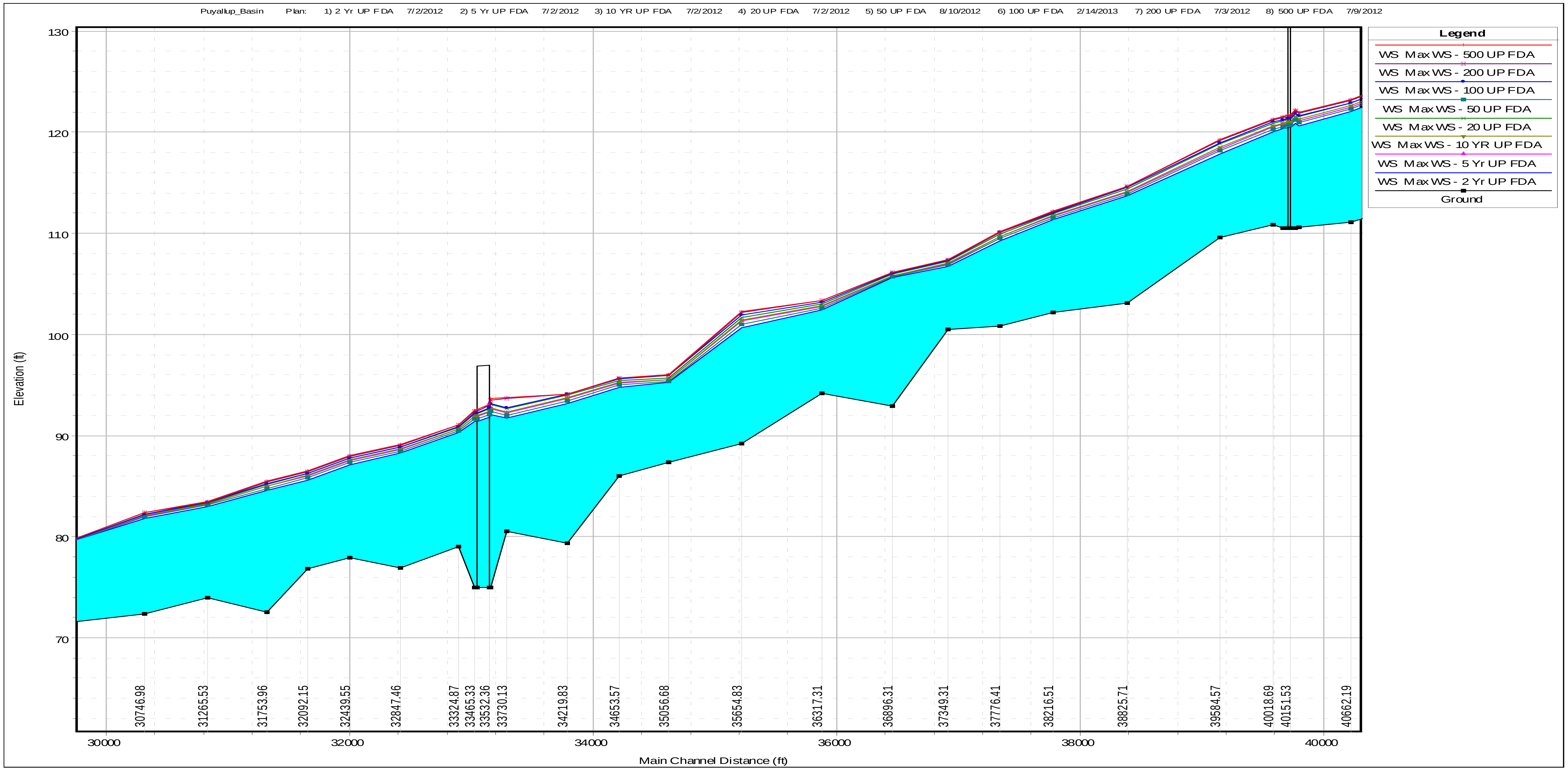


Figure 42: White River Water Surface Profiles (cont)

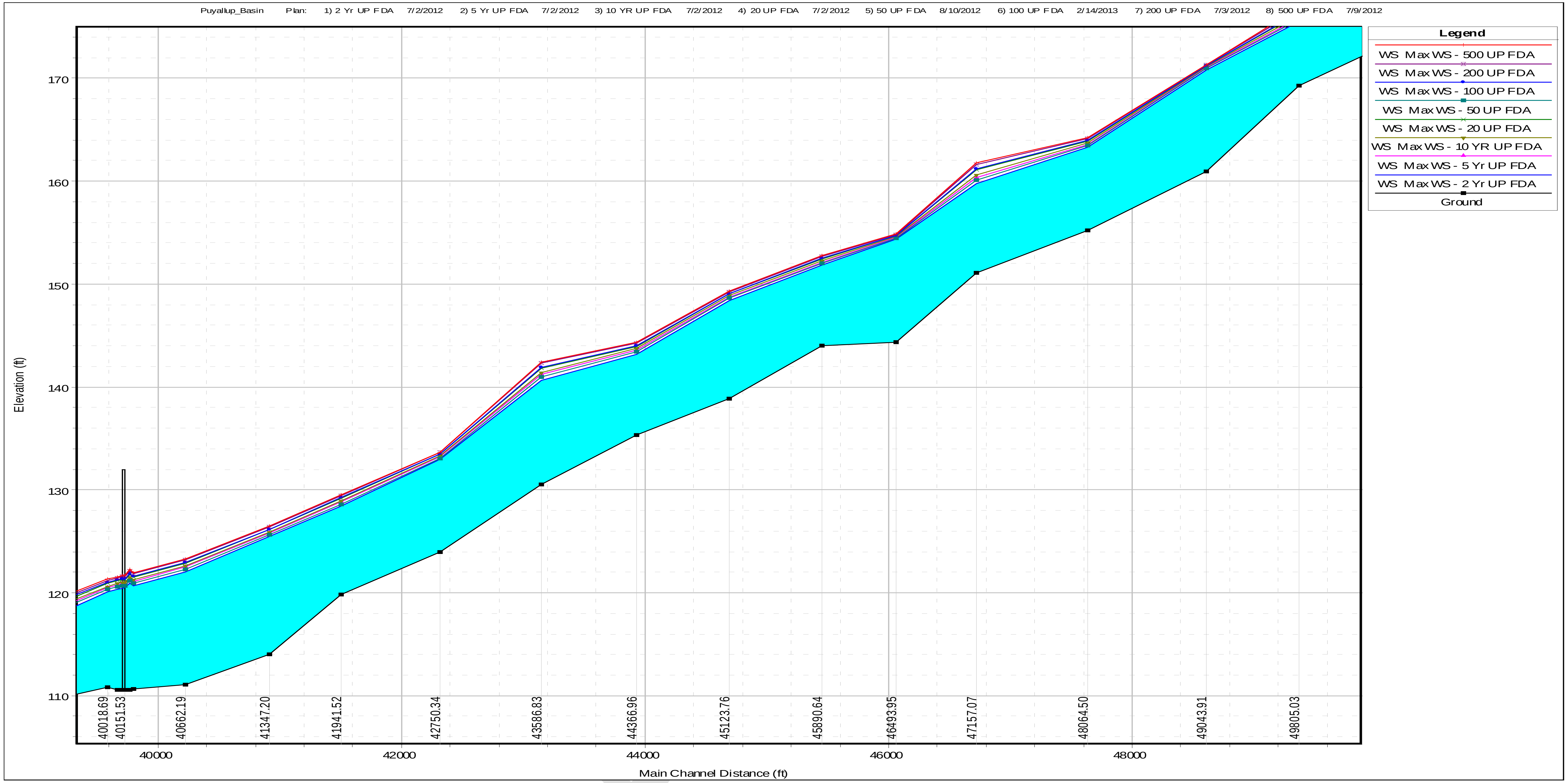


Figure 43: White River Water Surface Profiles (cont)

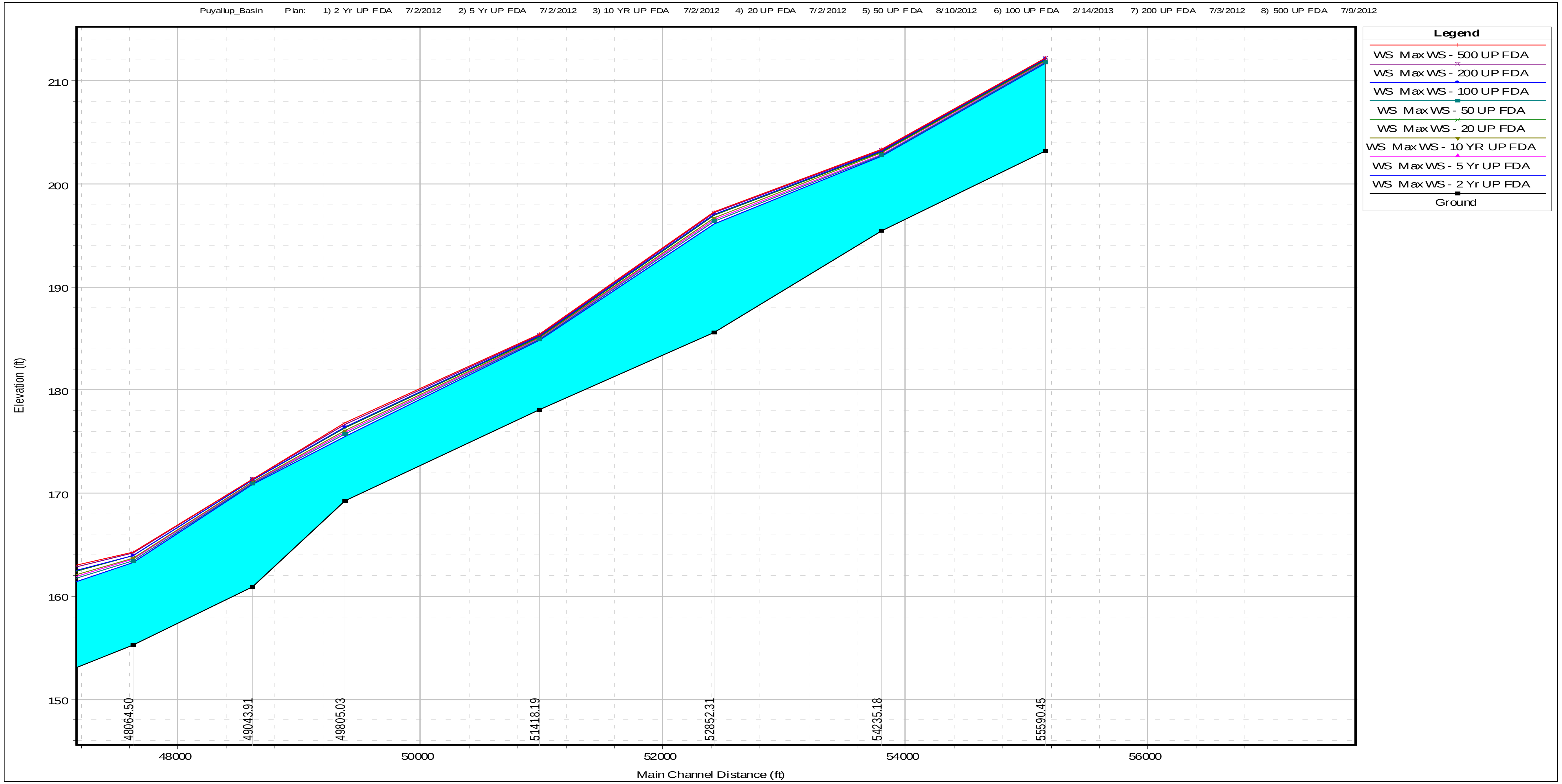


Figure 44: White River Water Surface Profiles (cont)

6.1.3 Lower Puyallup Reach

It was anticipated that each simulation for upstream return interval flood events would not necessarily produce the same return interval peak flow rate downstream in the Lower Puyallup reach. As such, each model run for the upper reaches was examined to see how well the simulated peak flow in the Lower reach matched up with the computed regulated peak flow frequency curve for the Lower Puyallup River (shown previously in Table 13). Where there was not a peak flow match for a return interval of interest, the three upstream boundary hydrographs were iteratively scaled until there was a match. In some cases, a simulation of one return interval for the upstream reaches turned out to be essentially the same peak flow as a different return interval for the Lower Puyallup reach. An example of this is the 0.005 Lower Puyallup flood event. The 0.002 event for the upstream reaches resulted in a peak flow value representative of a 0.005 event on the Lower Puyallup reach.

The Puyallup River at Puyallup streamgage is the location where flow is measured in this reach. This streamgage is located towards the middle of the reach. Based on the model simulations, at flows above about 50,000 cfs, water starts to leave the main river channel above the streamgage location. Given that the model is configured with a main channel and the flood plain represented as storage areas, the cross section at the streamgage does not necessarily see the peak flow of the simulation because some of the water is diverted to the storage areas. For large simulated events, the hydrograph at the cross section corresponding to the Puyallup at Puyallup streamgage is not representative of the inflow hydrograph to the Lower Puyallup reach. For the simulations intended to be representative of a certain return interval peak flow value, the comparison with the computed frequency curve (Table 13), was made at the upstream cross section of the Lower Puyallup reach. It should be noted that a model calibration event does not exist that is representative of frequency flows exceeding those of about a 1% AEP. These events are characterized by main channel flow attenuation due to overtopping of levees and other lateral structures and storage of flow in the floodplain (storage areas in the model).

6.1.4 Middle Puyallup Reach

This is a difficult reach to quantify flood flow statistics. There is not a long term, reliable streamgage in this reach from which to base a statistical analysis. For this effort the model simulations used for the Lower Puyallup reach were used for this reach as well. This approach is based on the limited incremental drainage area associated with this reach. This difference in drainage area on the Puyallup is 27 square miles with no major tributary. At the downstream end of this reach, in terms of drainage area, most of the unregulated portion of the basin is accounted for. Given that the regulated frequency curve for the Lower Puyallup is based on the unregulated portion of the basin, it is assumed that the flood statistics on the Middle Puyallup are very similar to those on Lower Puyallup from flood event to flood event.

6.2 Sensitivity

Quantifying the sensitivity of model results to variation of parameters discussed here gave a sense of how the model responds to changes, and ultimately helped to inform uncertainty used in the FDA model. Uncertainty in the FDA analysis is discussed in the FDA support appendix. The main adjustable coefficients in the hydraulic model are Manning's n values and the lateral weir coefficients. These values are initially set with handbook values based on experience with systems of similar characteristics. These values are usually given as ranges for certain conditions. Ideally these initial values can be further refined (calibrated) to a specific situation based on flow and stage information from observed flood events. For calibration of Manning's n there was some information available that allowed for some adjustment. The streamgages on the Puyallup at Puyallup and the Puyallup near Orting were primarily used for this. However for the case of the lateral weir coefficient there really was no information available to allow for fine tuning.

6.2.1 Lateral Weir Coefficient

The sensitivity of model results with respect to the lateral weir coefficient (i.e. the weir coefficient used for lateral structures) was examined using the 0.5% AEP flood event simulation. This event was selected because the base simulation is characterized by overtopping of levees and other structures adjacent to the river. The baseline weir coefficients used in the model are 2.0. For the purposes of sensitivity analysis the 0.%5 AEP simulation was re-run once with the values for the Lower Puyallup reach set at 1.0 and again with the values set at 3.0. The assumption here is that these values capture the range of possible values.

At a given location for the same upstream river water surface elevation, as expected, the simulation with the higher coefficient resulted in more flow over the lateral structure and, as a result, a higher water surface elevation in the adjacent storage area. The simulation with the lower coefficient produced the opposite. However, when looked at as a system, the simulation with the higher coefficient actually resulted in many storage areas having a lower peak water surface elevation than occurred with the base or lower coefficient.

This occurs because for the case with the higher coefficient, more flow is removed from the reach at the upstream end than with the low coefficient case, leaving less flow in the river. This lower flow results in lower downstream stages and less head (or elevations below the lateral structure crest) to drive lateral flow into storage areas. The lower available head is enough in some cases to result in lower computed lateral discharge despite the higher weir coefficient. The results of the sensitivity analysis show the opposite in many cases occurring when the lateral weir coefficient is lowered. The lateral weir coefficient values are set globally at 2.0. In reality this value can vary depending upon condition of the levee crest and vegetation. Most levees in the Puyallup system are eligible for our PL84-99 program and are surfaced with gravel roads with dense brush and no trees, as well as sloping sides. Post TSP modeling refinements will include more accurate determination of this coefficient however it may still be a global estimate. A sensitivity analysis was not performed on the weir coefficients for the storage area to storage area connections. These values were globally set to 0.5 in the model based upon discussion with HEC.

6.2.2 Hydraulic Roughness Coefficients (Manning's n)

Hydraulic roughness coefficients, (Manning's n values), are used to parameterize the resistance to flows across the channel bed and floodplain. For this study Manning's n values were first determined based on literature (example: Arcement, et al, 1989, Chow, 1959) and fine-tuned where possible using streamgage data and high water marks. Analyses of available streamgage data suggests (as does literature, Chow 1959) that Manning's n can vary by flow/stage. Based on analyses of the Puyallup at Puyallup and Puyallup near Orting streamgages, a set of factors (flow roughness factors in HEC-RAS) to adjust the channel n values were estimated. The results of the Puyallup at Puyallup analysis were applied to the Lower Puyallup reach and the results of the Puyallup near Orting analysis were applied to the other reaches. The thought process here is that the Upper Puyallup, Middle Puyallup, White and Carbon reaches are of similar physical character and thus the factors from the Orting location were transferable to the other similar reaches. The factors vary the base n values by no more than 25%. This variation is at the low end of the range given in the recommended guidance EM 1110-2-1619 to vary roughness by 1-2 standard deviations. Variation outside of 1 standard deviation often gave unrealistic roughness values.

To test the sensitivity of the model results to variation in Manning's n the base values used in the model were adjusted up by 25% and down by 10% and 25%. The minus 25% adjustment produced some unrealistically low values, particularly in the Lower Puyallup reach. Hydrographs from the 0.5% AEP event-based simulation were used in this test.

- White River: Based on the cross sections compared the plus 25% n water surface generally ranged about 0.75 ft. higher while the minus 25% water surface generally was approximately one-foot lower when compared to the base n value water surface. The minus 10% water surface was generally 0.5-foot lower than the base.

- Carbon River: Based on the cross sections compared the plus 25% n water surface generally ranged about 1.0 ft. higher while the minus 25% water surface generally was approximately 2 ft. lower when compared to the base n value water surface. The minus 10% water surface was generally 0.5 ft. lower than the base.

- Middle and Upper Puyallup: Based on the cross sections compared the plus 25% n water surface generally ranged about 0.75 ft. higher while the minus 25% water surface generally was approximately 2 ft. lower when compared to the base n value water surface. The minus 10% water surface was generally 0.5 ft. lower than the base.

- Lower Puyallup: Based on the cross sections compared the plus 25% n water surface generally ranged about 1.5 ft. higher while the minus 25% water surface generally was approximately 3 to 5 ft. lower when compared to the base n value water surface. The minus 10% water surface was generally 1.5 ft. lower than the base.

In reality reducing the base Manning's n values by 25% probably results in values which are lower than those which are defensible. For the Lower Puyallup the base values for the main channel are set at 0.028. A 25% reduction would reduce this to 0.021. USGS publication WSP 2339 (Acerment, 1989) lists a range of 0.025 to 0.032 for a stable channel with a bed material of firm soil. The given range for a channel of coarse sand is 0.026 to 0.035. For channel types similar to other reaches in the study, WSP 2339 gives wider ranges.

In an unsteady flow model, changing Manning's n values can impact the attenuation of the flood wave, resulting in different peak flow values being computed. For this study, the upper and Middle Puyallup reaches seem to be most sensitive to this. The water surface elevation ranges given in the preceding paragraph are reflective in part on the attenuation impact a different Manning's n value has when compared to the base Manning's n value.

6.3 Model Results Discussion

Given the large basin area, which spans from Mt Rainier to Puget Sound, the hydrology and hydraulics of the study area are complex. Runoff varies spatially from flood event to flood event. The goal of the hydraulic model simulations is to produce water surface profiles and flood plain inundation elevations and extents which are representative of the average or most-likely flooding scenario on each of the five reaches for each return interval (statistical flood event) of interest. This process started with developing hydrology that reflects the most-likely or average runoff scenario at the study boundaries and within the study area. The hydrology is then routed through the hydraulic model to obtain hydraulic conditions which are representative of given return intervals. This section covers some of the limitations and items of note as a result of this process.

6.3.1 Hydrology

Significant time and effort was spent developing hydrologic inputs to the hydraulic model which represent the average or most-likely flood runoff scenario mentioned above. Factors considered were: return interval of locations where a long-record streamgage exists, coincident flow relationships between the main reaches and tributaries and hydrograph timing. The one location where these inputs, once routed, can be checked to some degree is on the Upper Puyallup reach. Here, mid-reach, is a long-record streamgage where flood flow statistics were computed. As discussed earlier, hydrograph adjustments were required in order for the simulated flow for a given return interval to reasonably match that computed from the frequency curve. The adjustment was made by iteratively scaling up the original upstream boundary hydrograph until the simulated flow values matched the peak flow value from the frequency curve. There are other reaches, such as the Carbon River, where there was not the benefit of a mid-reach streamgage. It is unknown if the adjustment required on the Upper Puyallup applies to other reaches in the study area.

The need to the hydrology adjustments is in part a result of the wide variability in spatial runoff this basin experiences from flood event to flood event. It is possible, however, that part of the reason hydrology adjustments were required is because of hydraulic model limitations and/or shortcomings. For instance, between the upstream boundary and the Orting streamgage, it is possible that the representation of the floodplain in the model as storage areas instead of conveyance attenuates the flow more than actually occurs in the physical system. While this type of disconnect between the model and physical system may be a contributor, it is assumed to be a small one given that the latest detailed terrain data was used. It is assumed that the large ungaged basin area, and high variability in the coincident flow analysis from location to location points to the hydrology itself is a more likely cause.

6.3.2 Mud Mountain Dam

As discussed earlier, the Mud Mountain Dam component of the White River upstream boundary condition input hydrograph is based on a “by the book” on the Water Control Plan. From a practical standpoint real time execution of this plan involves team decision making efforts that take into account imperfect meteorological and hydrological forecasts, flood fighting activities and event-specific constraints that arise. These factors can make the actual observed event operation look different than the idealized ‘by the book’ operation.

The modeling inputs assume there are no constraints with the physical operation of Mud Mountain Dam. It is assumed gates operate as designed without issue, there are no structural failures and there are no unforeseen issues that impact the operation.

6.3.3 Hydraulics

The hydraulic model was calibrated to the extent possible using streamgage rating curves and highwater marks where available. The goal of model calibration is to provide a sense of how well the model will perform as a tool to predict hydraulic conditions that would occur for various flood events both observed and hypothetical. Ideally, model calibration initially gives the modeler an indication as to the adequacy of the modeling system—model type, terrain representation, cross section data, bridge information, etc. Using handbook coefficients (lateral weir, Manning's n), as well as some observed water surface elevation information, the modeler can determine how well suited the model is to a particular situation. Beyond this, observed data can also allow the model to be fine-tuned via the adjustment of the original handbook coefficients so that the model best reproduces observed conditions.

The greatest uncertainty in the model results based on model parameters may be in the depths and elevations computed in the floodplain. In the case of this study, the observed events available for model calibration/verification are not necessarily representative of the type of floods that are of interest. For example, all past observed flood events have essentially been contained by the levees along the Lower Puyallup. The model does a good job of reproducing these in-channel events along this reach. However, of interest are more extreme events that overtop the levees and flood the outlying floodplain area, for which there has not been an observed event for comparison. Factors specific to the model that could influence the computation of floodplain water surface elevations include lateral weir coefficients, the modeler's ability to delineate the storage area boundary properly, uncertainty of existence, elevation and type of floodplain conveyance structures and resolution of topographic mapping.

Debris is not accounted for in the model beyond the impact it may have on the observed event data for calibration. Debris accumulating on bridges for instance, could have an impact on computed water surface profiles.

Overtopping of structures adjacent to the river assumes no failure or degradation of the structure during the overtopping. In reality, structures may fail or degrade during an overtopping event changing the flow characteristics of the structure and the amount of water seen on the landward side of the structure.

The model results are based on an assumption of stable channel bed and banks during floods. In reality, during a large flood event, especially the steeper reaches (Carbon, Upper White, Upper Puyallup) the river could behave dynamically as sediment moves through and banks and gravel bars erode. These dynamic conditions influence the channel shape (morphology), flood capacity, and resulting flood elevations. Without running mobile bed hydraulic models, the best that can be done with this approach is to repeatedly verify the adequacy of the models using observed high water marks in areas of interest. If large discrepancies are discovered, new channel survey data can be acquired to update and recalibrate the model. Even with a mobile bed model it is difficult to capture the uncertainty and spatial variability of sediment loads.

6.4 Next Steps

The purpose of this effort is to provide the H&H inputs to a risk-based analysis that uses an HEC-FDA model that combines economic information to quantify expected annual damages as well as computed flood risk associated with existing structures. Inputs from the statistical flood event simulations include the annual peak frequency curve information by reach, water surface profiles, routed peak flow rates within each reach, lateral flow over structures and peak floodplain water surface elevations. Key to this analysis is quantifying the uncertainties of the inputs. The items from the discussion above incorporated these uncertainties. Support of the FDA analysis is described in the H&H Support of FDA Analysis Appendix.

The modeling effort documented thus far was intended to characterize the existing, without project conditions. Sediment deposition is an important component of future conditions characterization. Future without project conditions as well as future with-project conditions are characterized and sediment analysis is described in the H&H Hydraulic Modeling of Future Conditions Appendix. Model cross sections were adjusted to reflect estimated future sediment deposition within the study area. It should be noted that sediment modeling is planned for post-TSP to inform final feasibility level designs. Areas where the channel has changed significantly in recent years (in width, slope, etc.) may experience future deposition that differs from what has been observed historically. Sediment modeling is likely necessary to quantify such changes accurately. However, it is not expected that results of sediment modeling will significantly alter the TSP. Future changes to Commencement Bay elevations due to predicted sea-level rise were also accounted for. To facilitate alternatives analyses, model geometry (existing and future) was modified to reflect features of alternatives such as levees. This is also described in the H&H Hydraulic Modeling of Future Conditions Appendix.

All model runs were used for FDA support, however only combinations of conditions/events were selected by the PDT for inclusion in flood map products. The term “existing condition” used in this report is synonymous with the term “base condition” used in economic evaluation and refers to the point in time when the project starts accruing benefits (or year zero). Conditions include base with- and without project, and future with- and without project, and 50% through 0.2% AEP events. A total of 64 reach level maps will be included for the final feasibility study and documented in the H&H Mapping of Hydraulic Model Runs Appendix.

7.0 Existing Condition Flood Extents

The effort to capture existing flood conditions described in this report was intended to inform the alternative development process and provide data for further economic evaluation of measures. The use of flood extent estimations included locating flood risk reduction measures, informing discussions with local sponsors, and determining which areas may need further analysis. Additional flood maps are included as a map appendix to the Feasibility Report. Water surface profiles from the model were compared to levee profiles to determine incipient overtopping events for existing conditions. Shown in Table 19 is the first location of overtopping for each levee in the system (or the bank if there is no levee) and the corresponding event frequency.

Table 19: Levee overtopping

Reach	bank	XS	RM	levee	incipient overtopping event (% AEP)
Carbon	LB	31242	6	Alward 2	10
Carbon	LB	26253	5	Guy West	0.2++
Carbon	LB	16577	3.1	Bridge St	0.5
Carbon	LB	15144	2.8	Orting treatment plant	1
Carbon	LB	3530	0.6	Riddell	2
Carbon	RB	631	0.1	Lindsay	1
Carbon	RB	34095	6.5	Water ski	5
Upper Puyallup	LB	123046	23	McAbee	0.2+
Upper Puyallup	LB	117207	21.9	Old Soldiers Home	0.2+
Upper Puyallup	LB	105604	19.7	Leach Rd	5
Upper Puyallup	RB	122018	22.8	Ford	1
Upper Puyallup	RB	119316	22.3	Jones	5
Upper Puyallup	RB	106965	19.9	Calistoga	0.2+
Upper Puyallup	RB	96489	18	High Cedars	5
Upper Puyallup	LB	93316	17.5	Bower/Parker	5
Middle Puyallup	LB	91508	17.1	Bower/Parker	10
Middle Puyallup	LB	85543	16	McMillin	10
Middle Puyallup	LB	73625	13.8	Sportsman	5
Middle Puyallup	LB	71771	13.5	Bowman/ Hilton	2
Middle Puyallup	RB	91508	17.1	Lindsay	5
Middle Puyallup	RB	66487	12.5	Riverside	2
Middle Puyallup	RB	59586	11.2	River Grove	20
Lower Puyallup	RB	42525.43	8	North Levee Rd	2
Lower Puyallup	RB	10752.54	2	Puyallup Right Bank	0.2
Lower Puyallup	LB	23804.34	4.5	River Rd	2
Lower Puyallup	LB	9944.706	1.9	Puyallup Left Bank	0.5

White	RB	29676.23	5.6	none (Pacific area)	50
White	LB	11004.02	2	none	0.2+

Several bridges in the project area are impacted by existing conditions flows. Bridge decks that show flow impingement in the model are shown in Table 20. Several of the listed bridges are very close to impingement and may require further evaluation during later phases of future conditions modeling. Impacted sites may require either raising the bridge or dredging the bridge opening.

Table 20: Bridges impinged upon by existing conditions flows

Bridge	Location	Existing conditions
142nd Ave Bridge RM 1.4	White	X*
SR 162/ Railroad Bridge RM 17.7	Upper Puyallup	X*
SR 162/ foot bridge RM 5.8	Carbon	X

* Flow impingement is critically close but does not contact the bridge deck in the modeling (within 1 ft.)

Preliminary existing modeled basin flood extents are shown in the figures below for a 1% AEP event. An event of 1% AEP has not occurred throughout the basin in recent times. As mentioned previously under discussion of model calibration, flood extents from the recent 2006-2009 modeling period were compared to available flood photos and the model refined to reproduce observed flooding. Flooded areas match up well with descriptions given in the Pierce County Flood Hazard Management Plan.

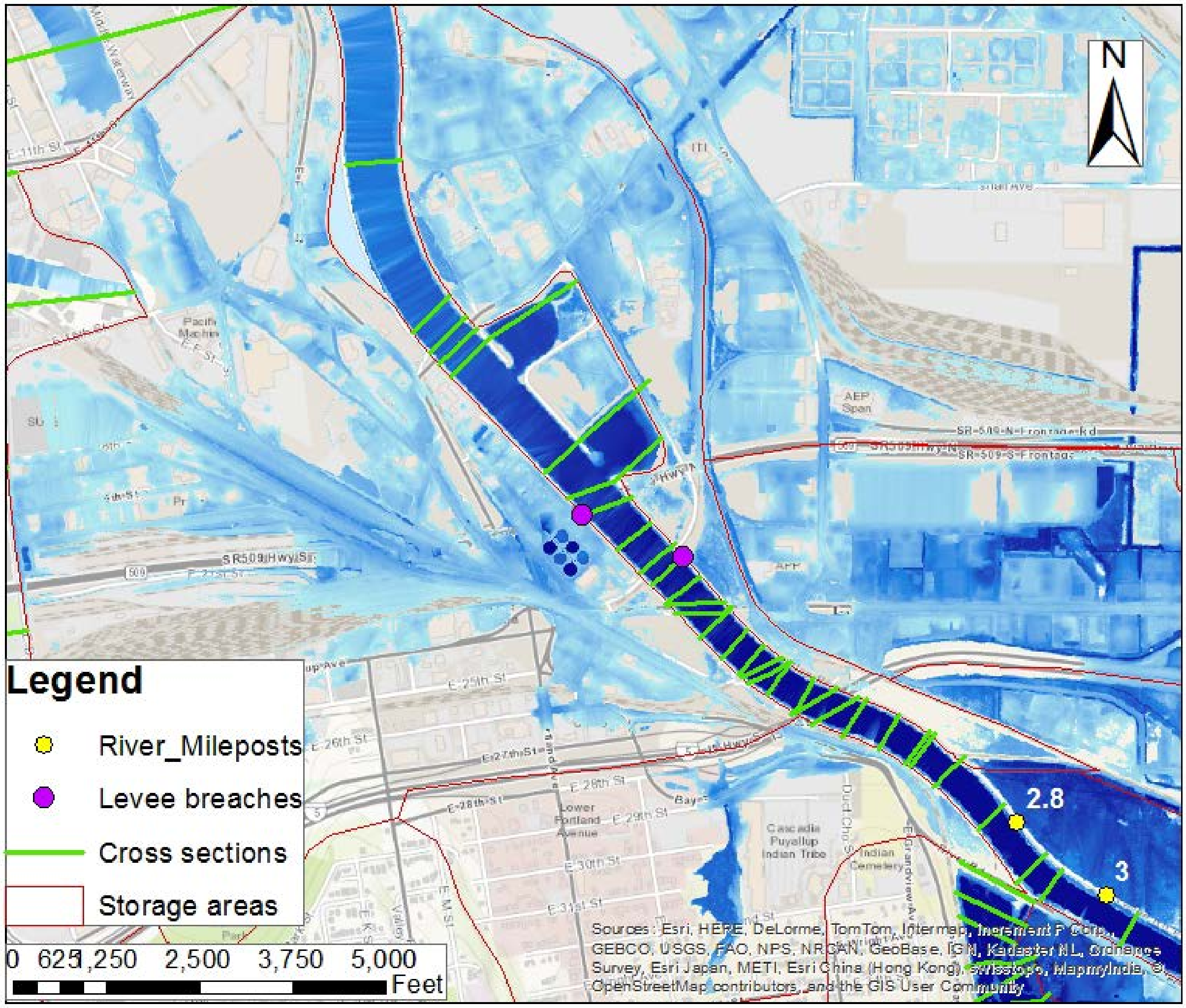


Figure 45: Lower Puyallup River below RM 3

The Lower Puyallup River below RM 3 experiences tidal influence along the left and right bank federal levees. This area has 2-3 ft. of residual levee height for a 1% AEP event. Levee breaches were simulated at two locations.

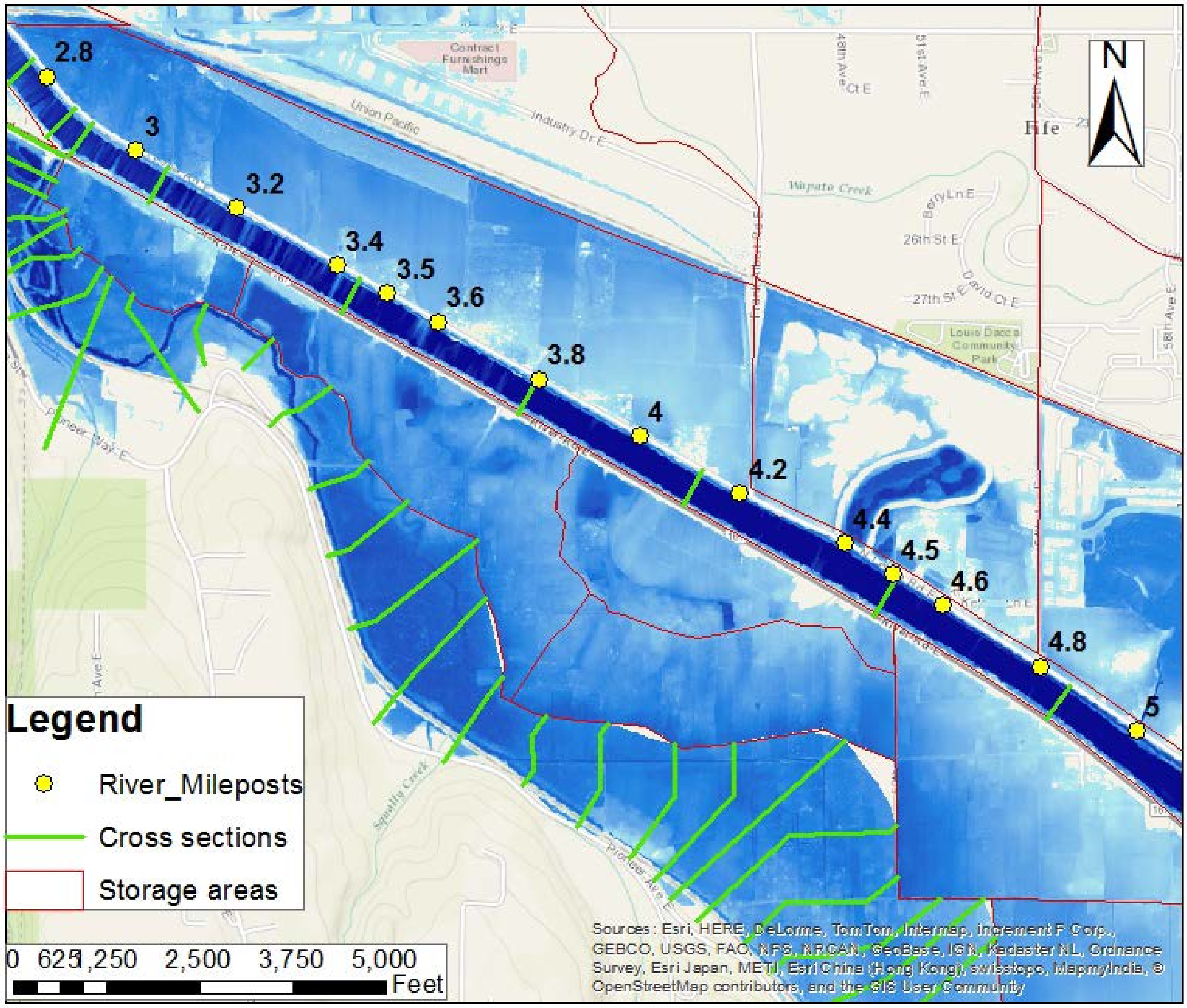


Figure 46: Lower Puyallup River from RM 2.8-5 including Clear Creek

The Clear Creek outlet on the left bank of the Lower Puyallup at RM 2.9 is gated and closed to prevent backwater. Interior flows are impounded at the outlet during flood events until river flows drop and the gates can be opened to release stored water. Impounded flood water has caused significant inundation of developed areas during the last few large flood events. The leveed area behind North Levee Rd. on the right bank and River Rd. on the left bank are overtopped in areas by a 1% AEP event.

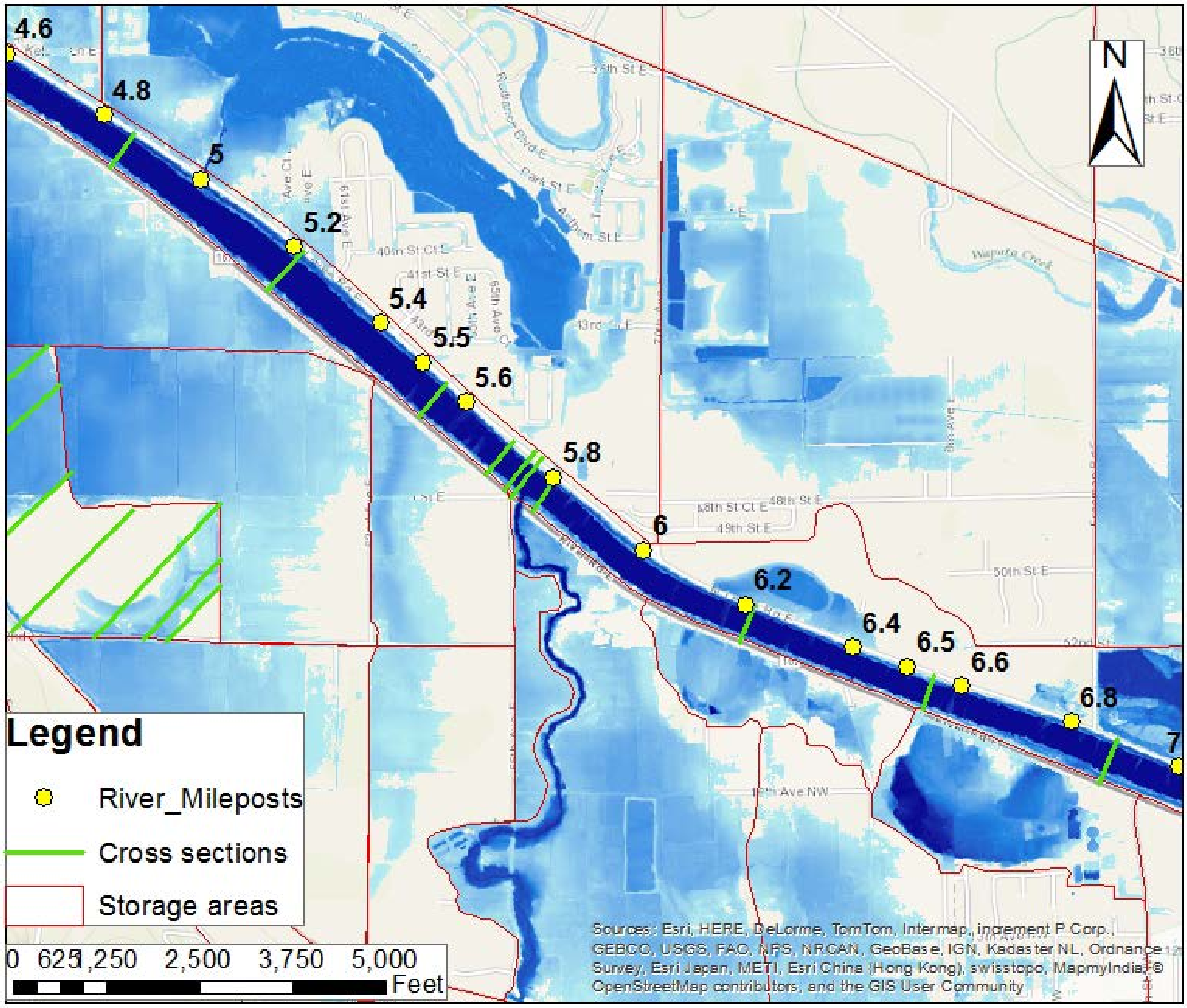


Figure 47: Lower Puyallup River from RM 4.6-7 including Clarks Creek

The outlet of Clarks Creek is free-flowing through the leveed left bank of the Lower Puyallup at RM 5.8. The creek backwaters from a 1% AEP event and flooding is caused by interior flows. The leveed area behind North Levee Rd. on the right bank and River Rd. on the left bank are overtopped in areas by a 1% AEP event.

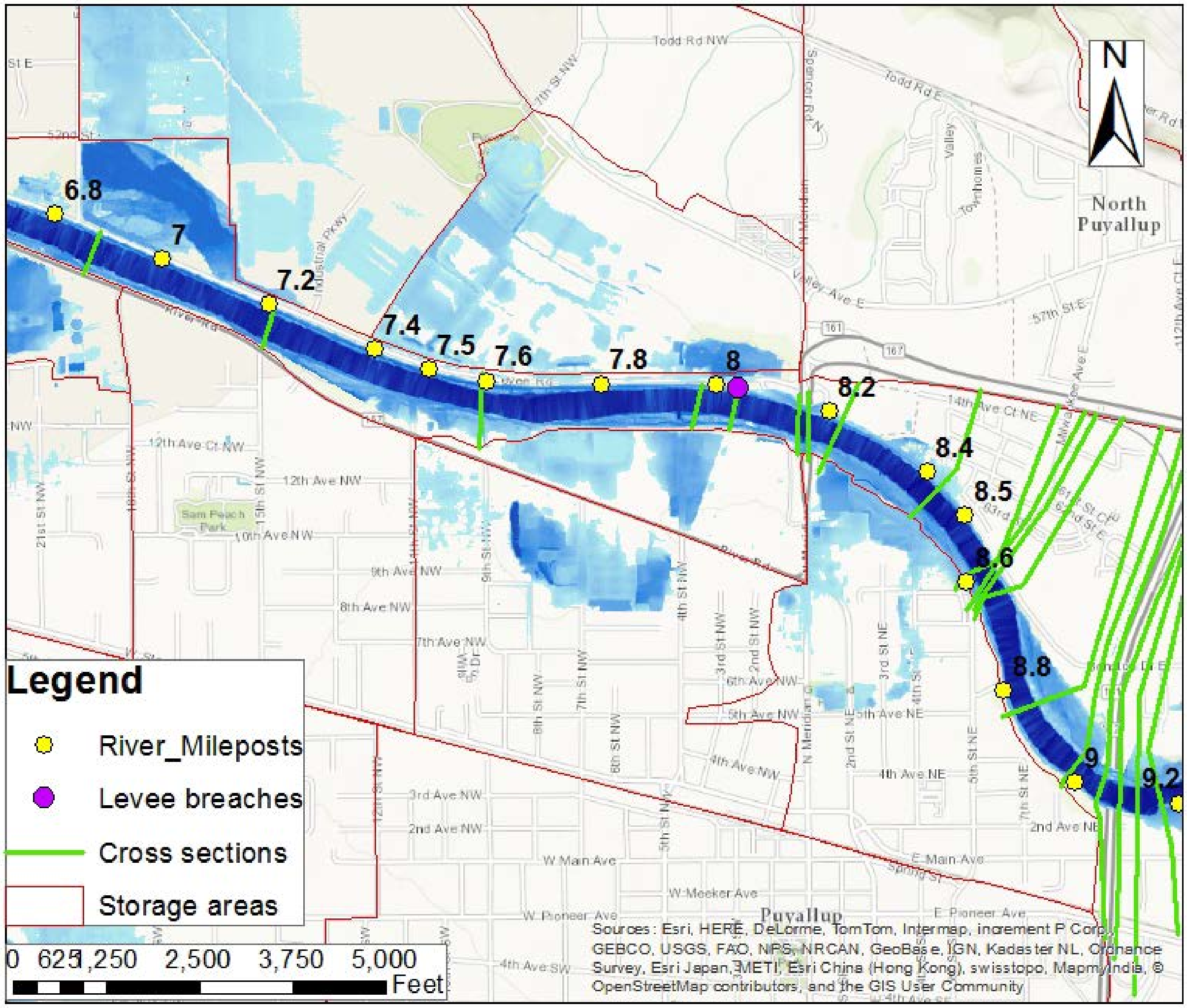


Figure 48: Lower Puyallup River from RM 6.8-9.2

The leveed area behind North Levee Rd. on the right bank and River Rd. on the left bank are overtopped in areas by a 1% AEP event. No levee presently exists from RM 8.2 to 9 and the left bank has been overtopped by the last few major flood events. The right bank is high terrain and well above the 0.2% AEP event. A levee breach was simulated at RM 8 on the right bank levee.

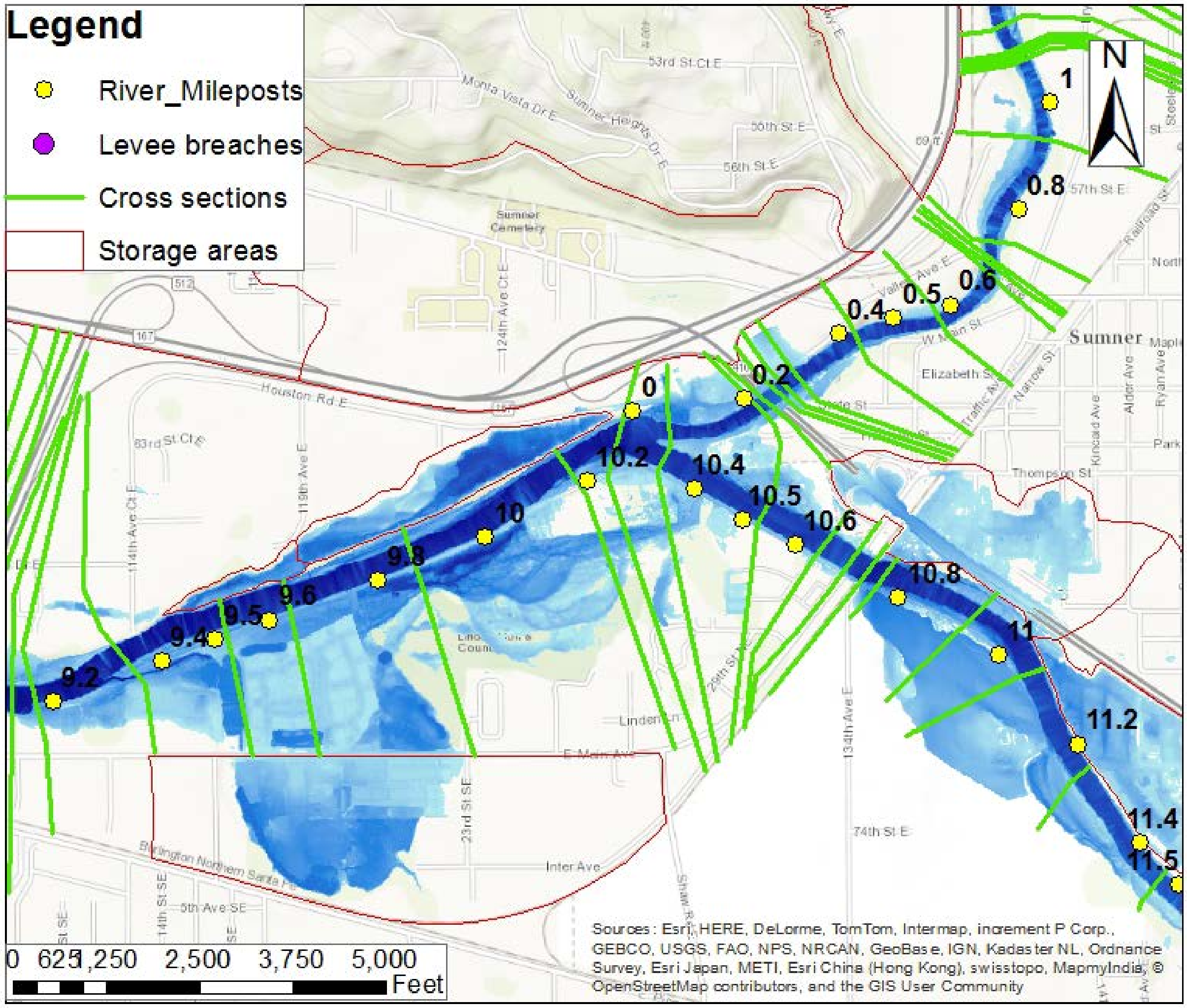


Figure 49: Middle Puyallup River at junction with the White River

Development is sparse in flood impacted areas of the Lower Puyallup from RM 9-10.2 and the lower mile of the White River is deep with high banks. The Middle Puyallup is prone to frequent flooding from RM 10.8-11.5 along the right bank developed area; the left bank is not developed.

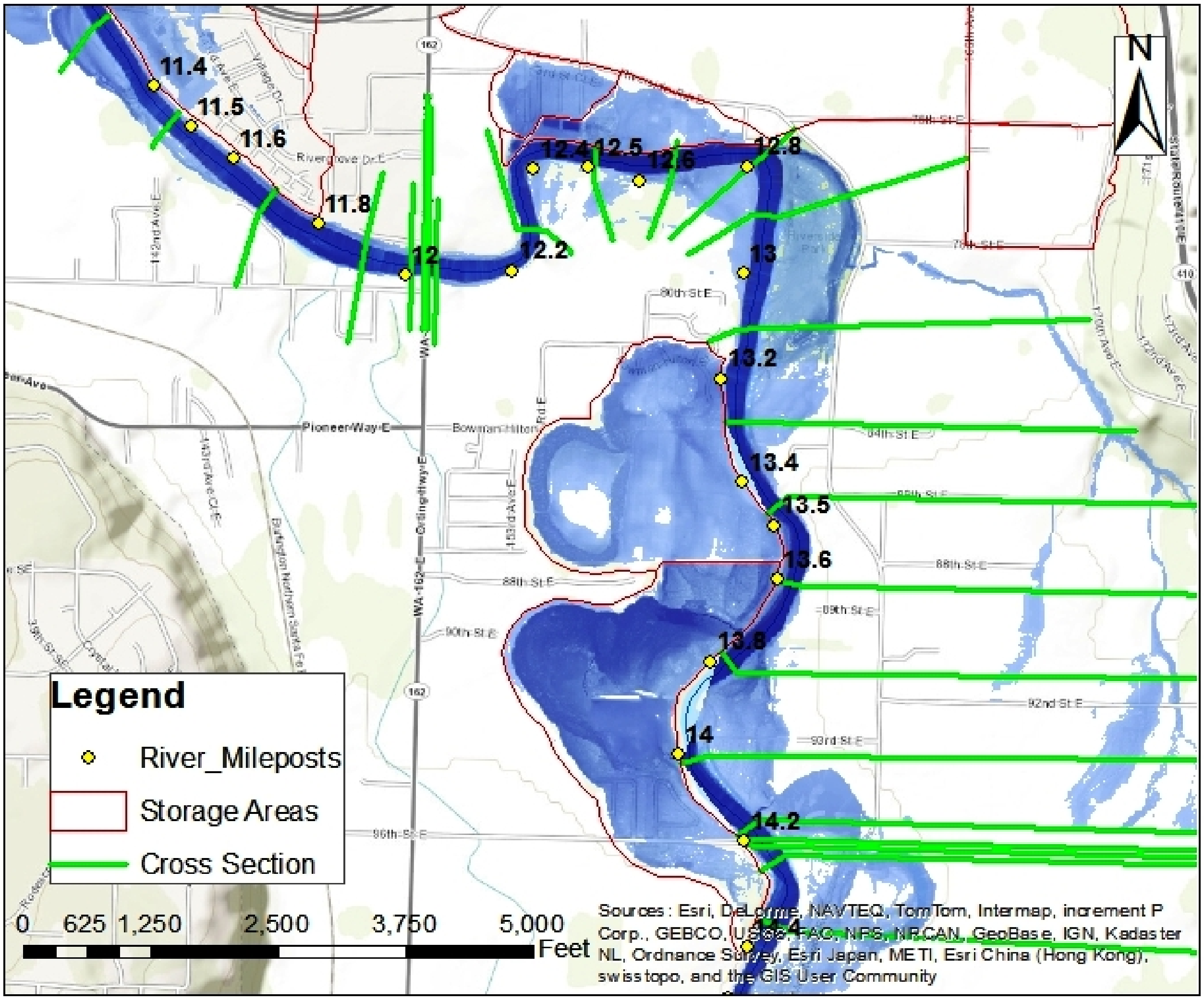


Figure 50: Middle Puyallup River from RM 11.4-14.4

The majority of floodplain through the Middle Puyallup from 11.4-14.4 is sparsely populated and consists of local levees and abandoned meander belts. The local levees typically have less than a 1% AEP level of protection.

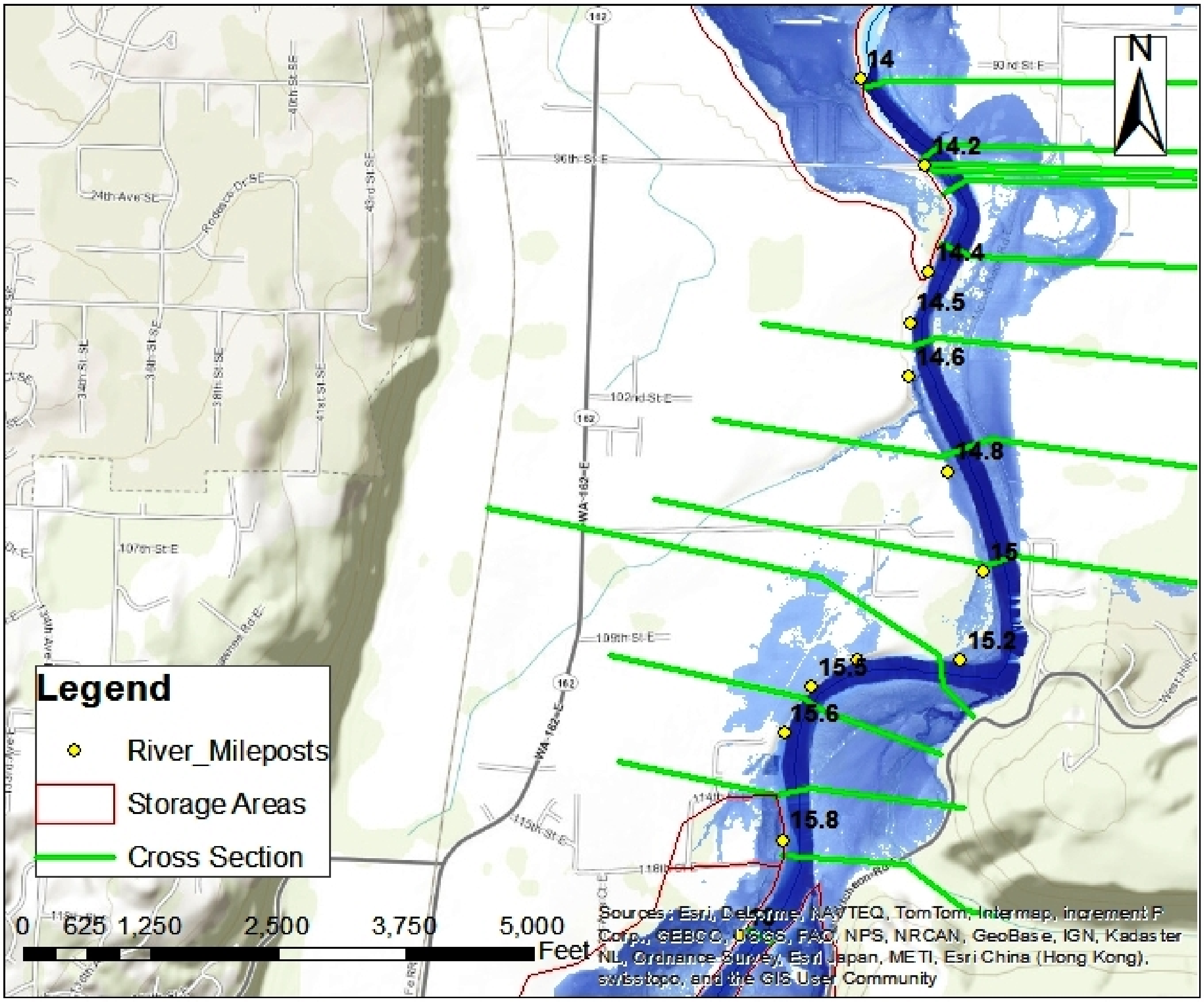


Figure 51: Middle Puyallup River from RM 14-16

The majority of floodplain through the Middle Puyallup from RM 14-16 is sparsely populated and consists of local levees and abandoned meander belts. The local levees typically have less than a 1% AEP level of protection.

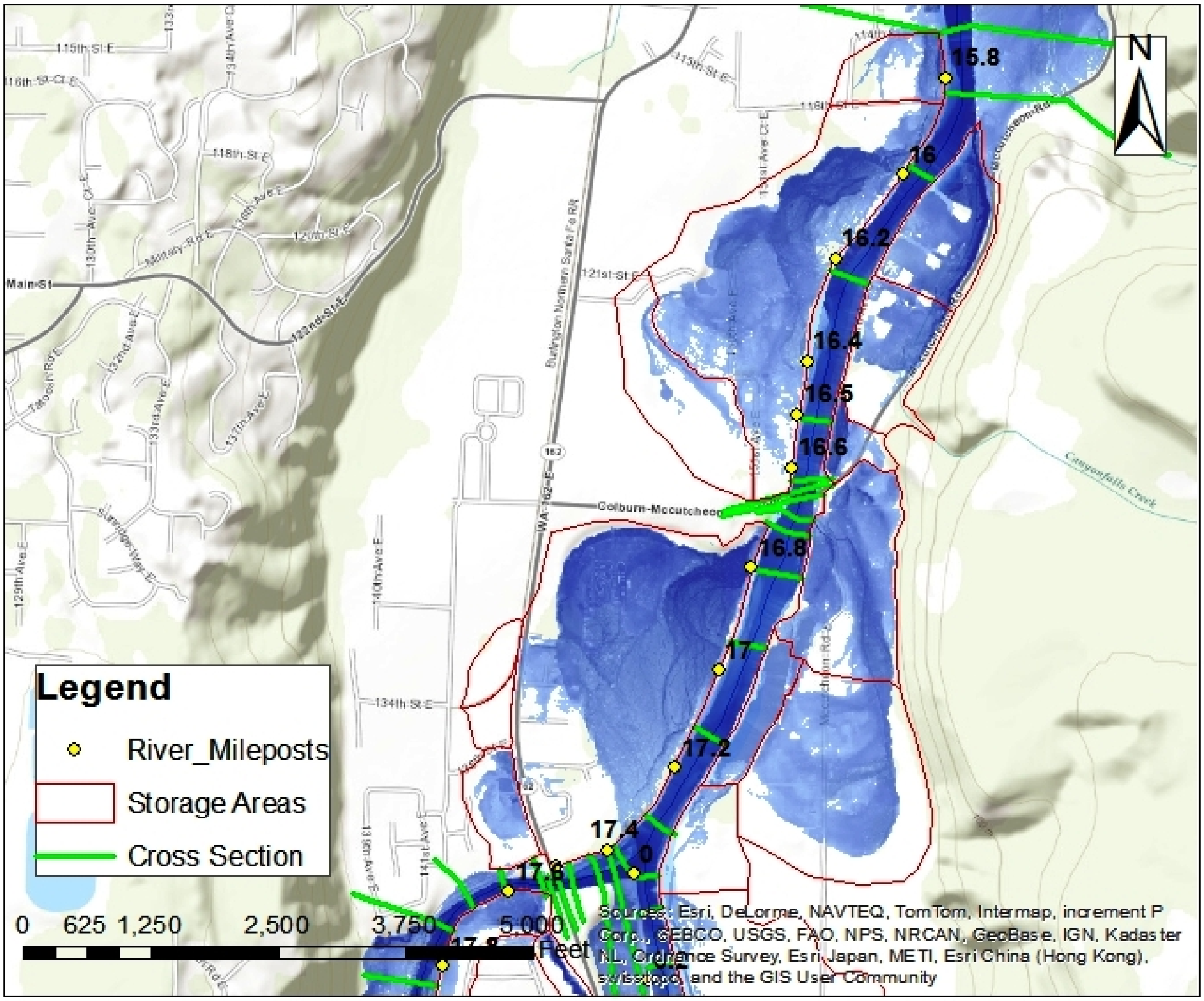


Figure 52: Middle Puyallup River from RM 15.8-17.4

The majority of floodplain through the Middle Puyallup from RM 15.8-17.4 is sparsely populated and consists of local levees and meander belts that have been cut off from the river by levees. The local levees typically have less than a 1% AEP level of protection.

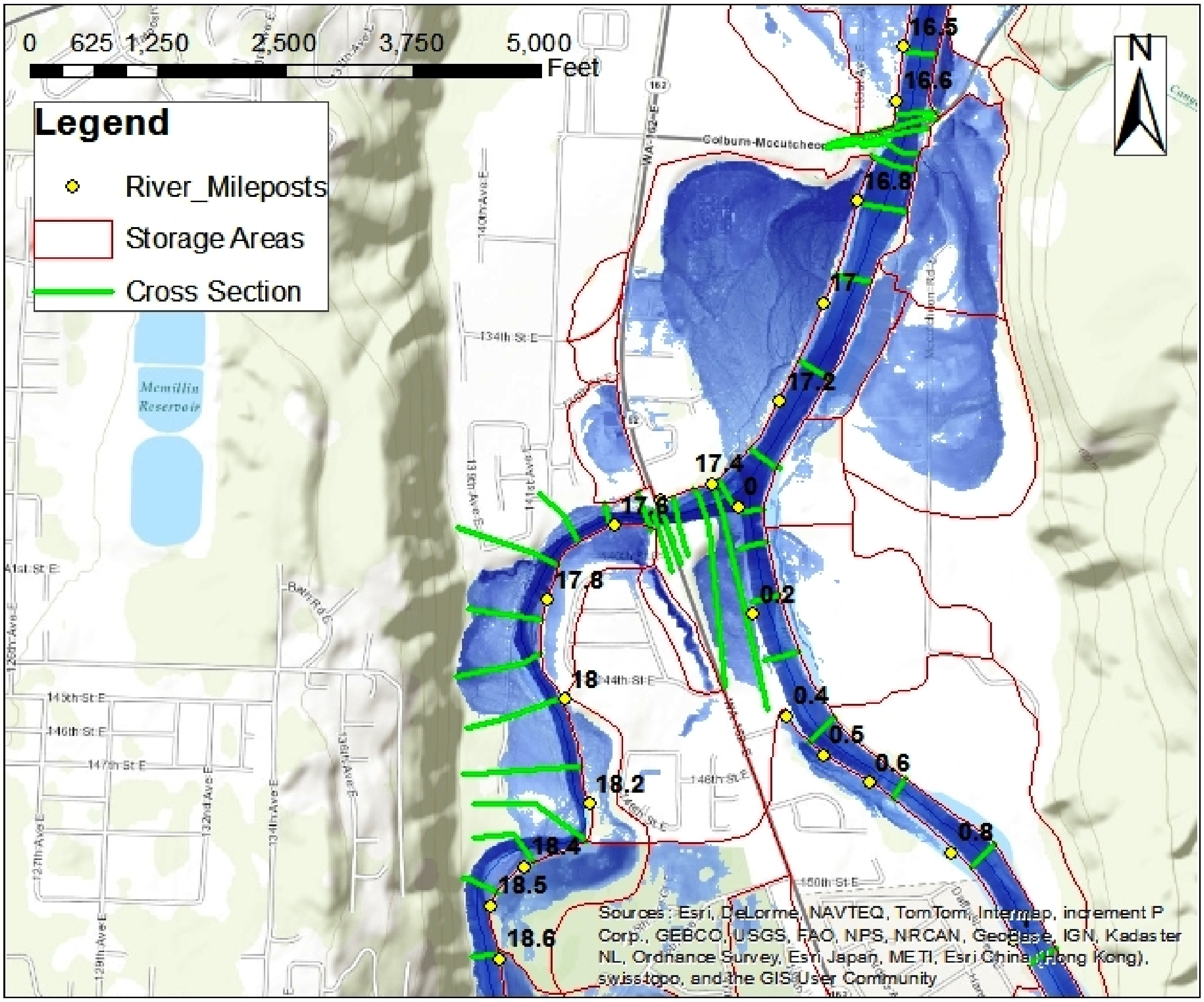


Figure 53: Upper Puyallup River at junction with the Carbon River

The area of Orting immediately south of the confluence is located on high ground and not prone to flooding. At RM 17.6 the left bank of the Upper Puyallup is at risk of overtopping at a 1% AEP event.

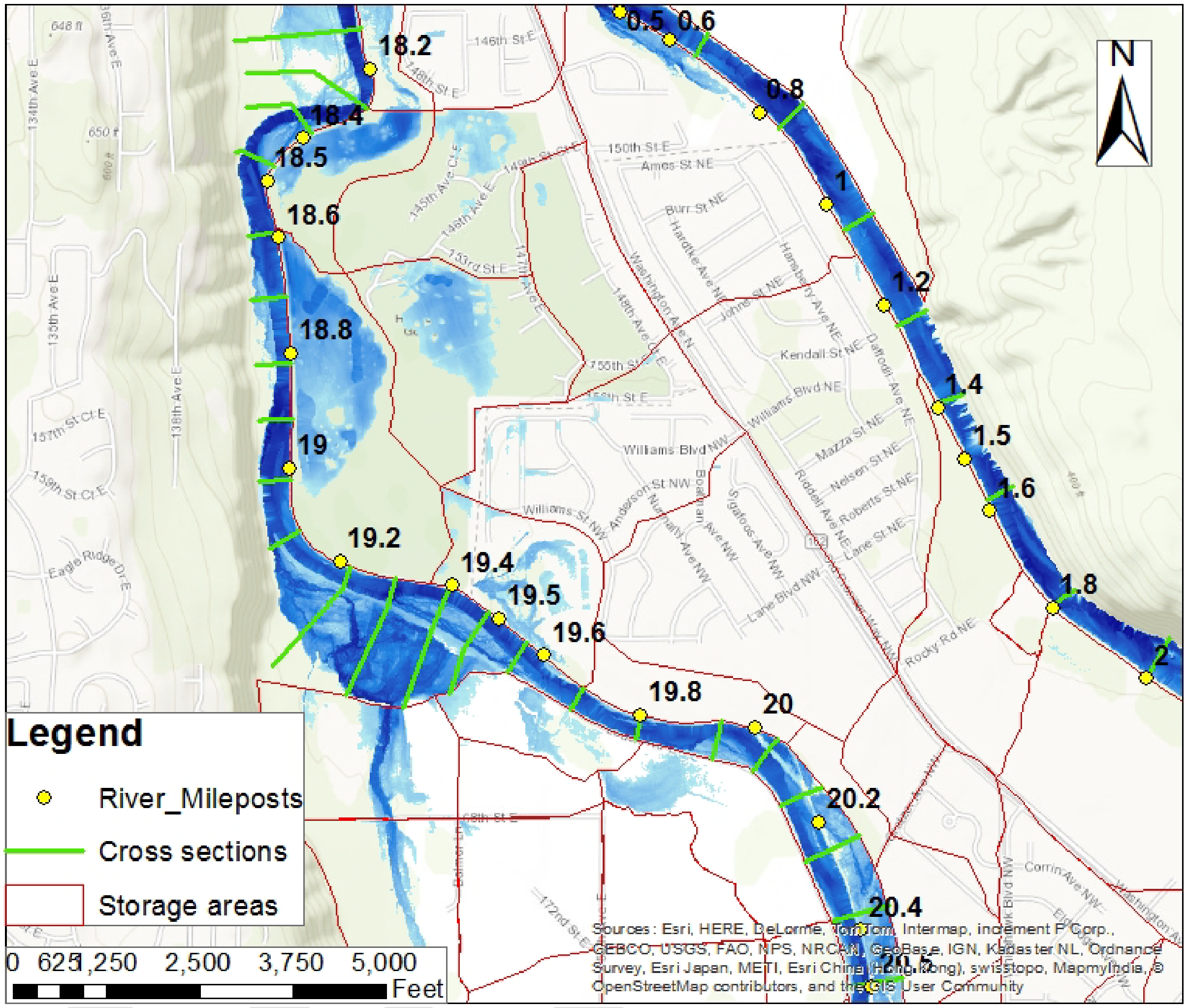


Figure 54: Orting area (north end) between the Upper Puyallup and Carbon Rivers

The area through North Orting is prone to flooding from overtopping at many locations along the right bank of the Upper Puyallup River from RM 18.2-19.5 (the High Cedars Levee) from a 1% AEP event.

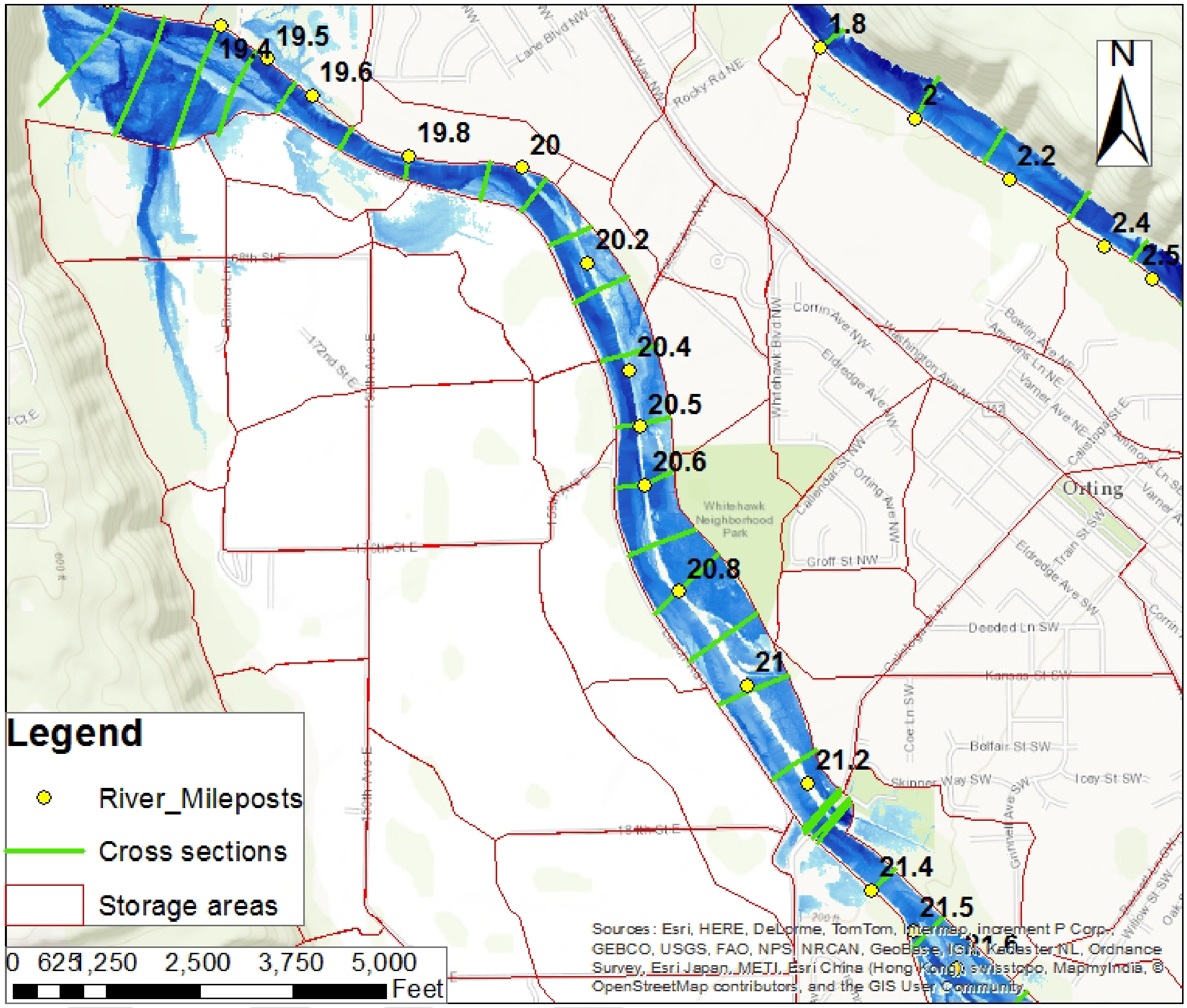


Figure 55: Orting area (central) between the Upper Puyallup and Carbon Rivers

The floodplain land slope behind the right bank levees is roughly equal to the riverbed slope and severe overtopping from larger events (0.2-0.5% AEP) would tend to flow behind the levees and pool in central Orting. The Calistoga levee along the right bank from RM 19.8-21.3 is incorporated into the model as a levee setback and is presently being constructed by the sponsor. This levee will provide protection for at least a 1% AEP event. Calistoga Bridge at RM 21.3 has experienced deposition over the last few large flood events and channel capacity there has become constricted. The left bank is composed of levees with a level of protection less than 1% AEP event. Located along the west Orting valley wall is a large abandoned channel braid that at one time was connected to the Upper Puyallup around RM 22-23. This abandoned braid is presently a small creek and is incised to a significantly lower elevation than the present main river. The Upper Puyallup River at Orting is slowly becoming perched above the surrounding valley due to deposition between levees.

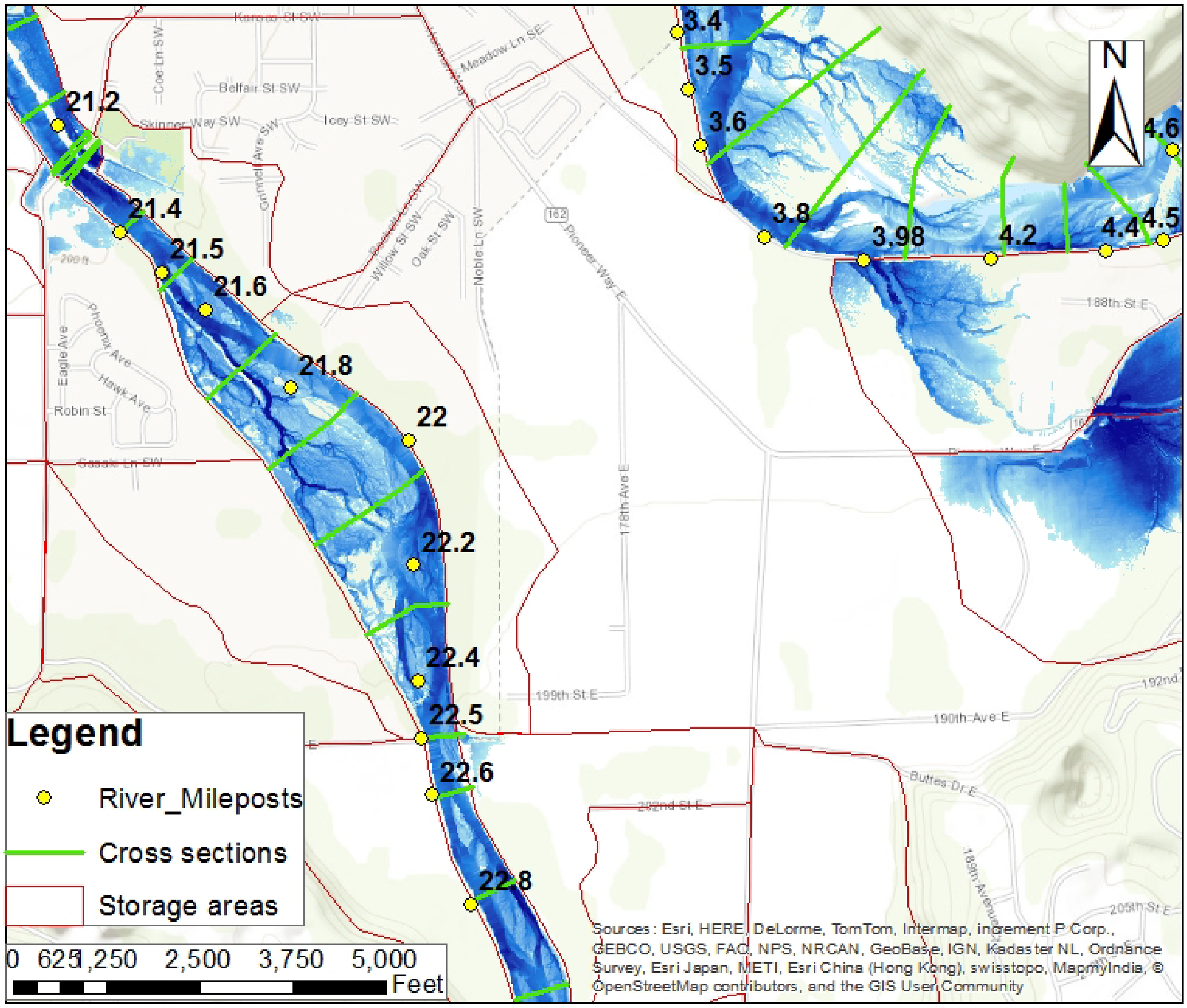


Figure 56: Orting area (south end) between the Upper Puyallup and Carbon Rivers

The floodplain land slope behind the right bank levees is roughly equal to the riverbed slope from roughly RM 21-23 (Jones and Ford Levees) and severe overtopping from larger events (0.2-0.5% AEP) would tend to flow behind the levees and pool in central Orting. The Ford levee from RM 22.5-24 is presently taller than the 1% AEP water surface, and the Jones levee is very close to the 1% AEP water surface. The pre-1997 levee at the Ford location failed during the 1996 flood event (as did the left bank levee) and flooded a significant area of central Orting. That levee was subsequently replaced with the current levee configuration (the Ford levee and the Matlock cutoff levee behind it at RM 22.5).

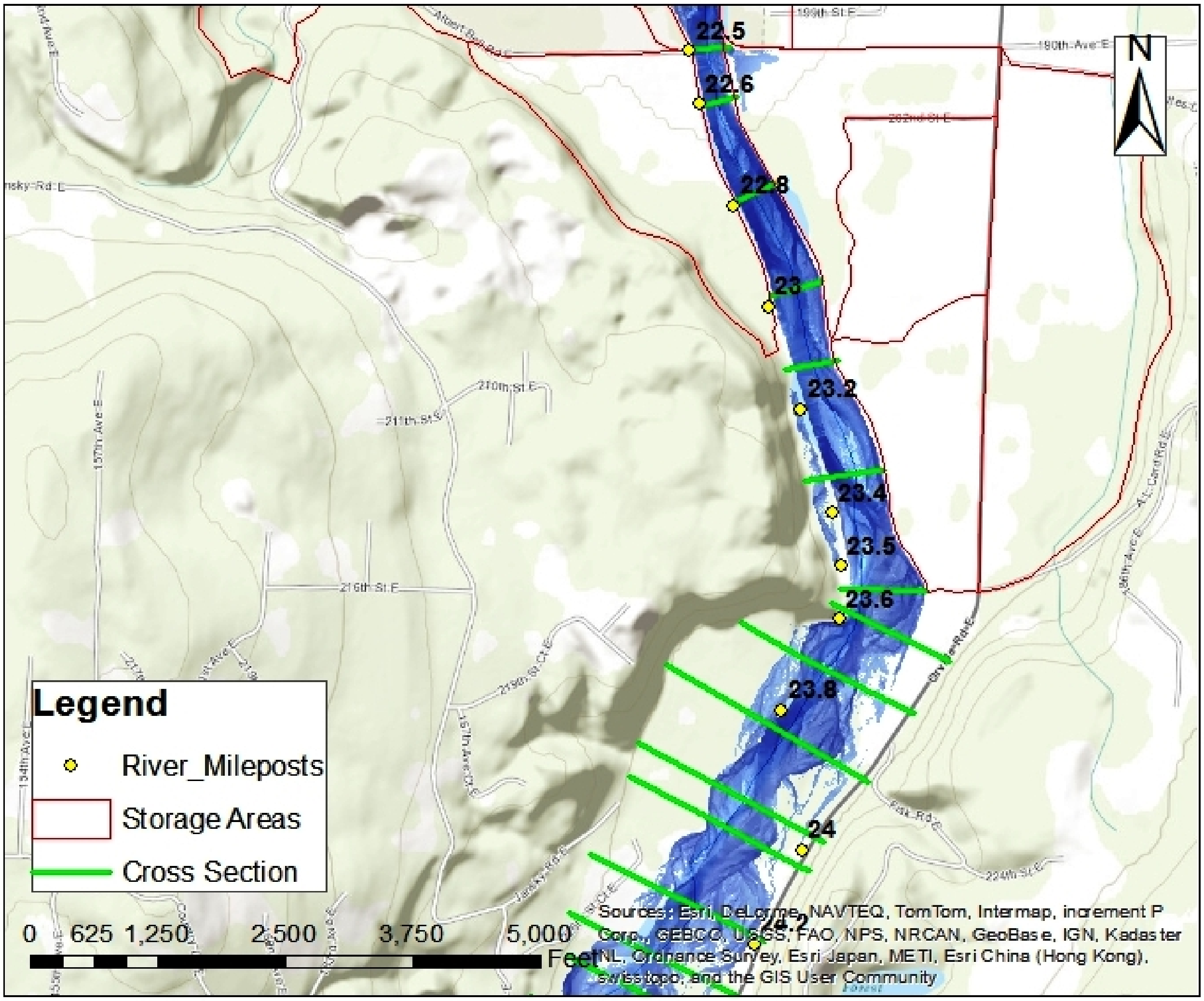


Figure 57: Upper Puyallup River from RM 22.5-25

The Upper Puyallup River above RM 24 is very sparsely developed, and consists of a high energy, steep, and braided river reach that frequently experiences damage to local levees. Levees along both sides of the river (upstream of the bend at the Ford levee) from roughly RM 25-23.7 were destroyed by the 2006 flood event and abandoned, and the river has become braided in this area. This has resulted in channel migration from RM 23.6-23.5 that has eroded the vegetated buffer in front of the Ford levee. Damage to the rock armor on the levee has occurred several times in recent years. About 250 ft. of the Ford levee was repaired in 2009 with class 5 rock. This area at RM 22.5-23.6 is a depositional zone with a significant break in bed slope.

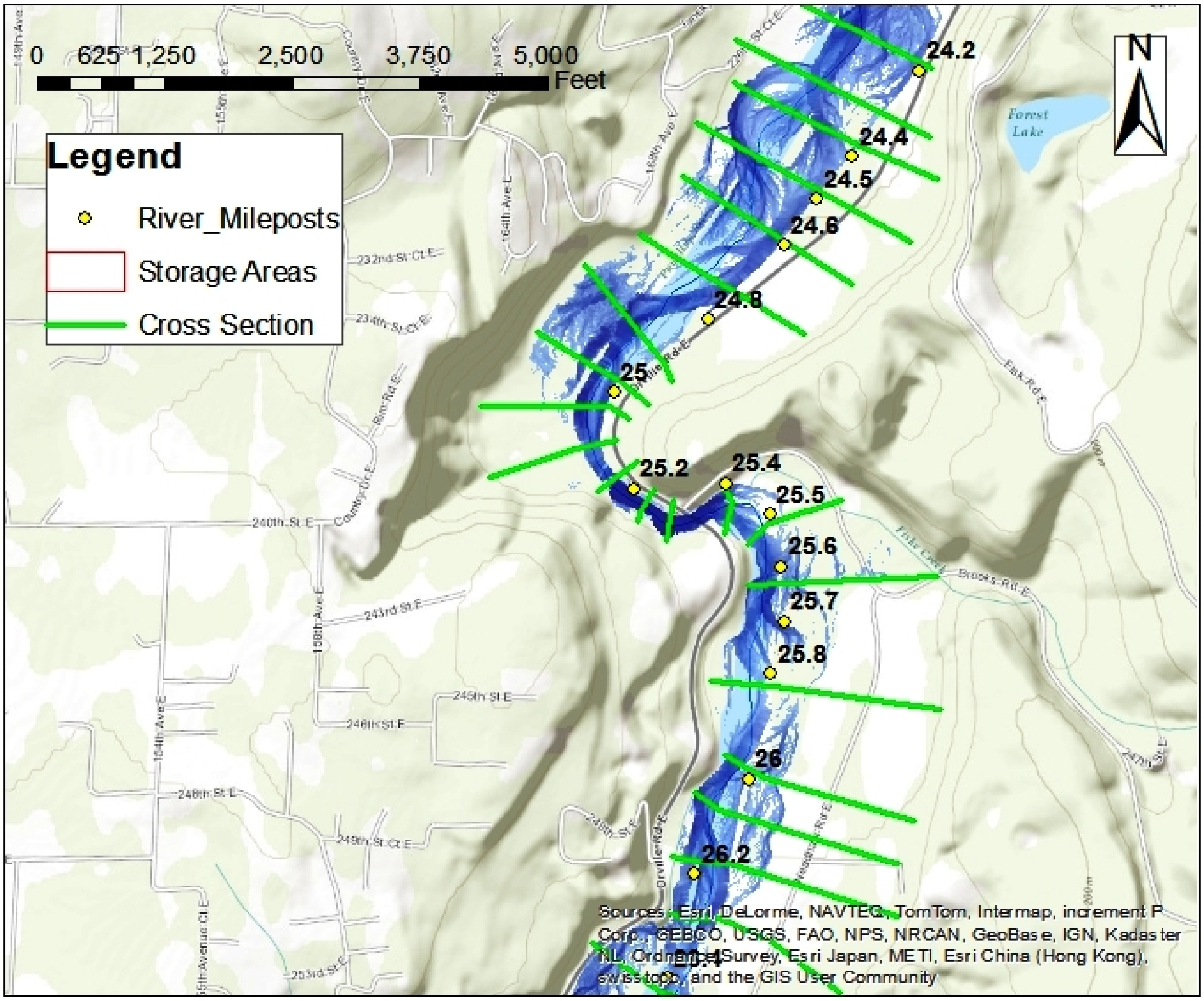


Figure 58: Upper Puyallup River from RM 24.2 to 26.4

The Needham Rd. levee is located along the right bank at RM 26 and has experienced significant repeated damage over the last decade. Pierce County has recently completed a major modification to this bank to prevent bank erosion and channel migration. The only USGS streamgage on the Upper Puyallup near Orting is located at RM 25.2. The right bank from RM 25.2-26 is sparsely developed and prone to flooding at very large events (0.2-0.5% AEP).

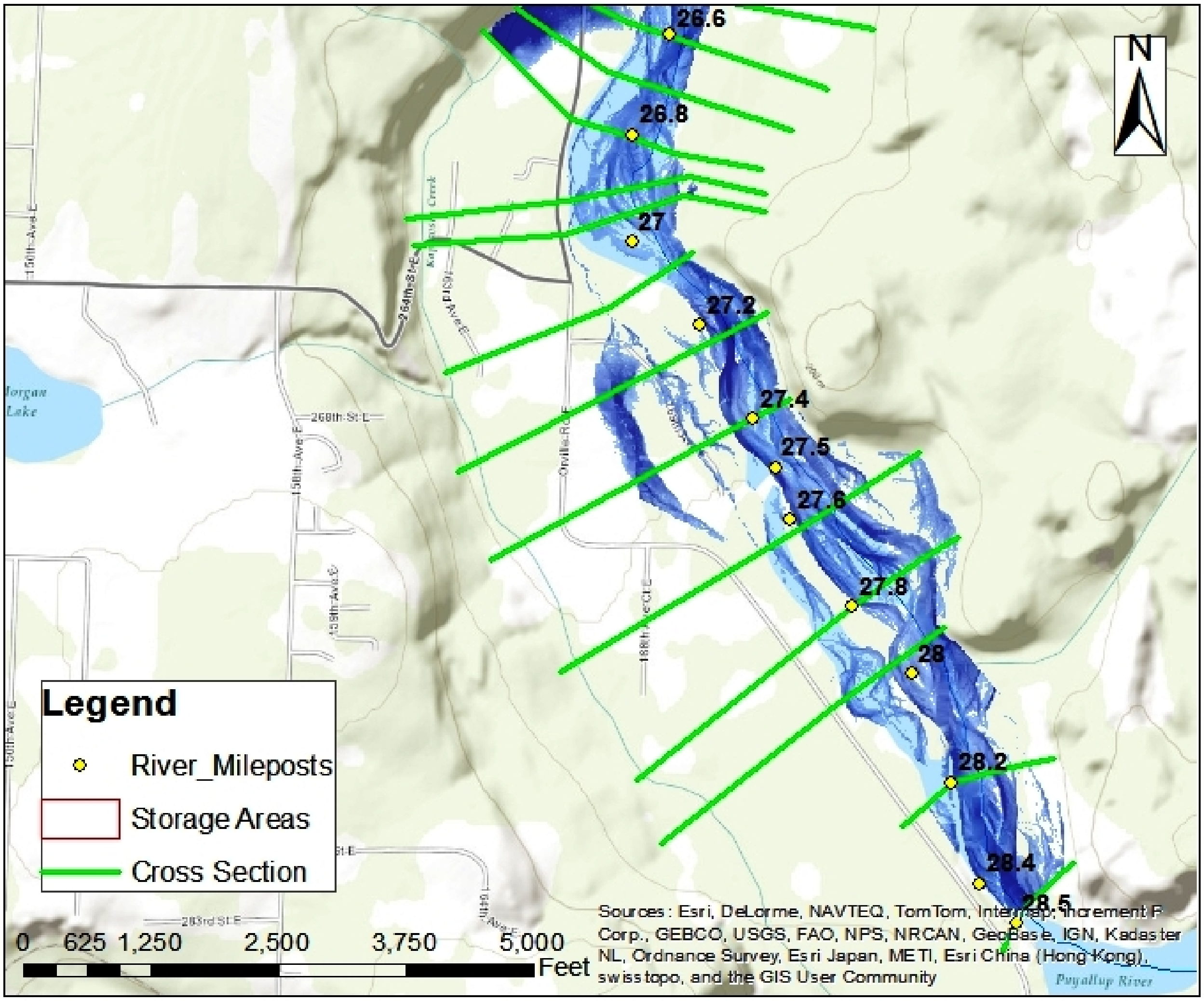


Figure 59: Upper Puyallup from RM 26.5-28.5

The left bank from RM 27-27.8 is the Orville Rd East road embankment and has experienced significant damage over the last decade. Pierce County has recently completed the first phase of a project to reduce channel migration and erosion on this side of the river. Kapowsin Creek comes in at RM 26.5 and can backwater from the main river.

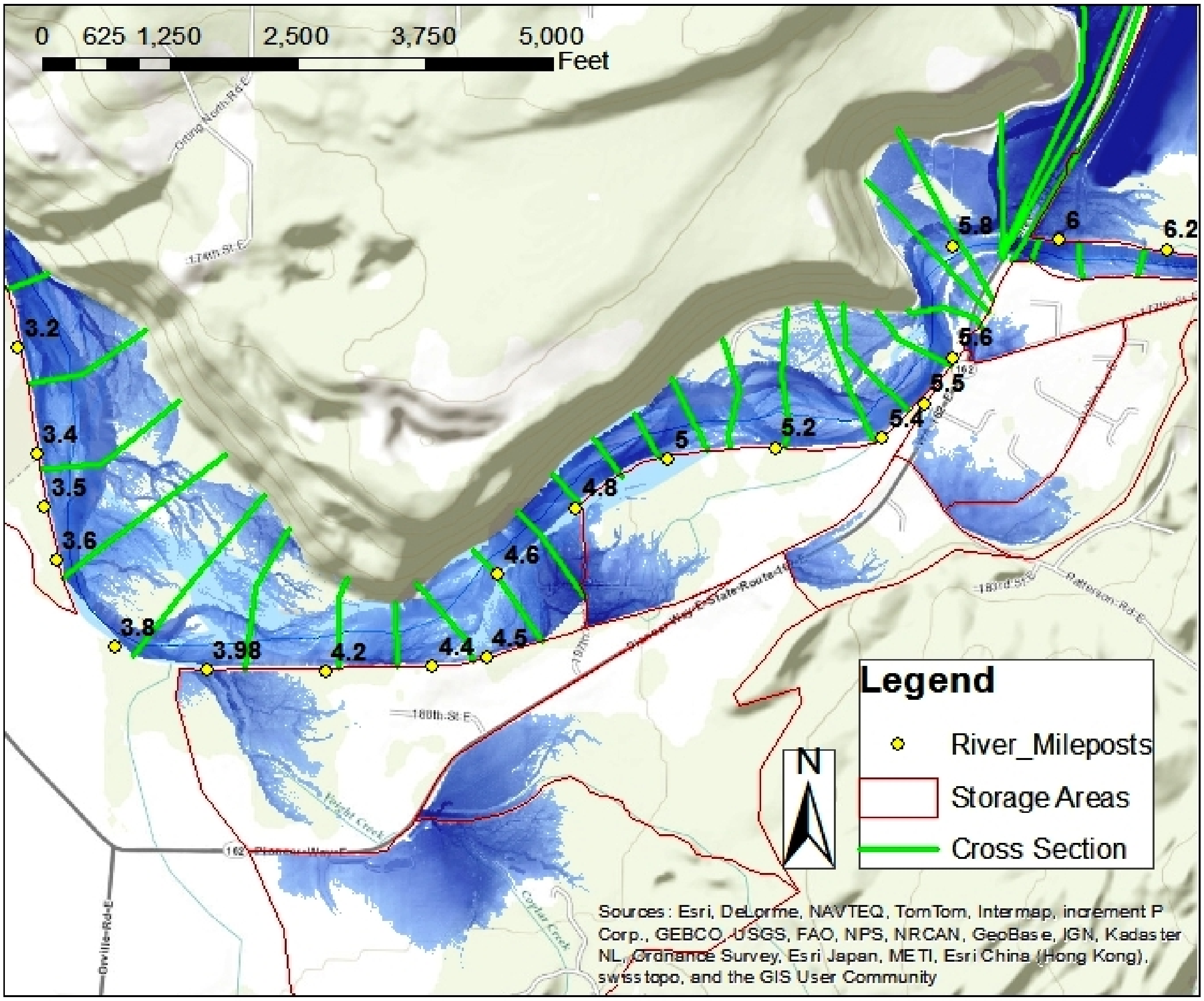


Figure 60: Carbon River from RM 3.2-6.2

The left floodplain along the Carbon River from RM 3.8-5.9 is sparsely developed. Voight Creek comes in at RM 3.9 and can backwater from the main river. The left bank from RM 3.9 to 3.5 has experienced significant erosion over the last decade as the river migrates into it. A major slope break occurs around RM 4 and the river is a braided depositional zone that has migrated into the left bank levee (Bridge St). The Guy West levee along the left bank from RM 4.8-5.6 is higher than the 0.2% AEP water surface. South Prairie creek comes in at RM 5.9 and contributes significant flow to the Carbon. Flooding up South Prairie creek is minor because its large valley was carved out by the White River thousands of years ago, and the present South Prairie Creek flow is significantly less than it was at that time.

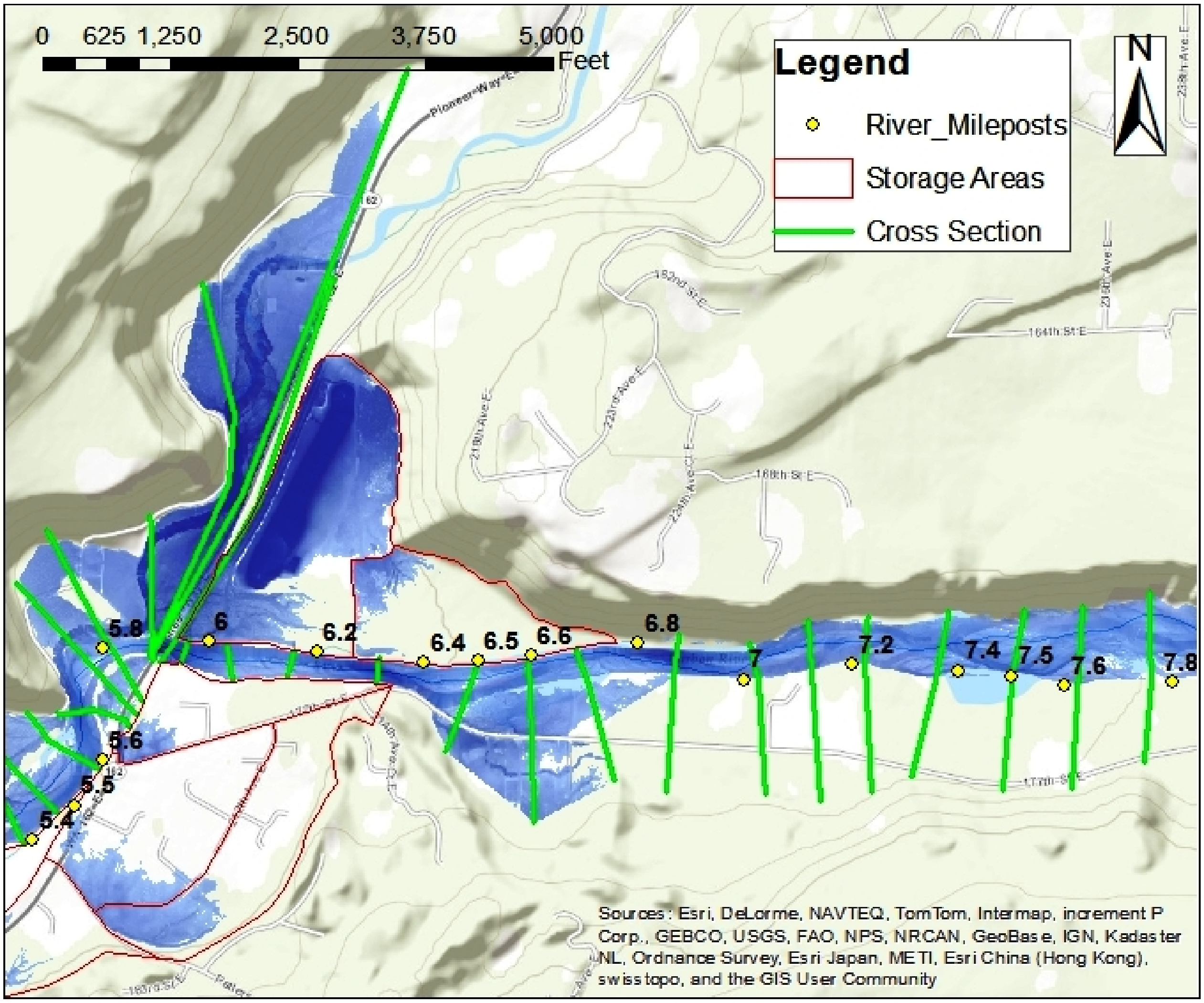


Figure 61: Carbon River from RM 5.4-7.8

The Carbon River above RM 6 is a high energy, steep, braided reach that frequently causes major damage to the left bank levee from RM 6.4-8. The Hwy 162 Bridge at RM 5.9 has experienced significant deposition and is now a constriction that causes flooding just upstream through a drainage opening in the levee on the left side of the river (Alward 1). The local levees in this area have less than a 1% AEP level of protection. It is conceivable that a major failure of the left bank levee (Alward 1) from RM 6-6.2 could send flowing water through the town of Crocker and into Voight Creek and on through to the Carbon at RM 3.9. A large lake is located on the right bank behind the levee (Water Ski) at RM 6.1 that is capable of absorbing some degree of right bank flooding. Preliminary modeling has shown that South Prairie Creek is capable of flooding across Hwy 162 into the lake.

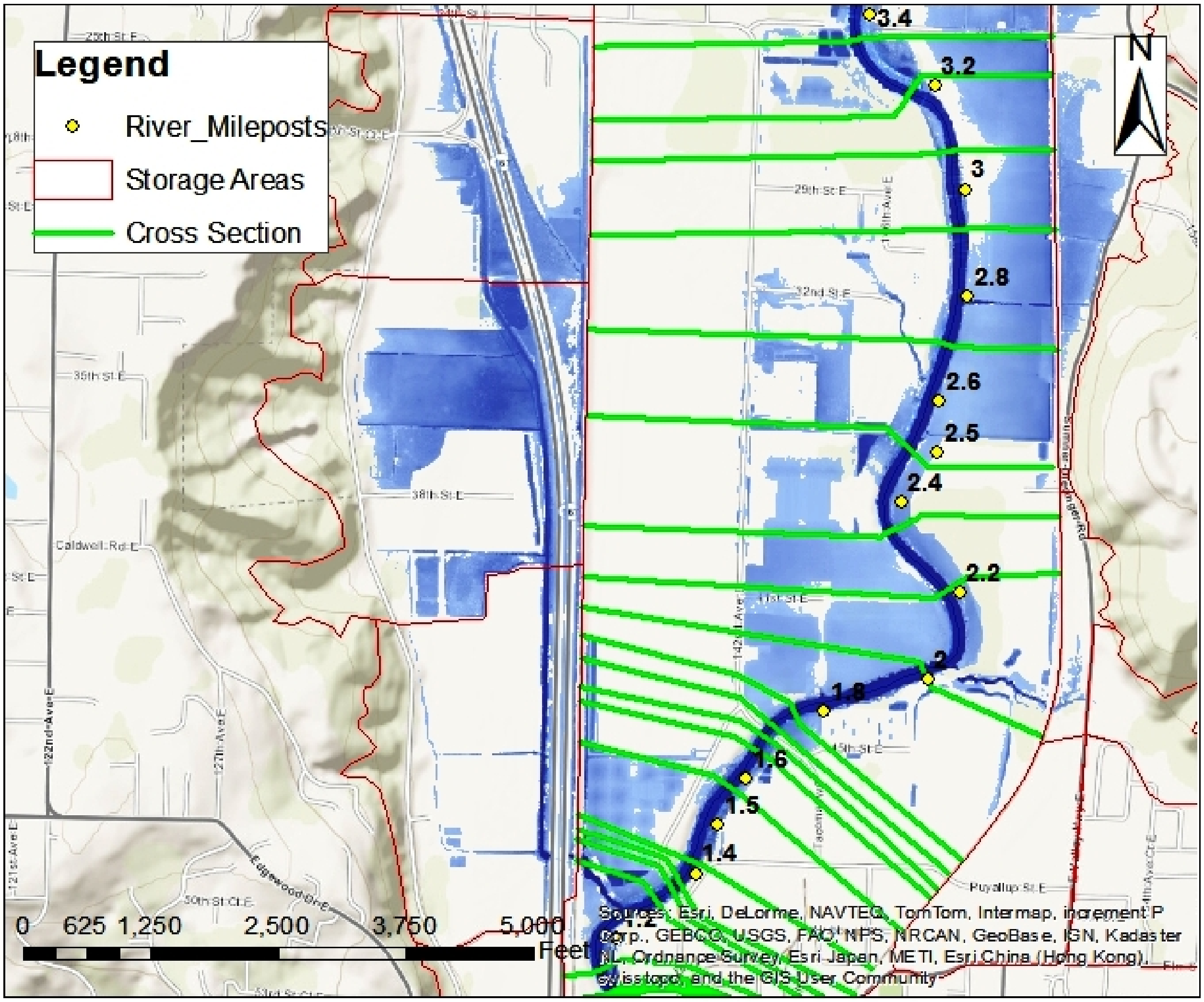


Figure 62: White River from RM 1 to 3.4

The Lower White River is heavily backwatered from the Puyallup and is a depositional zone for sand and fine sediment. The left bank is high terrain well above the 1% AEP floodplain up to Rm 2.5. The left bank from RM 2.5-4.2 is agricultural land and a golf course. The right bank is commercial development and prone to flooding. Backwater occurs along the west side of Hwy 167 up the drainage ditch a great distance.

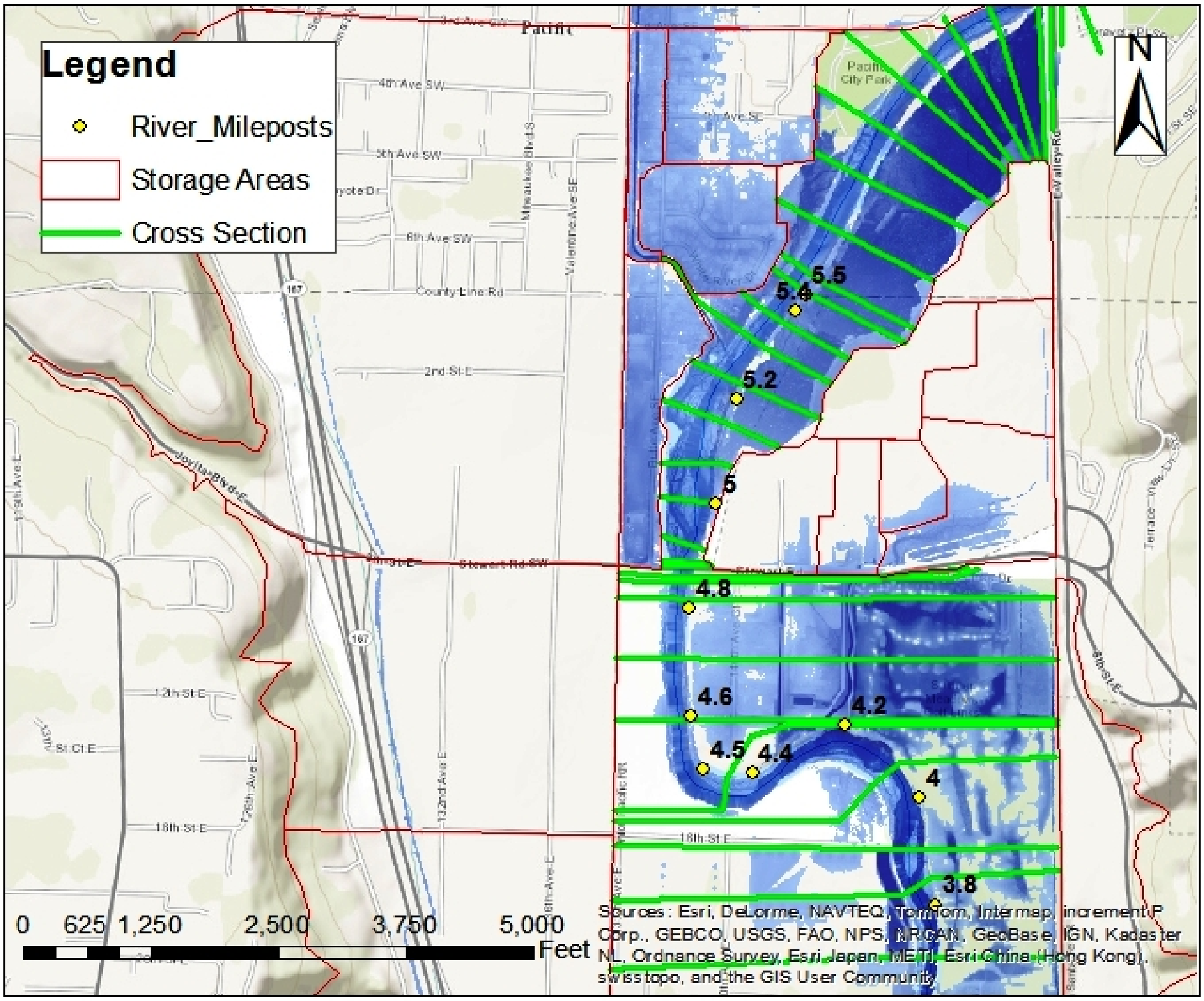


Figure 63: White River from RM 3.8-6

The White River from RM 4.8-6 is a depositional zone that has aggraded significantly since 2006 and has lost channel capacity. Stewart St Bridge has also lost capacity. The left bank from RM 4.6-4.8 is high terrain but prone to flooding from a 1% AEP event. The right bank from RM 4.9-5.2 is an old spoils pile from channel excavation over 1984-1987 and is high terrain but also prone to flooding. The left bank levee from RM 4.9-6 is scheduled to be setback beyond the historical meander belt over the next few years and is incorporated into the model as the County Line setback. The Government ditch enters the White River on the right bank at RM 5.3 and is prone to backwatering at frequent events through a 4 barrel culvert road crossing a great distance up into Pacific. This area is presently being analyzed in great detail by King County to reduce backwatering.

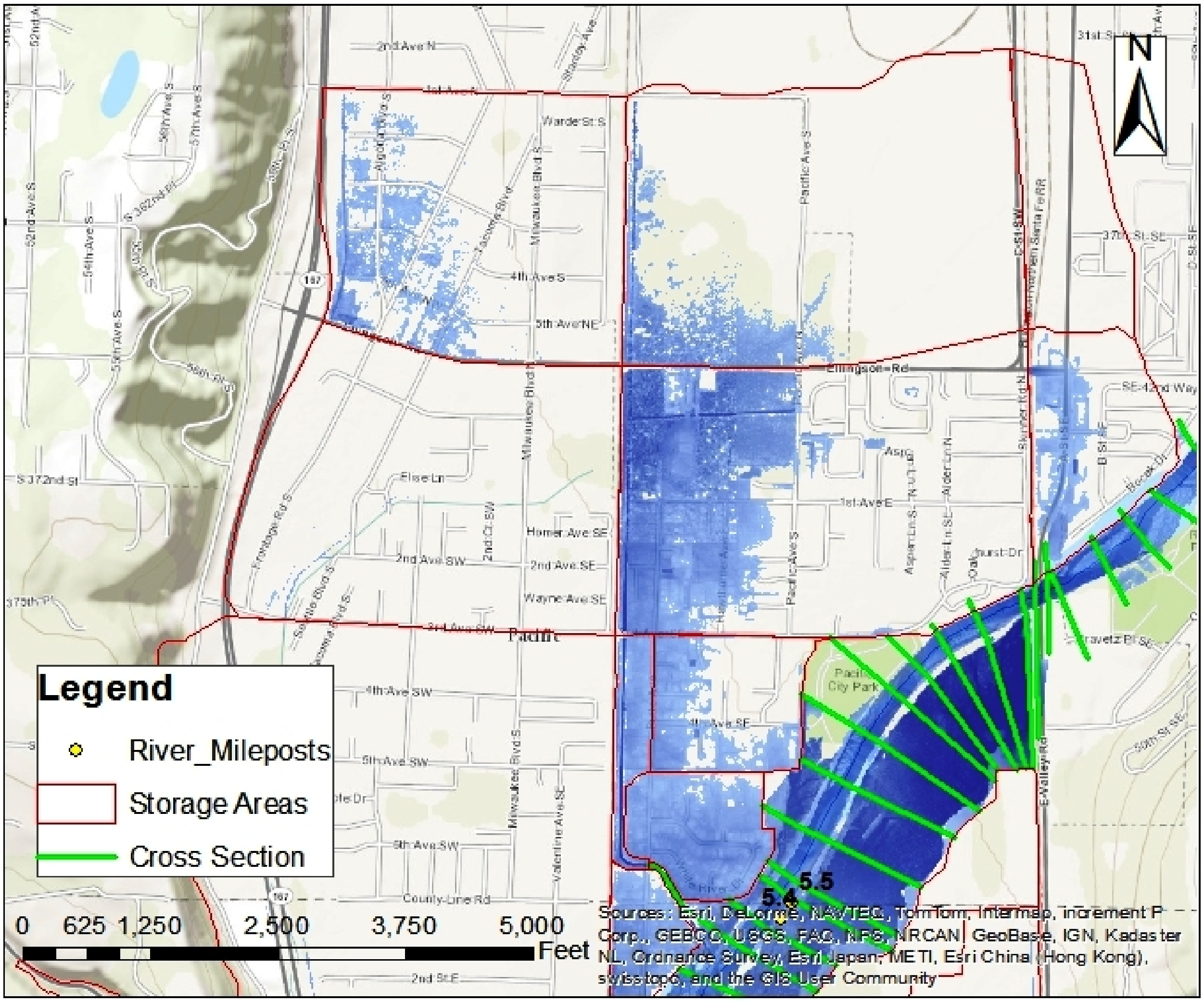


Figure 64: White River near the town of Pacific

The Right bank around RM 6 just upstream and downstream of the A St. Bridge has seen significant aggradation in recent years and is prone to flooding. The flood extent shown in central Pacific is from the Government ditch backwatering as well as right bank overtopping at RM 6 moving west to the lowest point in the storage area.

Flood extents shown for existing conditions were used for comparison with those of future conditions to site measures, determine real estate analysis necessary, and in general to illustrate the magnitude of possible flooding that may occur throughout the study area.

8.0 References

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Appendix B-2

Hydrology and Hydraulics

Hydraulic Modeling of Future Conditions And Sedimentation Analysis

Puyallup River Basin

Flood Risk Reduction Feasibility Study

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1.0 Sea Level Rise

USACE ER 1110-2-8162 requires feasibility studies to examine three scenarios to consider the sensitivity and adaptability of projects to sea level change (SLC). These scenarios include a low, intermediate, and high forecast of SLC. For TSP (Tentatively Selected Plan) modeling the economic life of 50 years (2026-2076) was used as the period of analysis for SLC. In 2014 the release of ETL 1100-2-1 defined the project planning horizon as 100 years, which is distinctly longer than the economic life of 50 years. This change to a 100 year prediction may be incorporated into modeling and design for post-TSP efforts if the PDT is directed to do so. Both of the estimate charts are shown below. The NOAA sea level rise (CESL) tool was used to estimate the long-term trends for the Seattle station. The range of sea level change projections (for 50 years) for the TSP design at the mouth of the Puyallup River is low 0.34 foot, intermediate 0.86 foot, and high 2.53 feet. A value of 1 foot increase was used, to be conservative with the intermediate estimate, for future conditions modeling which gave an elevation of 12.36 Ft NAVD88 at the mouth of the Puyallup River (from the 11.36 ft. value used for existing conditions). The residual levee height factored into designs at TSP is more than sufficient to account for the intermediate 100 year estimate. The effect of the one foot change was an increase in water surface profile from Commencement Bay to roughly 2.5 miles upstream on the Puyallup River. Above this point in the river the change made little difference. Within this 2.5 miles loading will be increased slightly on the two federal levees. Existing levees are typically 5-8 ft. tall through the tidal area. An increase in one foot will increase the frequency and magnitude of loadings to a given percentage.

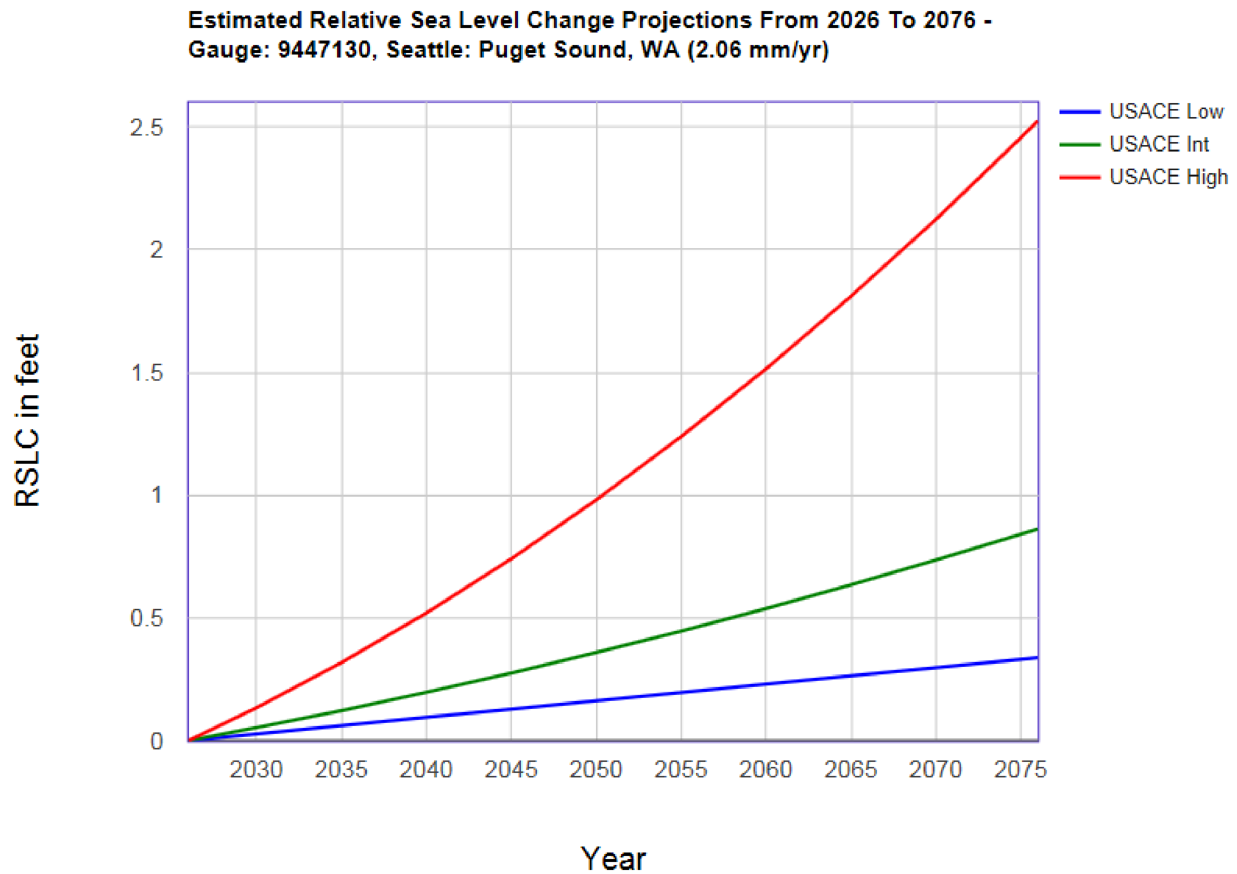

Figure 1: Relative Sea Level Change for 50 years

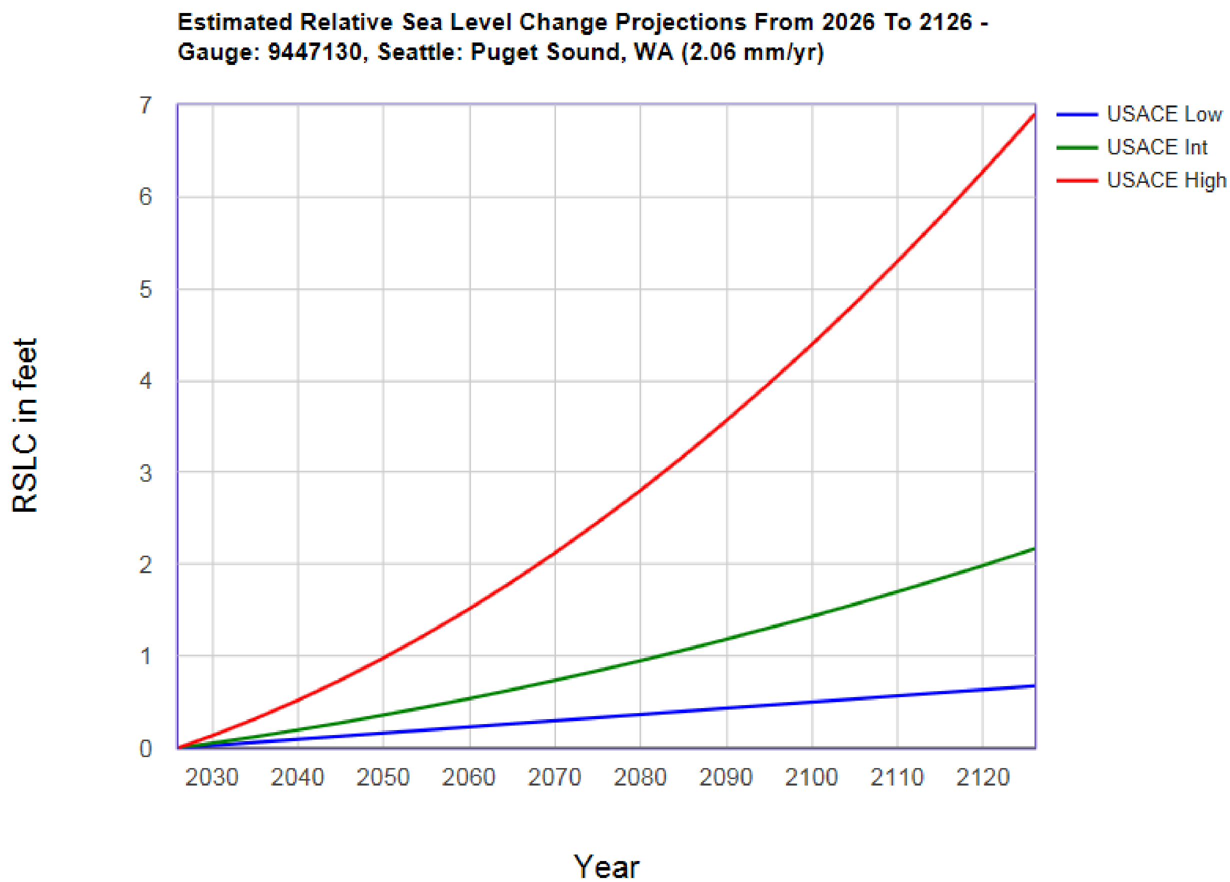

Figure 2: Relative Sea Level Change for 100 Years

2.0 Climate Change

USACE ECB 2014-10 requires a qualitative analysis of climate change variability in hydrologic analysis for inland watersheds for projects that have not reached the TSP milestone prior to issuance of the ECB (May 2014). Hydrologic conditions in the Puyallup River Basin are expected to change with climate change over the next 50 years. A recent literature survey by the USACE (CWTS 2015-23, for region 17) has summarized the state of existing research and conclusions about climate change for the Pacific Northwest (USACE, 2015). There is a strong consensus that future storm events in the region will be more intense and more frequent compared to the recent past. However, consensus over future projections of hydrologic change translating to change in streamflow is mixed.

The University of Washington has attempted to quantify possible changes and this was summarized by the USGS in their 2012 study of the Puyallup River Basin. Models from the University of Washington Climate Impacts Group indicate that over the next century, the Pacific Northwest will likely see a trend toward wetter, warmer winters and hotter, dryer summers in response to climate change. Climate change during the next 50 years is projected to result in earlier snowmelt and reduced summer river flows (Vano et al. 2009). Based on recent studies, if climate change leads to increased rainfall, temperatures, and snowmelt, such changes may impact the runoff and discharge of rivers in the Puyallup River Basin. Prediction of future climate change in Washington State have included an 11.6% increase in runoff for 2010-2034 and an 18.1% increase in runoff for 2035-2059 (USGS, 2012). Over the project life, an 18% change would translate a present day 0.01 AEP flow into a 0.05 AEP flow. An increase in runoff in the upper basin may transport more sediment from Mt. Rainier to leveed areas of the lower basin. It may also generate more sediment from within the National Park Boundaries. This would likely accelerate deposition in many leveed areas. The USGS has estimated that bedload in these rivers may increase on the order of 30-50% with increased flows (USGS, 2012).

3.0 Estimation of Future Sedimentation

For TSP selection an estimate of future deposition in the project area was needed to provide for a relative comparison between alternatives. Described here is the process by which preliminary estimates were developed from historic data. The estimates developed are expected to change based upon the result of sediment modeling expected to occur between TSP selection and NED (optimization of the TSP). Sedimentation in this basin is extremely complicated. Guidance was sought from the USACE Committee on Channel Stabilization on how to approach the topic and when analysis should occur in the General investigation process. The committee recommended that reach specific sediment models be developed in the feasibility phase (USACE, 2014 issue four). The sediment modeling is expected to provide the final answer to inform future conditions which will be carried forward to later design phases. Estimates described below, in addition to informing TSP selection, will also be useful in calibrating sediment models to bed volume change.

Supply of sediment from Mount Rainier and surrounding areas to the Puyallup basin study area is highly variable, with much of the bedload coming from sporadic rock falls at the glacial origins of its rivers. Transport of glacial material into the fluvial system is highly dependent upon the occurrence of extreme rainfall events (USGS, 2010). Residence time of sediment from glacial origins to the basin study area can be on the order of decades to centuries (USGS, 2010). A significant amount of sediment can be produced from within the national park boundaries. Future conditions within the project study area will likely continue to be characterized by excess sediment supply. Estimation of historic annual sediment load range from 1,200,000 to 860,000 tons/yr. on the Lower Puyallup and around 500,000 tons/yr. on the White River, with the remainder coming from the Upper Puyallup and Carbon Rivers (USGS, 2012). Areas that have historically seen aggradation of sediment will likely continue to aggrade as the system transports large quantities of sediment to leveed reaches of the lower basin.

In a typical river system with high sediment loads, deposition of material on the channel bed occurs when the available supply of sediment exceeds the transport capacity of a given reach. This process typically continues, in the absence of destabilizing events, until some dynamic equilibrium state is reached where the system has adjusted its bed slope to generally transport sediment through a given reach without net erosion or deposition (Leopold, 1964). Due to the highly developed and leveed state of the Puyallup system within the study area, it is doubtful that such an equilibrium state could exist within the confines of the existing leveed system. Historically, developed areas such as Orting, Pacific, and Puyallup existed as massive alluvial fans consisting of braided channels and massive gravel bars that were periodically buried by catastrophic mud flow events. Returning the system to even a quasi-equilibrium state, if it ever really existed, is likely not possible given the present level of development. The result of confining rivers in this system has been a need to dredge or to raise levees higher in the most active depositional areas. The consequence of continuing to raise levees in depositional areas is the river channel can become perched above the surrounding floodplain. This would add an additional risk of damages caused by levee failures that would not exist if the channel were dredged to maintain a consistent bed elevation. As levee projects age the risks associated with a perched channel increase. The residual risk after the design life may be much greater than current levels. As of the TSP milestone, the potential for perched channels to develop in depositional areas is being assessed through sediment modeling.

For the purposes of this study we are concerned with a 50 year project life span. Estimation of bed elevation change from deposition at levees is necessary to estimate future water surface elevations at the end of the project life (the with-project condition), and potential levee overtopping if no further modifications were made to the leveed system (the without-project condition). Typically the method employed to estimate future water surface change from sedimentation is to develop sediment transport modeling from observed data. At this time in the study, while the TSP is developed, that level of detail is not needed in estimation of future conditions. It will, however, come in later in the study for future with- and without-project conditions when the selected alternative is developed to a higher level of design. Preliminary methods were used for TSP to estimate water surface change from observed changes in channel bed material volume over a 25 year window of data. Rates of bed material volume change were extrapolated over the project life. This provided for a relative comparison between existing conditions and future without project conditions. As mentioned above, the

PDT has not been instructed to incorporate changes in sedimentation with climate change as of the TSP milestone.

3.1 Analysis of Bed Material Volume Change From 1984-2009 and Extrapolation

Cross sections surveyed in 1984 and again in 2009 were obtained from the USGS (Prych 1988, USGS 2010). Many of the cross sections were coincidental (i.e. 1984 cross section locations were re-occupied in 2009), and the USGS was able to numerically calculate an area difference between them from the survey data (USGS, 2010). The area difference at each cross section location was then used by the USGS with neighboring cross sections to determine volume in what is commonly called the average end area method:

$$\Delta V_{i \to i+1} = {}^{1}/_{2}\,(\Delta A_i + \Delta A_{i+1})\Delta x_{i \to i+1}$$

The result of calculating this throughout the study area was incremental volume of bed change that has occurred over the 25 year time period. Note that these are snapshots in time of channel conditions and do not provide much insight into what has happened in-between. The analysis is presented in their Channel Conveyance-Capacity investigation of the Puyallup basin and the reader is referred there for further explanation of theory (USGS, 2010). The USGS analysis was presented in feet of bed change rather than volume of change, which was desired for this analysis. The USGS analysis was obtained and volumes were checked for accuracy from reach lengths and channel widths from the model. A plot of the volume data is shown in Figure 3 for Puyallup River and Figure 4 for the White River. Each of these figures shows volume change by river mile over the 25 year period. Note that leveed areas are shown as colored lines in each figure, with black line indicating areas that are not leveed.

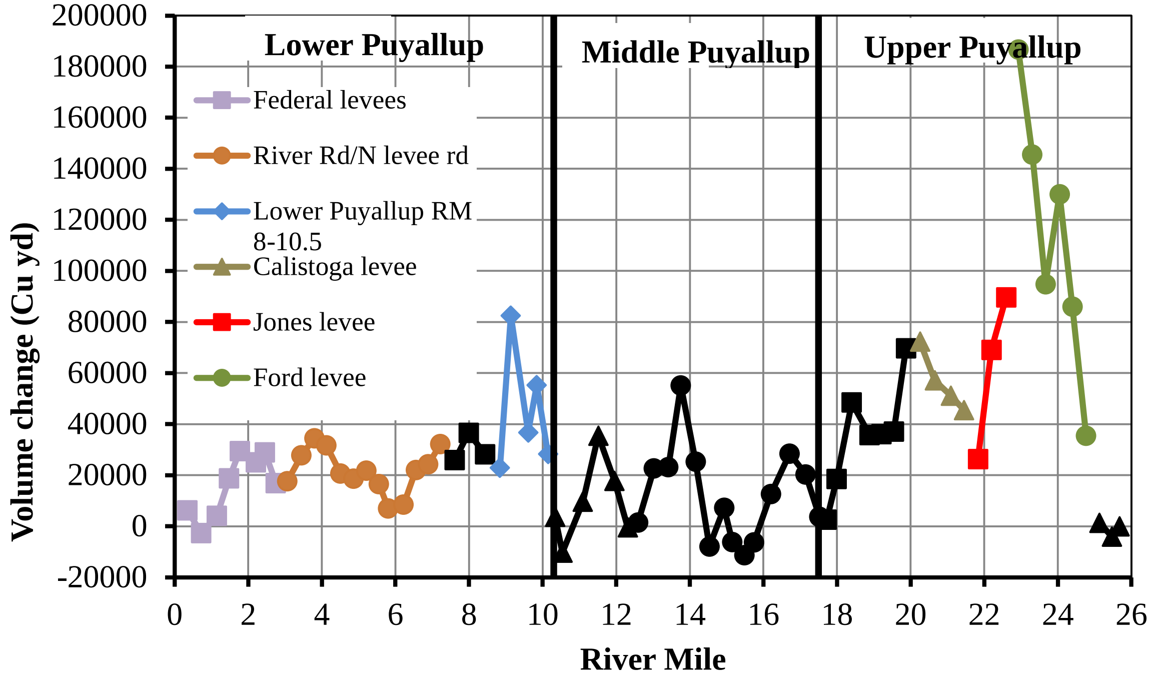

Figure 3: Puyallup River- bed volume change (1984-2009)

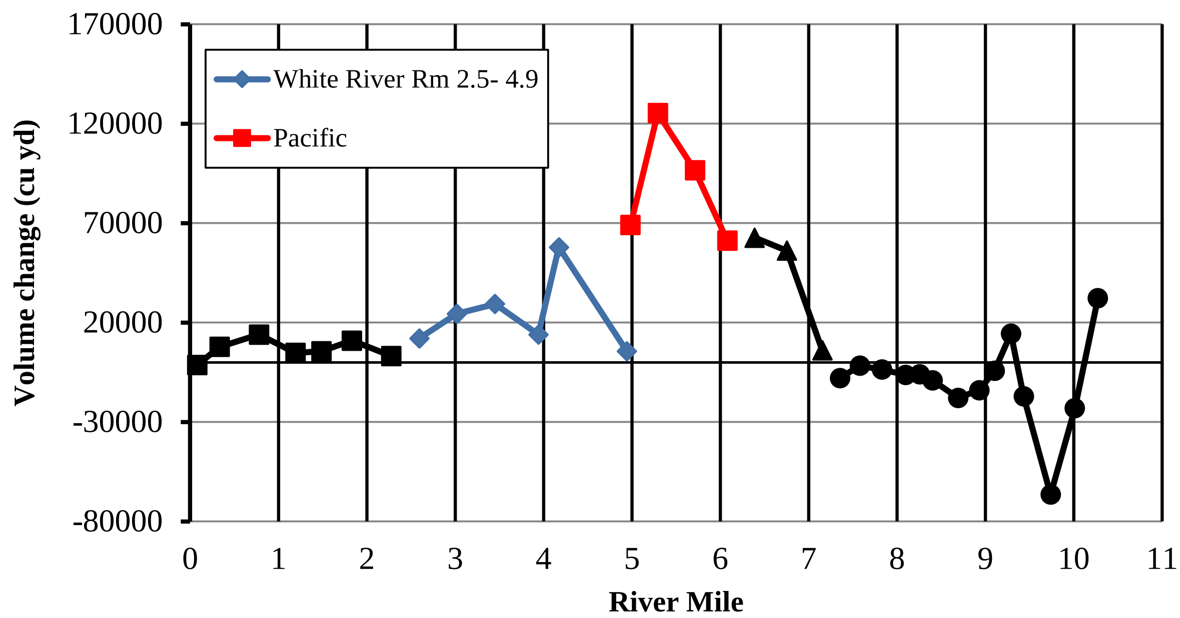

Figure 4: White River- bed volume change (1984-2009)

It should be noted that the Lower Puyallup USGS gage rating was observed to change between 2006 and 2009 for a similar discharge (lower stage in 2009). This suggests that the channel capacity at the gage location changed between flood events, and the reach may not be strictly depositional through time. The fine bed material at that location may be flushed out by these large events creating more channel capacity, however this remains to be verified.

Significant quantities of sediment dredged during the 1984-2009 period were added in by reach to the calculated volumes (USGS 2010). Dredged volumes are shown in Table 1 below and were verified by Pierce County (Motsenbocker, 2013). Only volumes that could be substantiated were included in the analysis. A large volume of over 250,000 cu yd. was initially thought to have been removed on the Upper Puyallup between RM 18-24 in 1984, but upon further investigation it was discovered that the dredging contract was never awarded and the work never performed.

Table 1: Significant Historical Dredged Quantities

Location	Year	Approx. Quantity (Cu. Yd.)
Upper Puyallup River RM 18-25	1987, 1990, 1991, 1994	190000
White River RM 5-5.5	1984, 1986, 1994	84000
White River RM 3.6-4.6	1986, 1987, 1994, 1998	28000

At this point in the study the primarily concerned is trends over leveed areas. The value in this approach comes from determination of average bed volume change over reaches that experience similar trends, rather than focusing in on any specific location. Estimated deposition from the approach discussed here is given in Table 2 below for several key locations in the basin.

Table 2: Estimated Future Deposition from Historical Trends

Location	Cu yd./yr.	Tons/yr.	Tons 50 yrs.	Deposition 50 yrs. (ft.)*
Lower Puyallup RM 0-3	4543	5704	285210	0.9
Lower Puyallup RM 3-7.6	11027	13845	692228	1.8
Lower Puyallup RM 7.6-8.4	3065	3849	192426	2.1
Middle Puyallup at SR 410 RM 11.5	2301	2889	144447	1.8
Upper Puyallup at the Calistoga levee RM 21	11427	14346	717308	4.7
Upper Puyallup at the Jones levee RM 22	10216	12826	641283	4.8
Upper Puyallup at the Ford levee RM 23	10216	12826	641283	4.0
White R. above A St. RM 6	3623	4549	227445	4.7
White R. at Pacific RM 5.5	17325	21751	1087559	3.2
White R. RM 2-4.8	6838	8585	429253	4.8

*Note that depth of deposition is estimated for a rectangular plane bed, actual depths in the model were based upon volume of deposition and varied. Specific weight of sediment is assumed to be 93 lb/cu ft.

From this analysis it was apparent where the most rapidly aggrading locations have historically been: The White River from RM 7-4.9, the Puyallup River from RM 24-20 and RM 10-9. It was interesting to note that the White River above R St. is in a degradational state and the Lower Carbon River near Orting is in a quasi-equilibrium state tending towards slight degradation. Reasons for this are explained in the USGS report (USGS, 2010). The approach taken here was to extrapolate the calculated historic deposition rates in cu. yd. /yr. into the future 50 years. The total 50 yr. volume over each reach that exhibited similar trends of aggradation was applied to the base geometry of the hydraulic model discussed previously. The channel modification tools available in the HEC-RAS program were used to input the calculated volumes. The channel modification tool fills from the lowest point in the cross section upwards, so there is not capability to manipulate where the sediment aggrades (i.e. such as on bends). This is not problematic, however, due to the one-dimensional nature of the modeling. For areas where levee setbacks were completed since 2007 or are expected to be completed in the next few years (Old Soldiers Home, Calistoga, and County line Levees) the total volume was spread over the new set-back width. It was assumed that levee setbacks open up the area behind the levee to active channel migration. Areas where setbacks are located (Calistoga and Levee on the Upper Puyallup and the Pacific area on the White) are already depositing all the available bedload supply, and may cause an increase in deposition of fines. This will be better quantified during later phases of sediment modeling. The result of applying the estimated deposition was typically similar to what is shown in Figure 5 below.

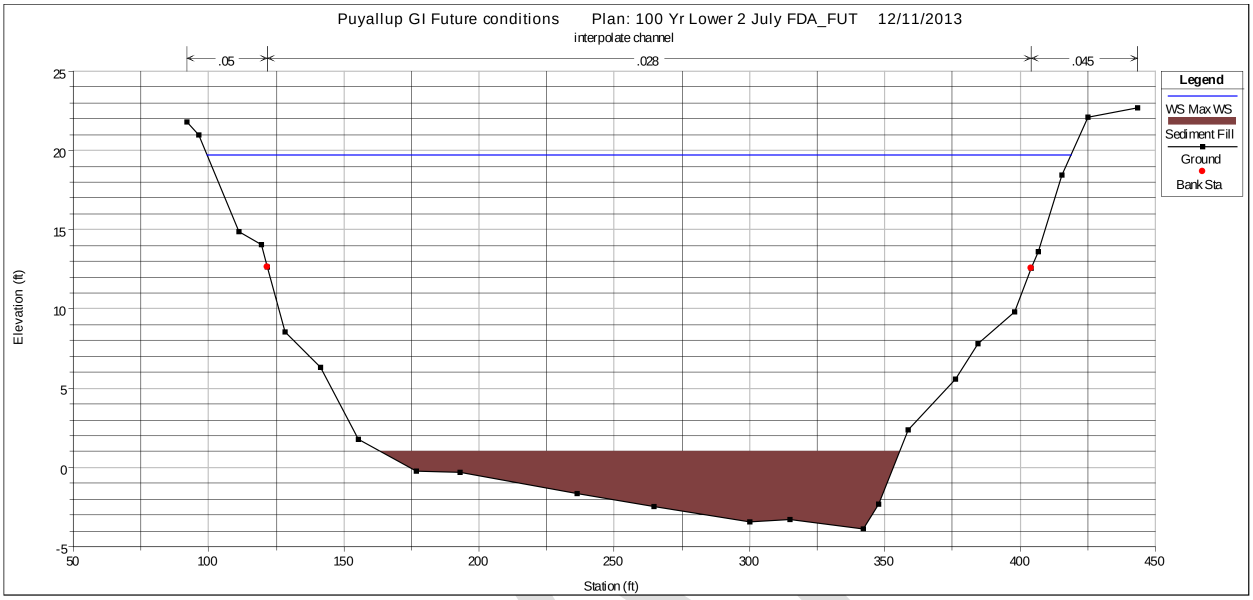

Figure 5: Typical Deposition Applied to Model Geometry

Once the channel bed was updated in the model geometry, the hydraulic model was run for 50-0.2% AEP hydrology to determine resulting water surface change over the project life. Estimated water surface change for a 1% AEP event is shown in Table 3 below. This same overall approach was applied for developing future with-project conditions.

Table 3: Estimated Future Water Surface Change from Historical Deposition Trends

Location	1% AEP event WSE change (ft.)
Lower Puyallup RM 0-3	0.9
Lower Puyallup RM 3-7.6	1.7
Lower Puyallup RM 7.6-8.4	1.9
Middle Puyallup at SR 410	1.3
Upper Puyallup at the Calistoga Levee (future levee setback)	3.4
Upper Puyallup at the Jones Levee (2009 levee setback area)	3.3
Upper Puyallup at the Ford Levee	3.5
White R. above A St.	2.9
White R. at Pacific (future levee setback area)	1.0
White R. RM 2-4.8	0.8

3.2 State of Existing Data for Sediment Modeling of Future Conditions

A sediment model can be calibrated to changes that occur over a period time (i.e. input bed gradations, transport rates, and cross sections at a starting time with the goal of matching data from a later point in time). Several data sources exist for information needed to perform sediment modeling:

- 1984 basin-wide cross sections (Prych, 1988)
- 2009 basin wide cross sections (USGS, 2010)
- 2012 White River cross sections (West, 2013)
- 1990 sediment (suspended and bed) load samples for all reaches (Sikonia, 1990)
- 2011 White R sediment (suspended and bed) load samples (USGS, 2011)
- 1978-1994 Lower Puyallup R. suspended load data (NHC, 2008)
- 1990 bed gradation samples (Sikonia, 1990)
- 2009 bed gradation samples (USGS, 2010)
- Additional sediment load data available from the USGS website

Several approaches were proposed to model future conditions throughout the basin including: build a 1984 era model and calibrate to 2009 (and then use the same transport parameters to run the 2009 model with updated sediment load data as it is collected); model critical areas with localized calibration and expand the model as additional data is collected; do a largely uncalibrated computational analysis for relative comparison between measures. Calibrating a sediment model to incoming load, bed volume change, and bed gradation is generally the most accurate approach. Ultimately the USACE Committee on Channel Stabilization was consulted on this topic. It was decided to accomplish sediment modeling prior to completion of the final feasibility study. More information on their recommendations is presented in section 6.0.

4.0 Future With-out Project Conditions Hydraulic Modeling

The hydraulic model geometry was updated following the methodology outlined above with estimated channel bed change and run for 50-0.2% AEP events. The remaining geometry was left as developed for the existing conditions model. The downstream model boundary was updated to reflect the estimated sea level rise of one foot discussed previously. The 1% AEP estimates for future without-project and the existing without-project are shown in the figures below. Essentially the difference between these two conditions shows flooding due to sediment deposition.

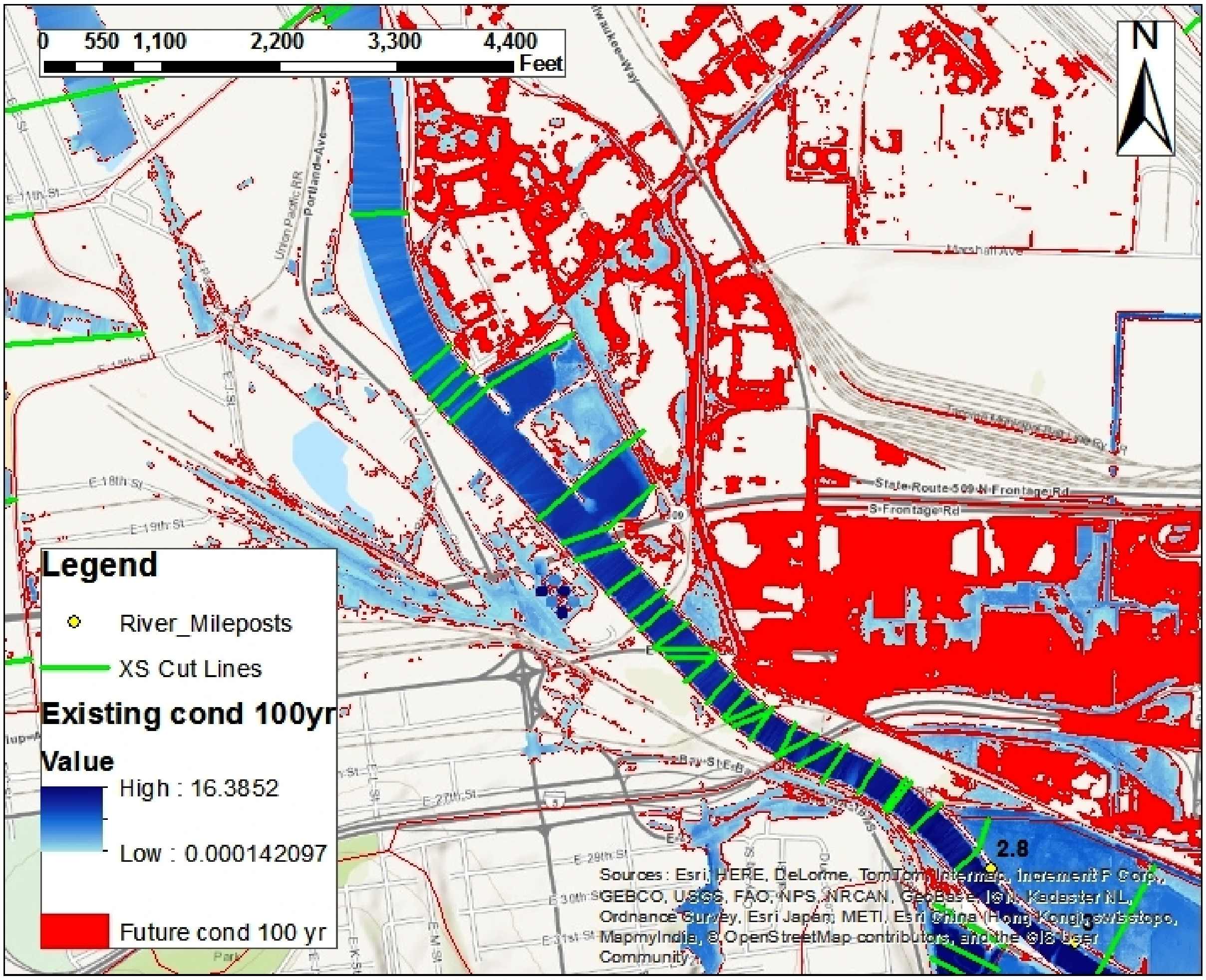

Figure 6: Lower Puyallup River below RM 3

The Lower Puyallup River below RM 2 experiences tidal influence along the left and right bank federal levees. This area generally has less than 2 ft. of residual levee height for a 1% AEP event, neither of the federal levees overtops, and flooding is from overtopping of non-federal levees slightly upstream. Sea level change is modeled in this figure; however it did not result in additional overtopping from the existing condition.

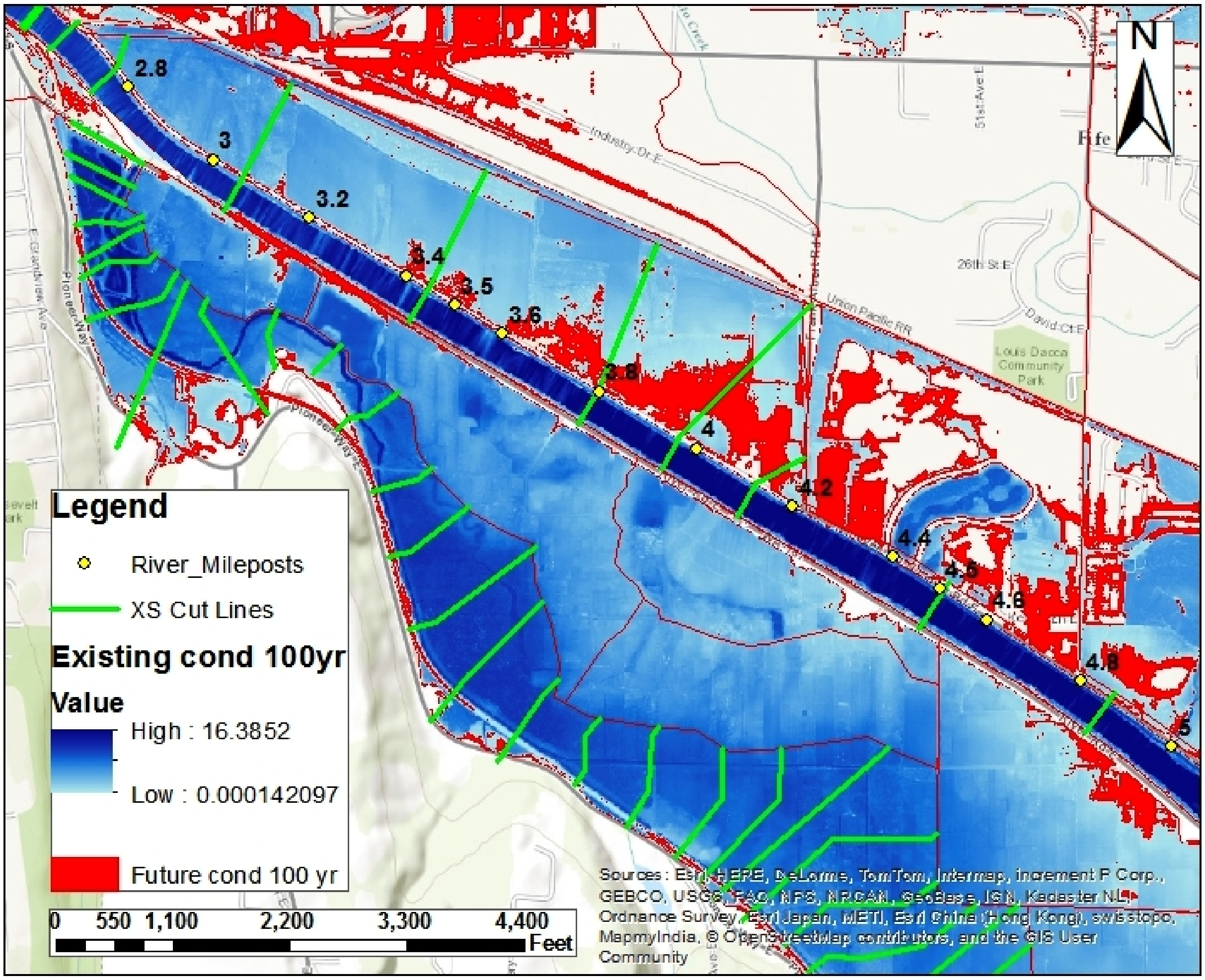

Figure 7: Lower Puyallup River from RM 2.8-5.2 including Clear Creek

The Clear Creek outlet on the left bank of the Lower Puyallup at RM 2.9 is gated and closed to prevent backwater. Interior flows are impounded at the outlet during flood events until river flows drop and the gates can be opened to release stored water. Impounded flood water has caused significant inundation of developed areas during the last few large flood events. The leveed area behind North Levee Rd. on the right bank and River Rd. on the left bank are overtopped in areas by a 1% AEP event.

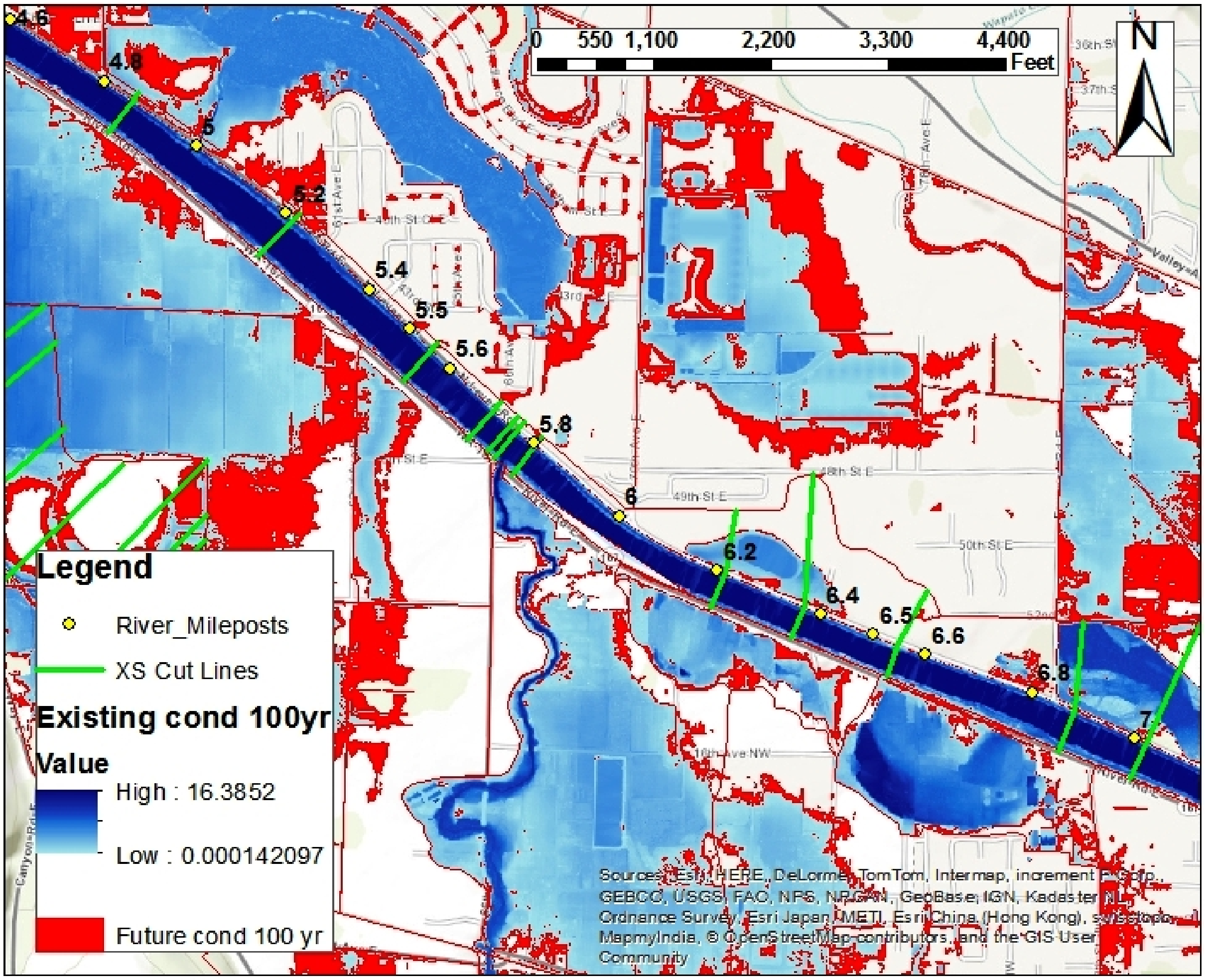

Figure 8: Lower Puyallup River from RM 4.6-7 including Clarks Creek

The outlet of Clarks Creek is free-flowing through the leveed left bank of the lower Puyallup at RM 5.8. The creek experiences increased backwatering. The leveed area behind North Levee Rd. on the right bank and River Rd. on the left bank are overtopped in areas by a 1% AEP event.

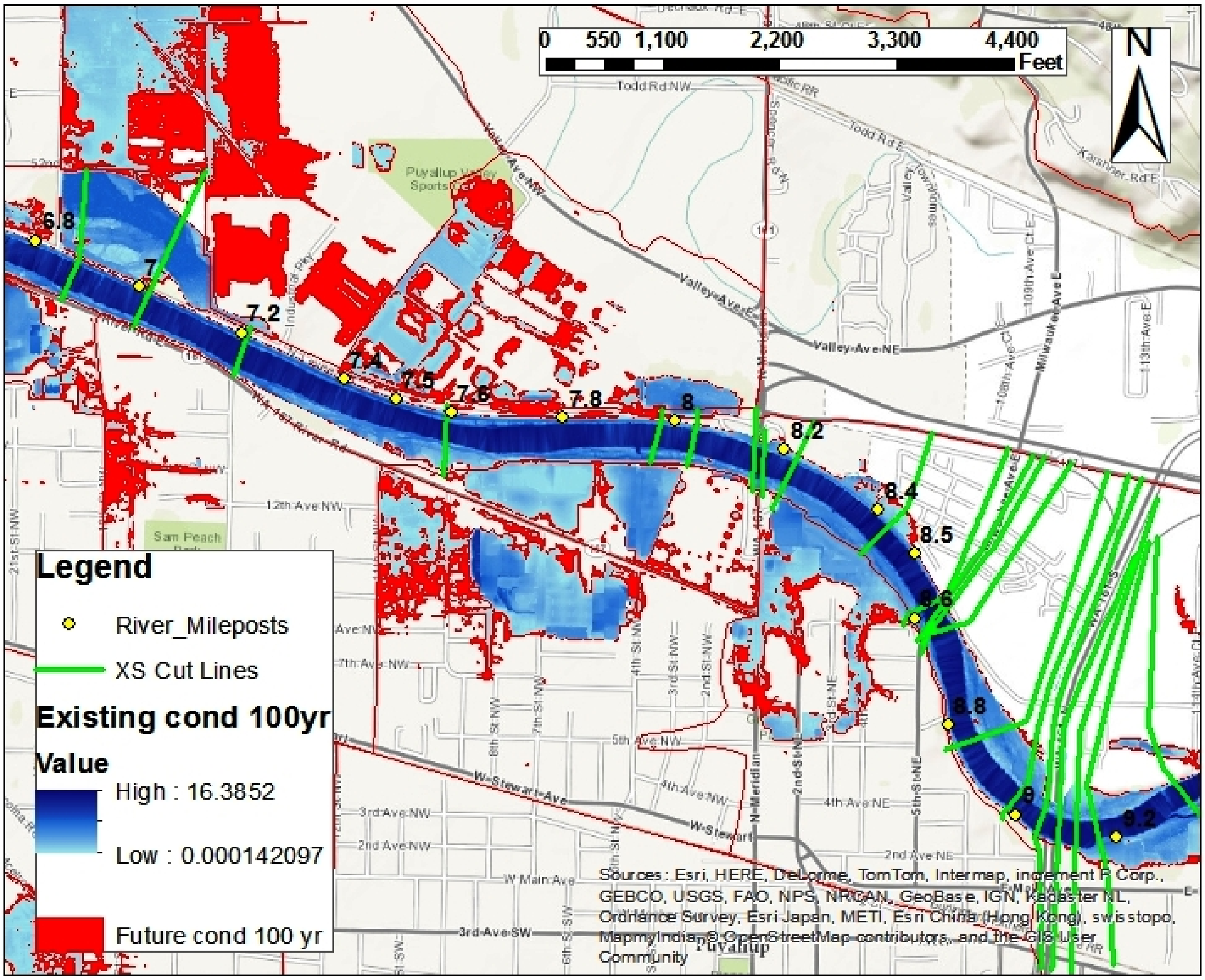

Figure 9: Lower Puyallup River from RM 6.5-9.2

The leveed area behind North Levee Rd. on the right bank and River Rd. on the left bank are overtopped in areas by a 1% AEP event. No levee presently exists from RM 8.2 to 9 and the left bank has been overtopped by the last few major flood events. The right bank is high terrain and well above the 0.2% AEP event.

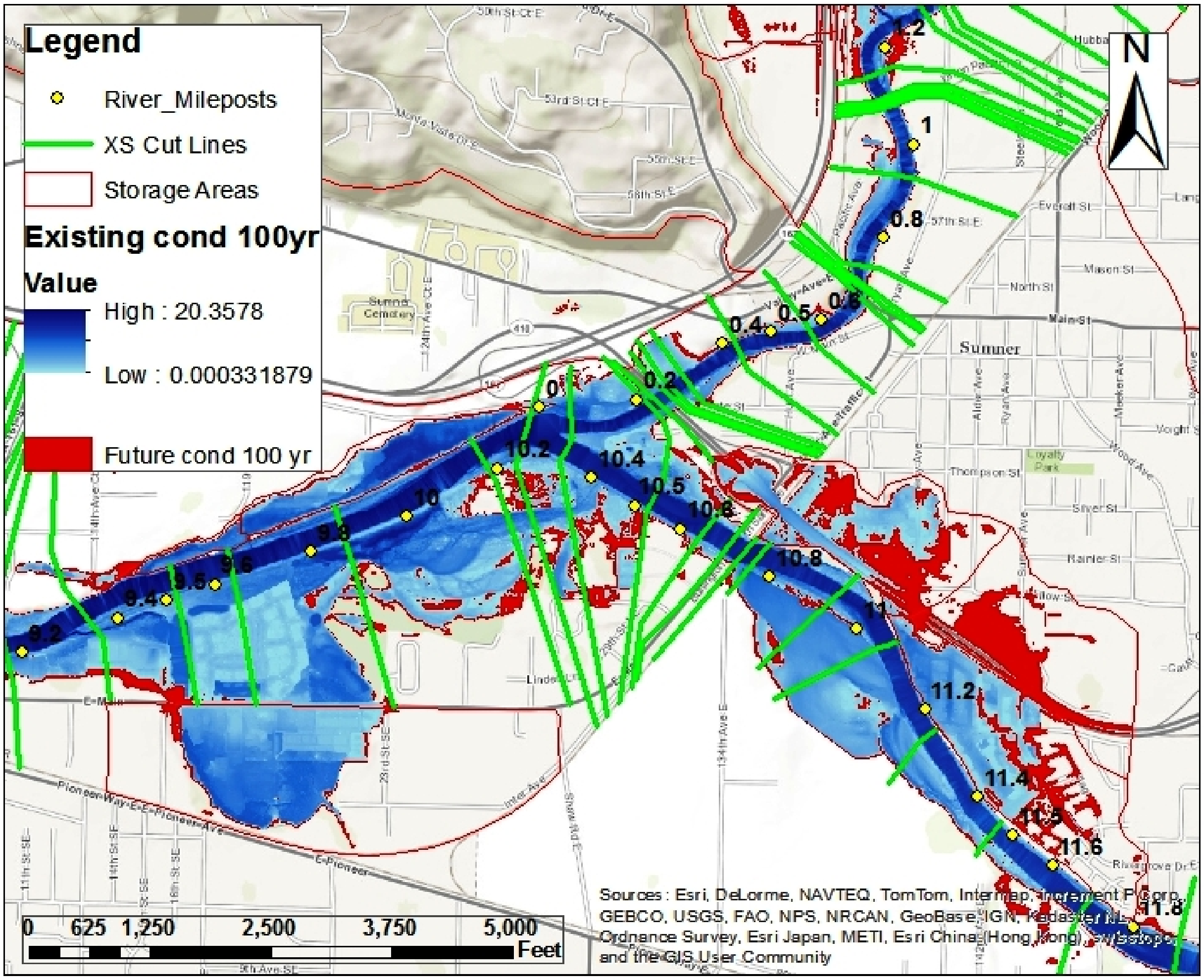

Figure 10: Middle Puyallup River at junction with the White River

The Middle Puyallup is estimated to experience heavy deposition, but is isolated by high terrain from the interior area of Sumner. Development is sparse in flood impacted areas of the Lower Puyallup from RM 8.4-10.2 and the lower mile of the White River is deep with high banks. The Middle Puyallup is prone to frequent flooding from RM 10.8-11.5 along the right bank developed area; the left bank is not developed.

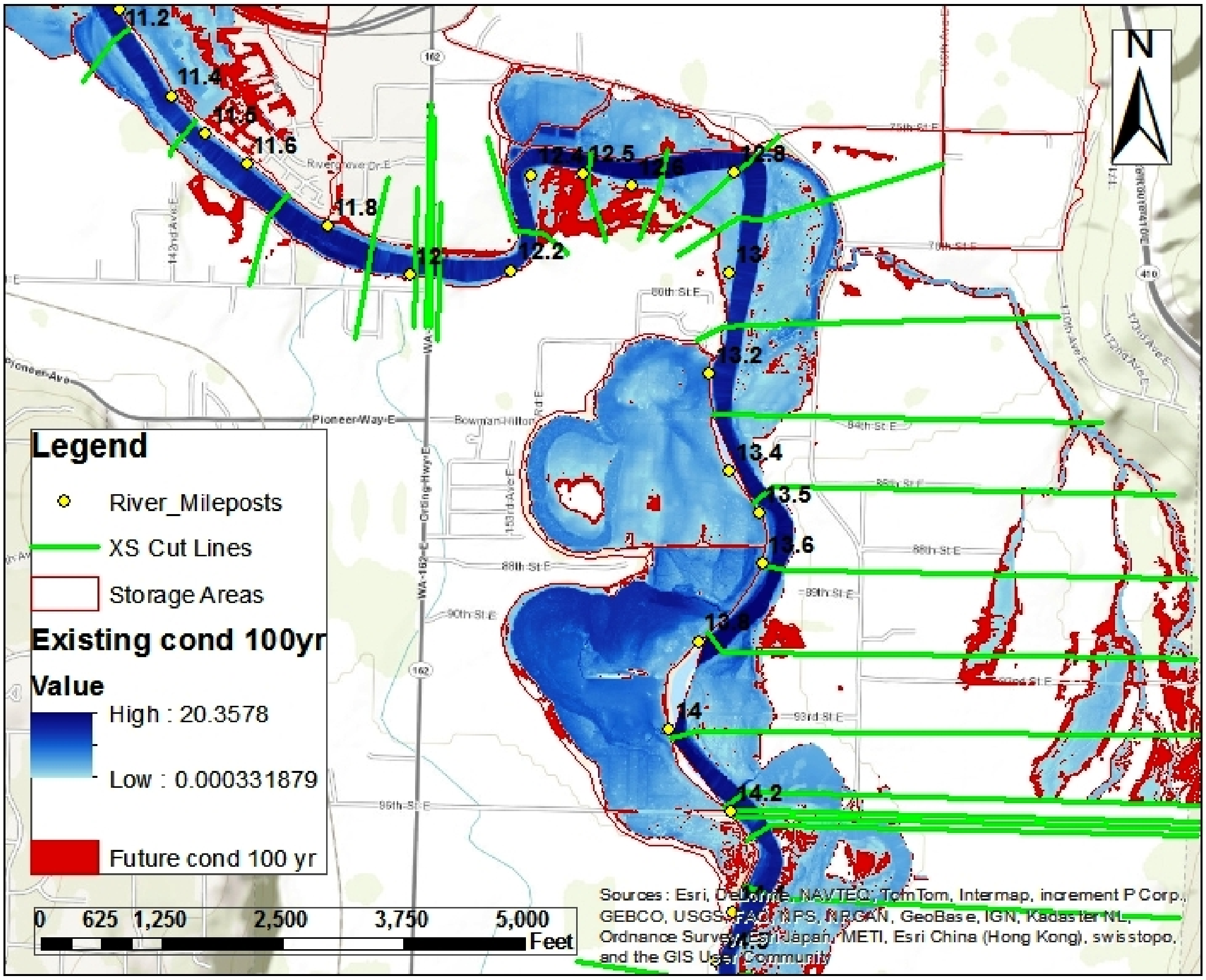

Figure 11: Middle Puyallup River from RM 11.4-14.4

The majority of floodplain through the Middle Puyallup from 11.4-14.4 is sparsely populated and consists of local levies and abandoned meander belts. The local levies typically have less than a 1% AEP level of protection.

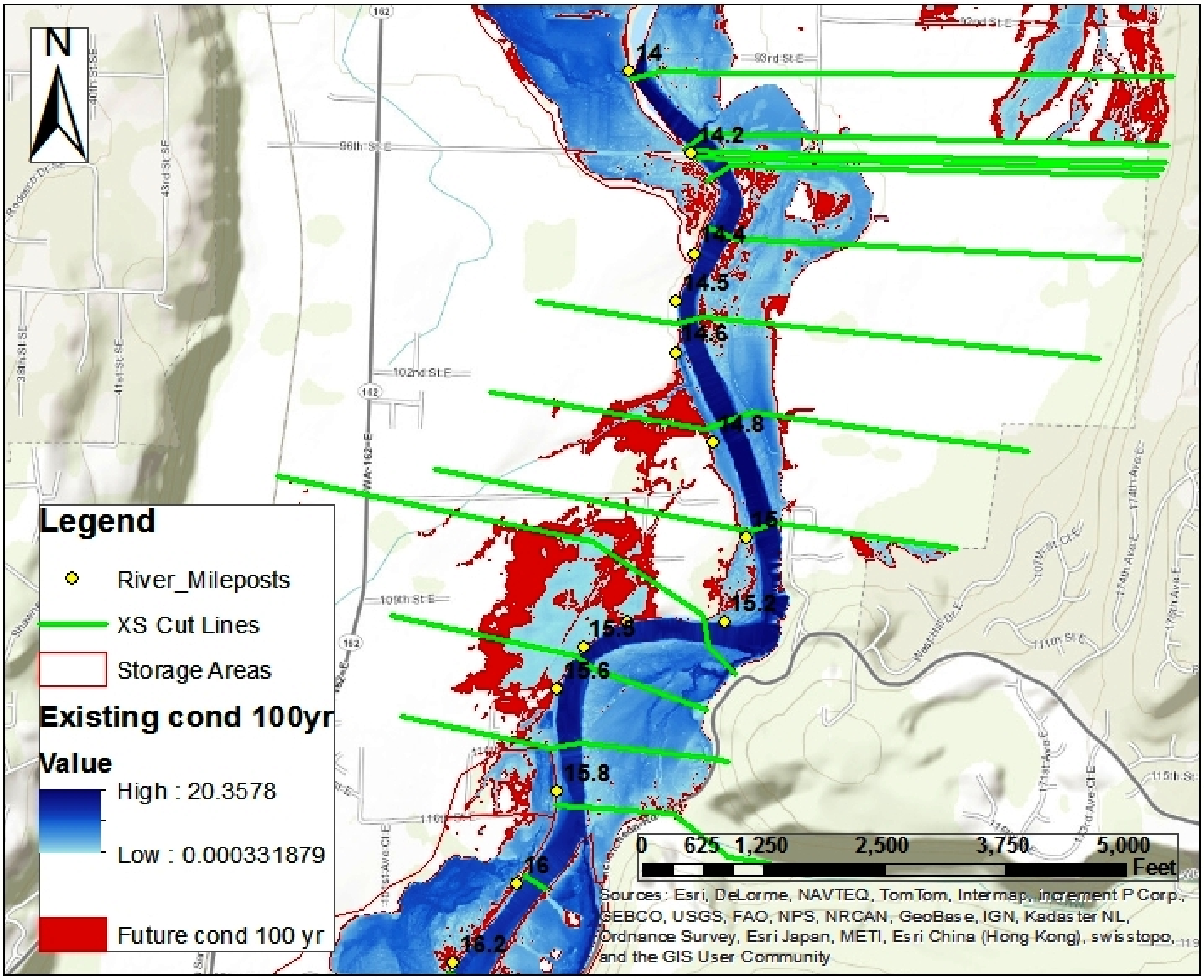

Figure 12: Middle Puyallup River from RM 14-16

The majority of floodplain through the Middle Puyallup from RM 14-16 is sparsely populated and consists of local levees and abandoned meander belts. The local levies typically have less than a 1% AEP level of protection.

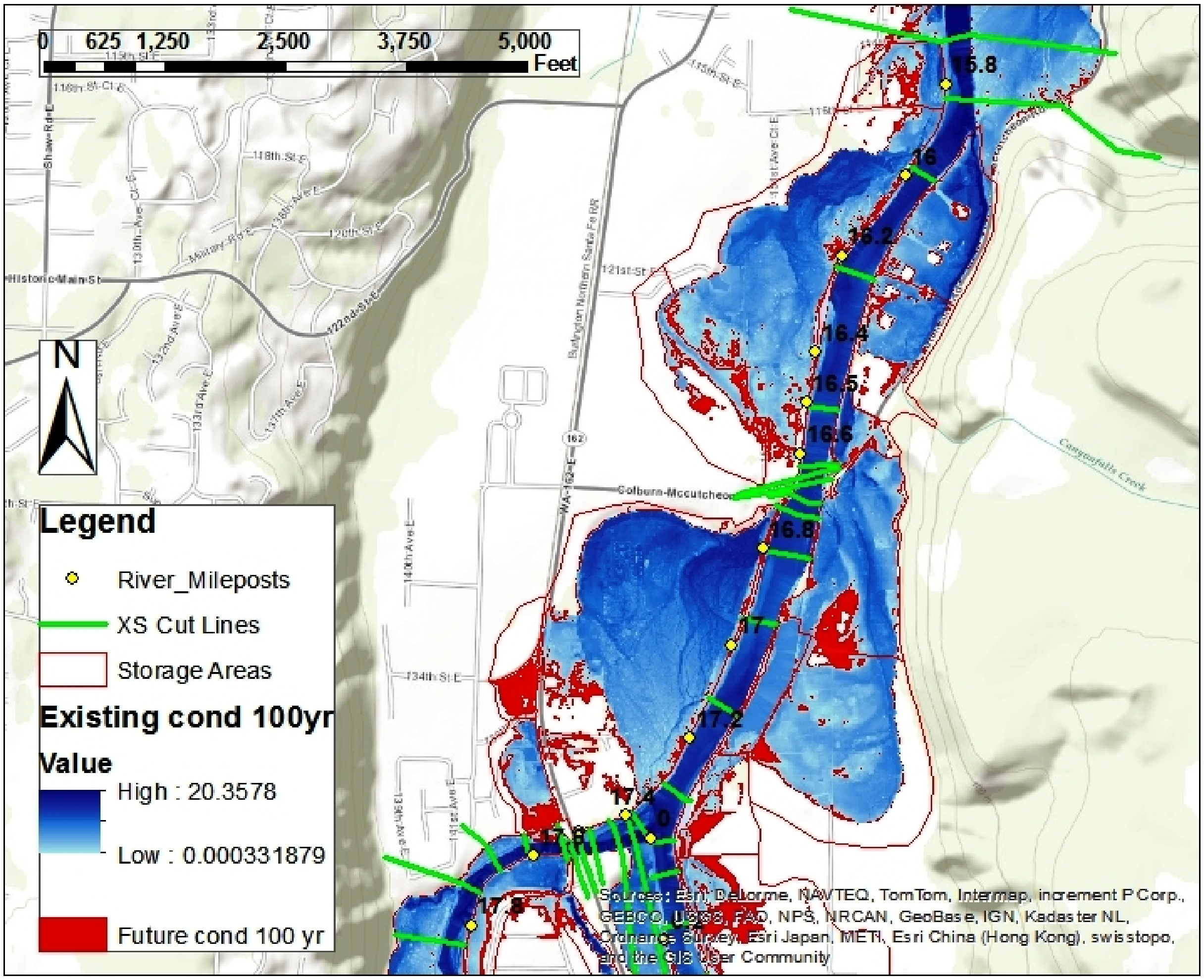

Figure 13: Middle Puyallup River from RM 15.8-17.4

The majority of floodplain through the Middle Puyallup from RM 15.8-17.4 is sparsely populated and consists of local levees and abandoned meander belts. The local levees typically have less than a 1% AEP level of protection.

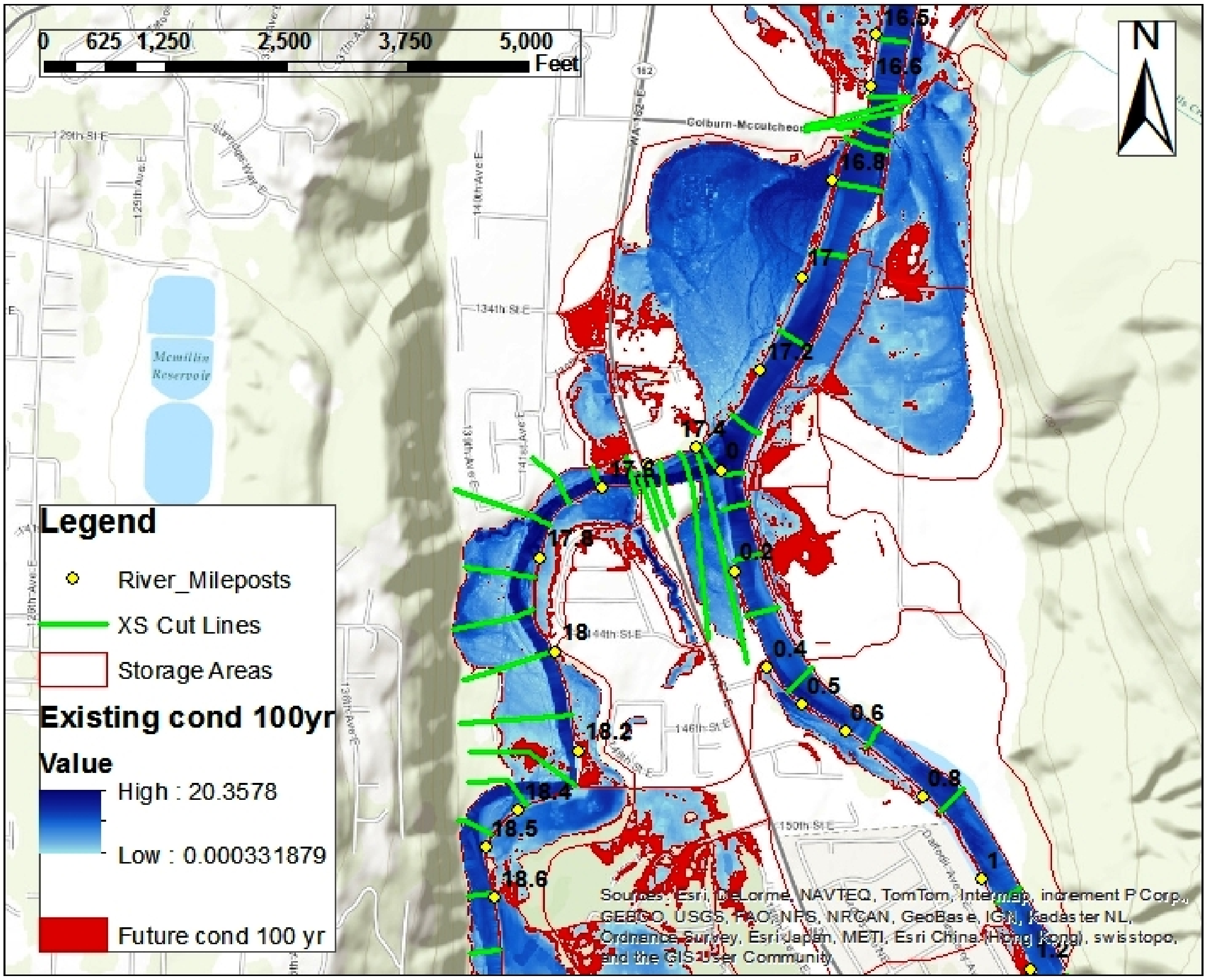

Figure 14: Upper Puyallup River at junction with the Carbon River

The area of Orting immediately south of the confluence is located on high ground and not prone to flooding. At RM 17.6 the left bank of the Upper Puyallup is at risk of overtopping at a 1% AEP event.

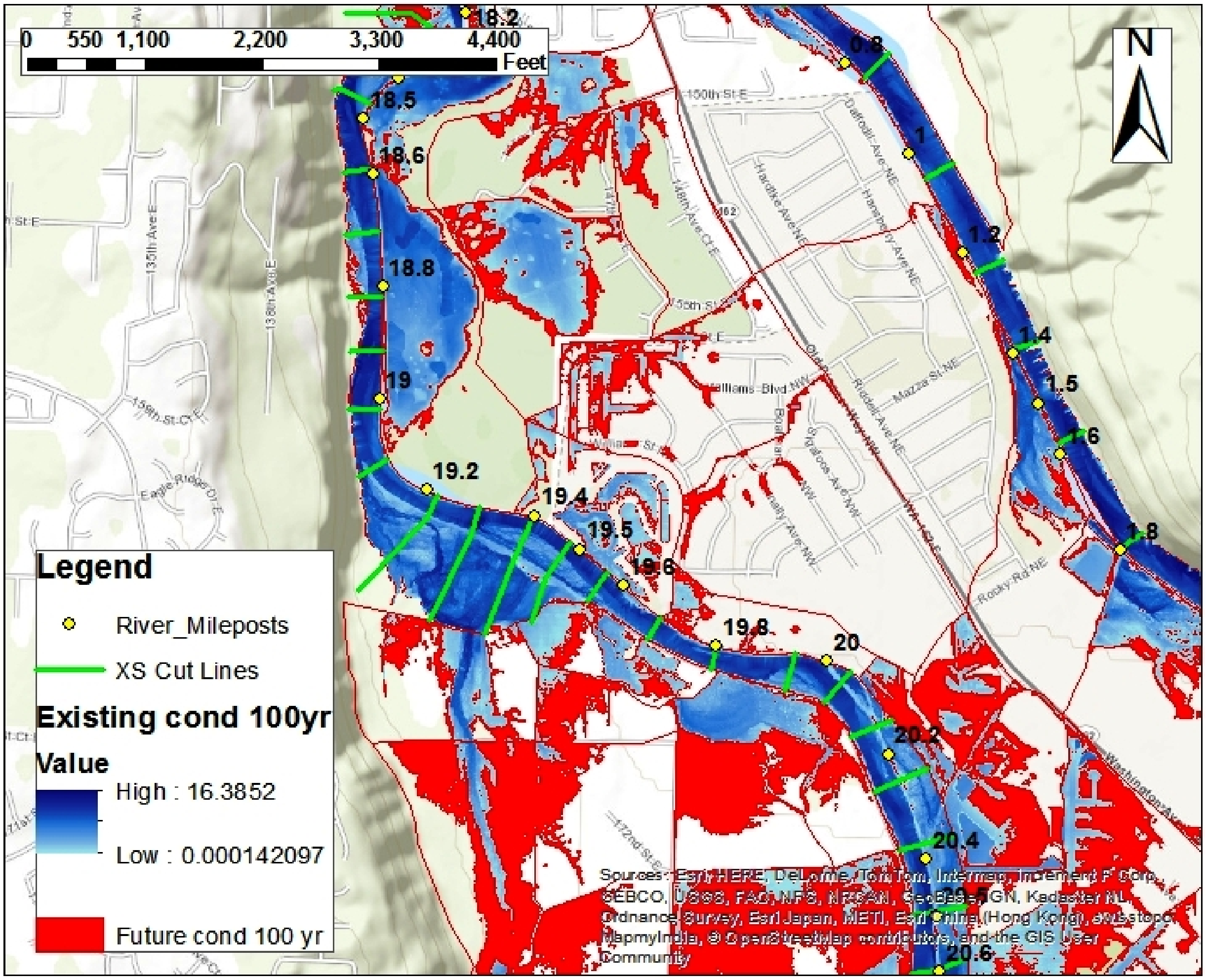

Figure 15: Orting area between the Upper Puyallup and Carbon Rivers

The area through central Orting is prone to flooding from overtopping anywhere along the right bank from RM 18.2-22.5. Overtopping of levees along both sides of the river is significantly increased from estimated deposition. The floodplain land slope is roughly equal to the riverbed slope and severe overtopping from larger events (1-0.2% AEP) would tend to flow behind the levees and pool in central Orting. The High Cedars golf course is located along the right bank from RM 18.4-19.3 where the levee has a level of protection less than 1% AEP.

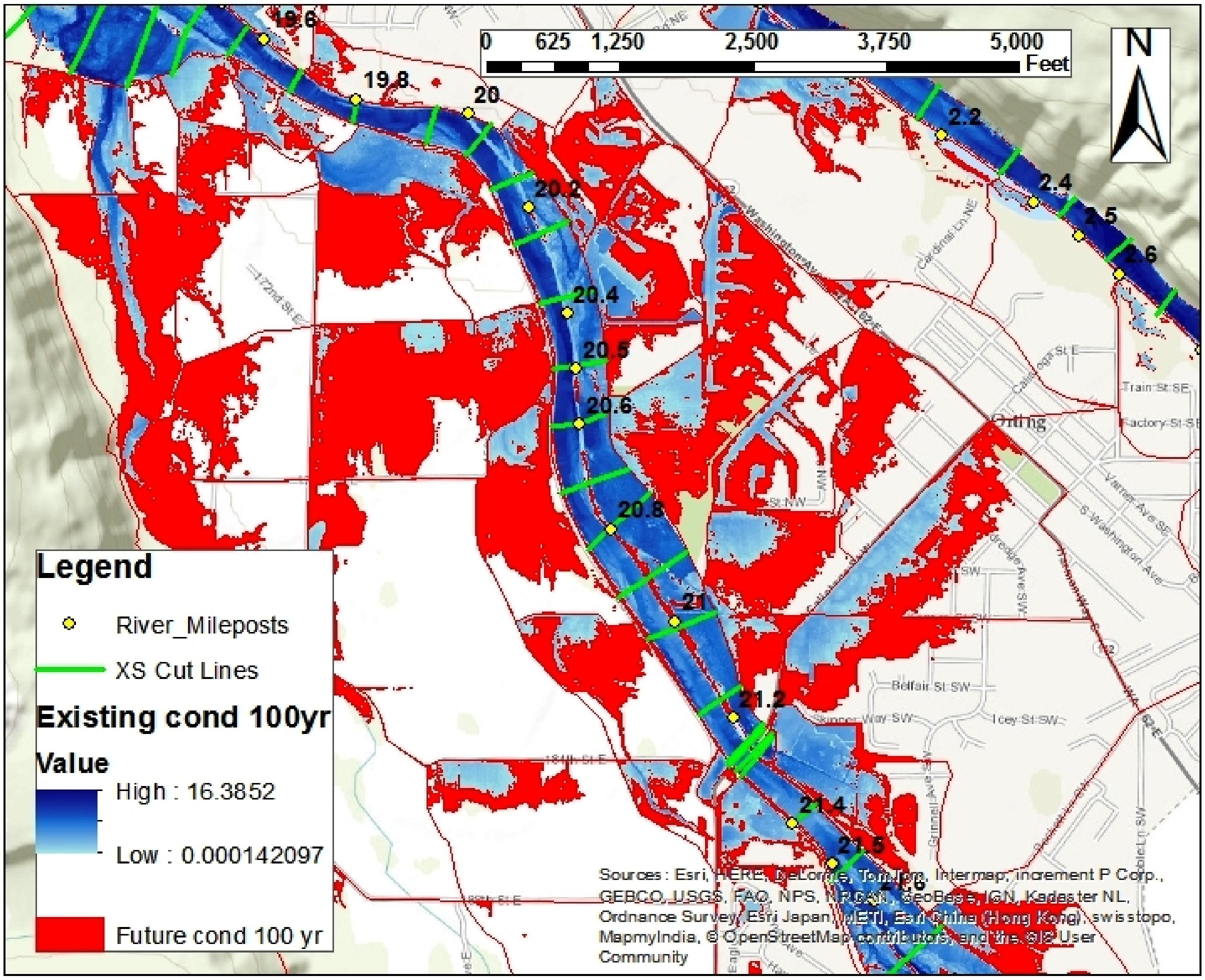

Figure 16: Orting area between the Upper Puyallup and Carbon Rivers

The area through central Orting is prone to flooding from overtopping anywhere along the right bank from RM 18.2-22.5. Overtopping of levees along both sides of the river is significantly increased from estimated deposition. The floodplain land slope is roughly equal to the riverbed slope and severe overtopping from larger events (1-0.2% AEP) would tend to flow behind the levees and pool in central Orting. The Calistoga Levee along the right bank from RM 19.8-21.3 is incorporated into the model as a setback and is scheduled to be built by Pierce County over the next few years. This levee will have a height of the 1% AEP water surface plus 4 ft. of residual levee height, which is greatly reduced from estimated deposition. Calistoga Bridge at RM 21.3 has experienced deposition over the last few large flood events. The left bank is composed of agricultural levees with a level of protection less than 1% AEP event. Located along the west Orting valley wall is a large abandoned braid that at one time was connected to the Upper Puyallup around RM 22-23. This abandoned braid is presently a small creek and is incised to a significantly lower elevation than the main river. Backwatering of the creek is increased by estimated deposition. Central Orting is also low and main Upper Puyallup River becomes more and more perched above the surrounding valley due to deposition between levees.

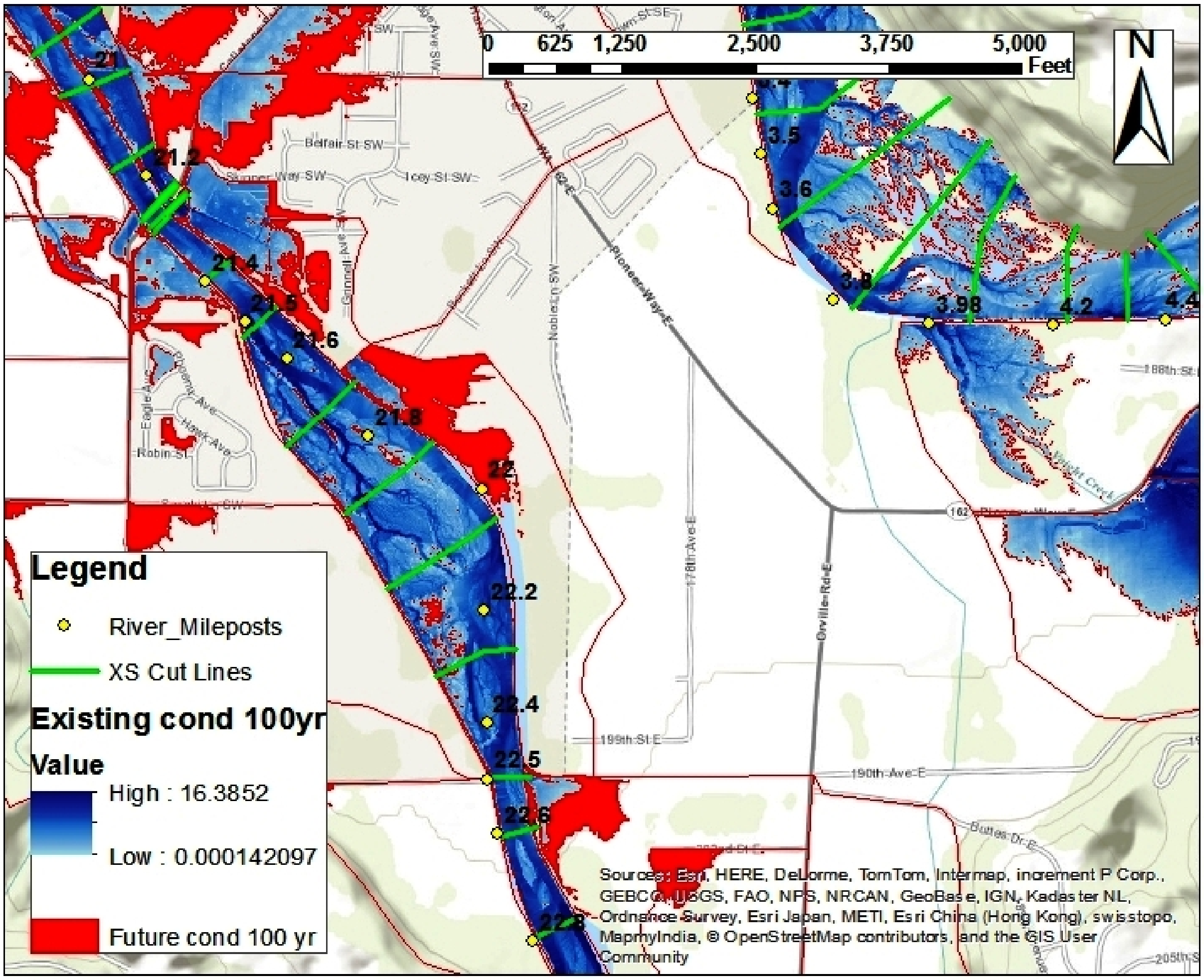

Figure 17: Orting area between the Upper Puyallup and Carbon Rivers

The area through central Orting is prone to flooding from overtopping anywhere along the right bank from RM 18.2-22.5. The floodplain land slope is roughly equal to the riverbed slope and severe overtopping from larger events (1-0.2% AEP) would tend to flow behind the levees and pool in central Orting. The only levee that is presently taller than the 1% AEP water surface is the Ford levee from RM 22.5-24. The lower end of Ford overtops significantly more from estimated channel deposition. The pre-1997 levee at the Ford location failed and flooded a significant area of central Orting. That levee was subsequently replaced with the current levee configuration.

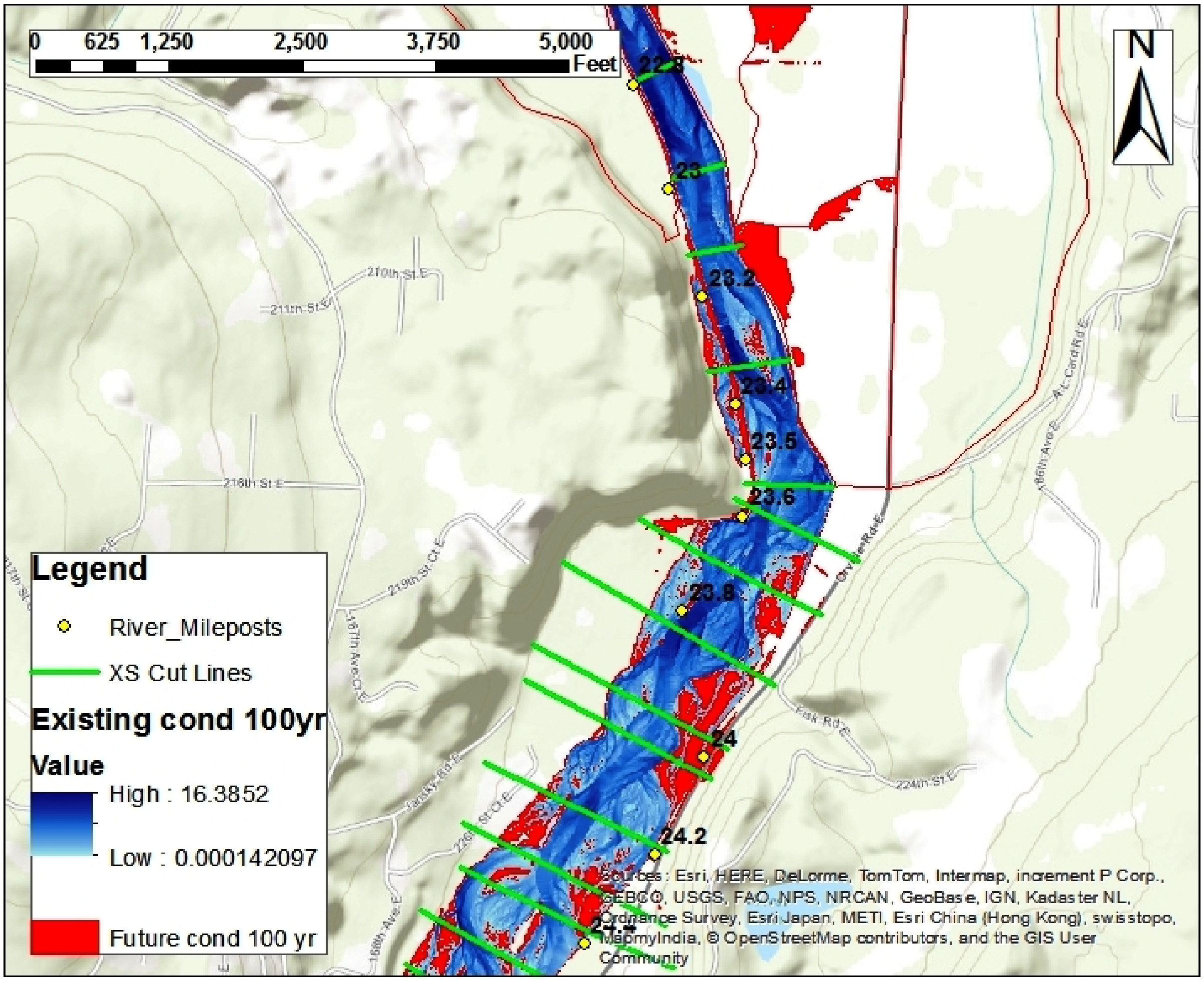

Figure 18: Upper Puyallup River from RM 22.5-24.2

The Upper Puyallup River above RM 24 is very sparsely developed, and consists of a high energy, steep, and braided reach that frequently causes significant damage to local levees. Levees along both sides of the river from RM 25-24 (upstream of the bend at the ford levee) were destroyed by the 2006 flood and abandoned, resulting in channel migration at RM 23.6. Migration has eroded the vegetated buffer in front of the Ford Levee causing significant damage to the levee rip rap. About 250 ft. of the Ford levee was repaired in 2009 with class 5 rock. This area at RM 22.5-23.5 is a depositional zone with a significant break in bed slope.

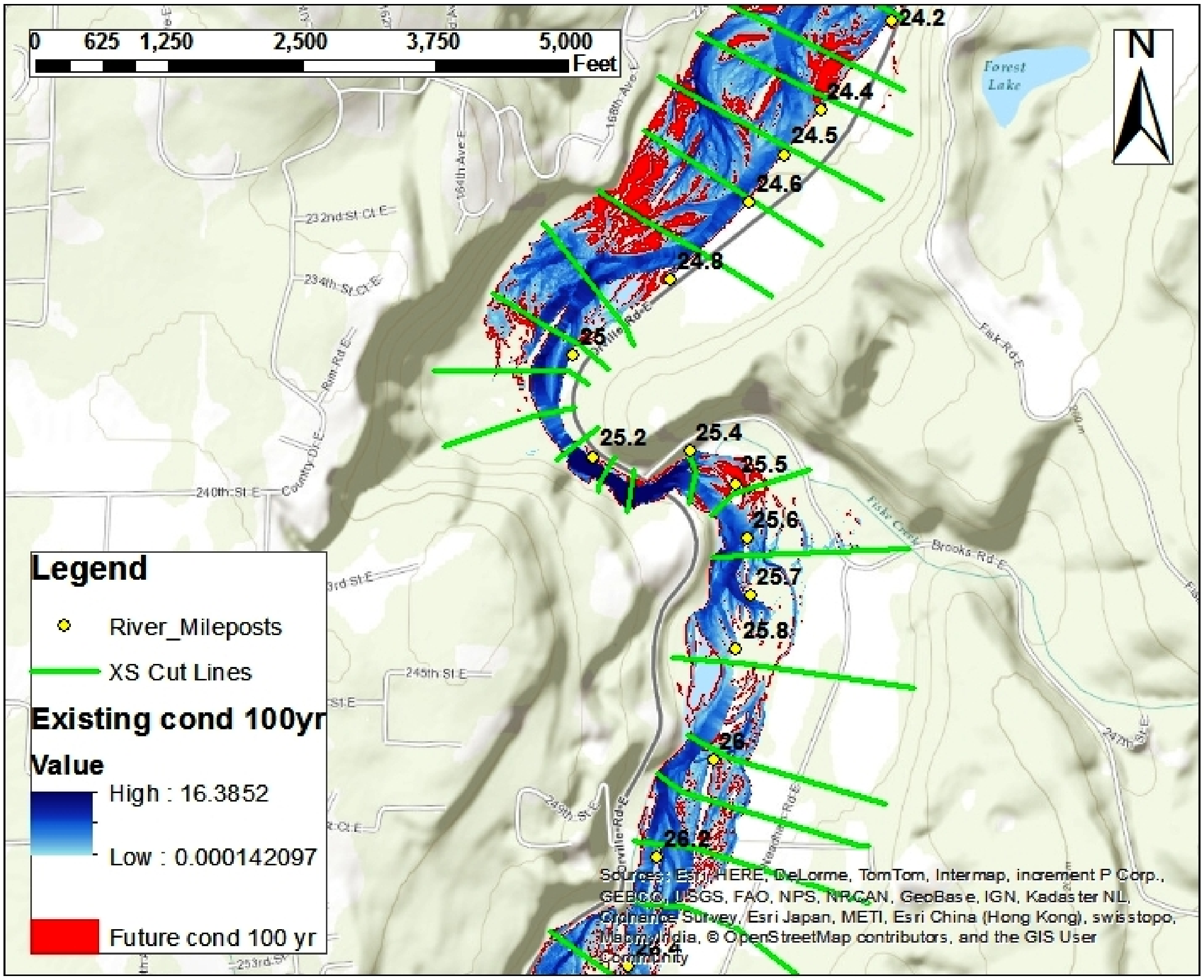

Figure 19: Upper Puyallup River from RM 24.2 to 26.4

The Needham Rd. Levee is located along the right bank at RM 24.7 and has experienced significant repeated damage over the last decade. Pierce County has recently completed a major modification to this bank to prevent erosion and channel migration. The only USGS gage on the Upper Puyallup near Orting is located at RM 25.2. The right bank from RM 25.2-26 is sparsely developed and prone to flooding at very large events (0.5-0.2% AEP).

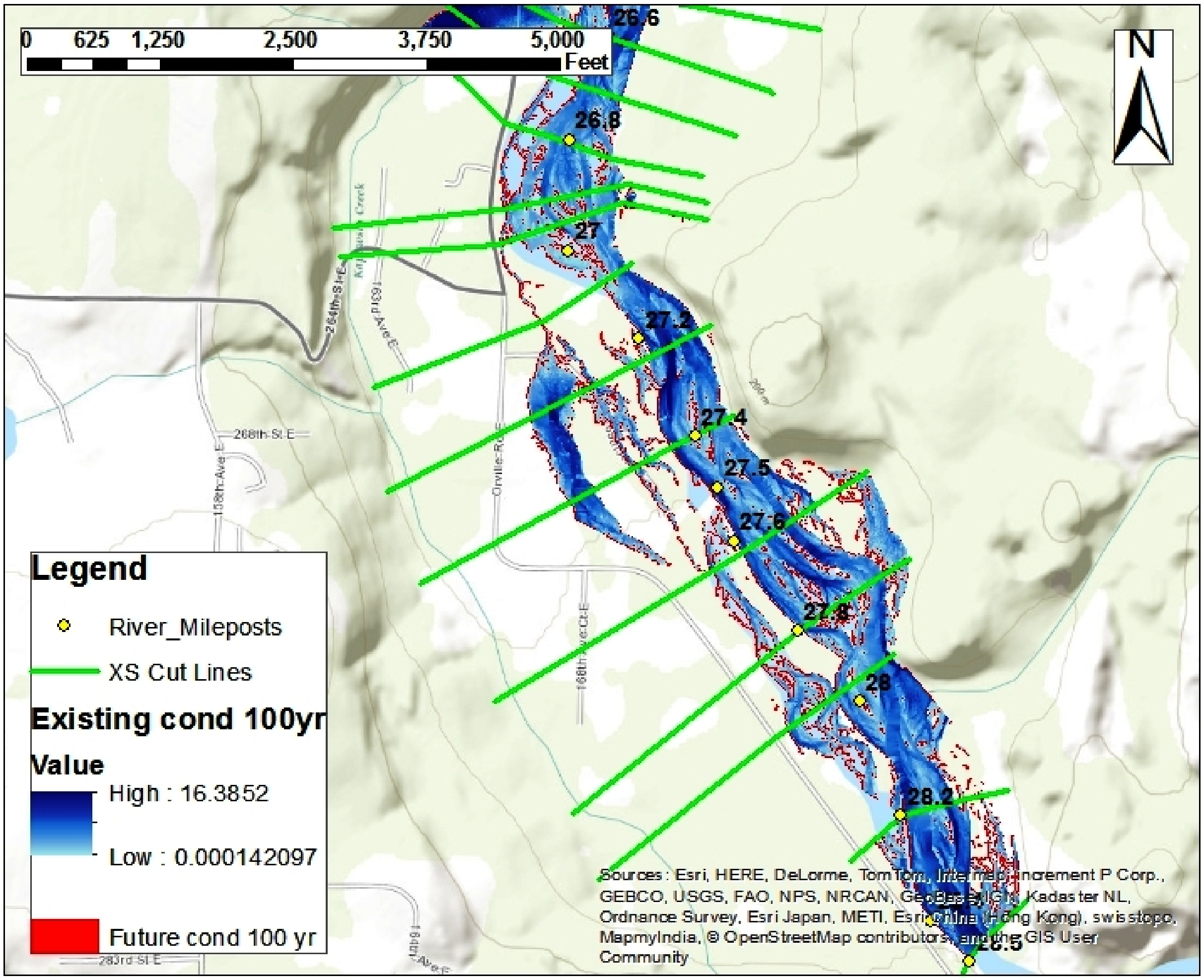

Figure 20: Upper Puyallup from RM 26.5-28.5

The Upper Puyallup in this area is not estimated to experience significant deposition. The left bank from RM 27-27.8 is the Orville Rd East road embankment and has experienced significant damage over the last decade. Pierce County has recently completed the first phase of a project to reduce erosion damage and channel migration. Kapowsin Creek comes in at RM 26.5 and can backwater from the main river.

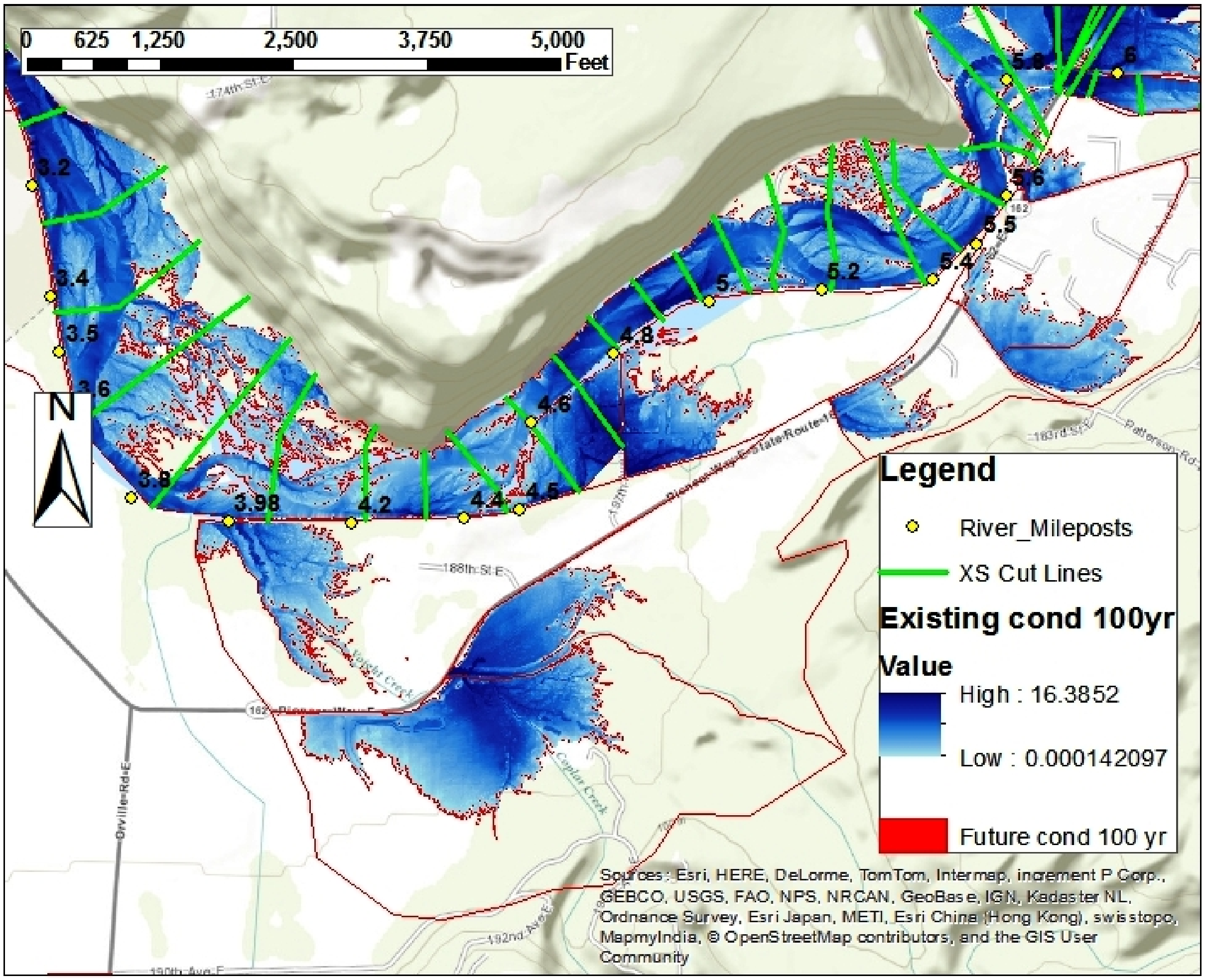

Figure 21: Carbon River from RM 3.2-6.2

The left floodplain along the Carbon River from RM 3.8-5.9 is sparsely developed. Voight Creek comes in at RM 3.9 and can backwater from the main river. The left bank from RM 3.9 to 3.5 has experienced significant erosion over the last decade as the river migrates into it. A major slope break occurs around RM 3.8 and is a depositional zone that has caused the river to migrate into the left bank levee. The Guy West levee along the left bank from RM 4.8-5.6 is higher than the 0.2% AEP water surface. South Prairie creek comes in at RM 5.9 and contributes significant flow to the Carbon. Flooding up South Prairie Creek is minor because its large valley was carved out by the White River thousands of years ago, and the present South Prairie Creek flow is significantly less. Little change is expected from deposition on the Lower Carbon River.

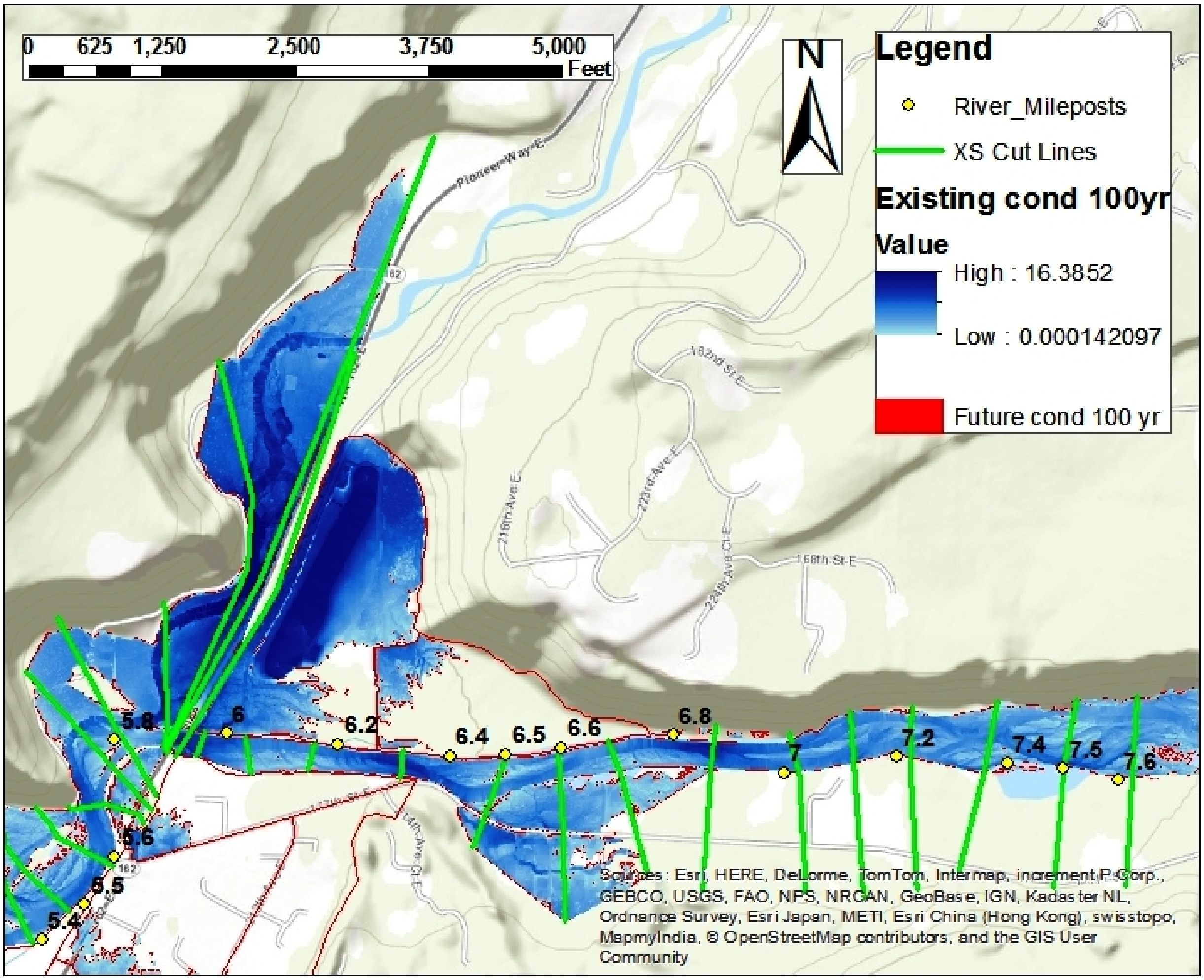

Figure 22: Carbon River from RM 5.4-7.8

The Carbon River above RM 6 is a high energy, steep, braided reach that frequently causes major damage to the left bank levee from RM 6.4-8. The Hwy 162 Bridge at RM 5.9 has experienced significant deposition and is now a constriction that causes flooding just upstream through drainage openings in the levees on both sides of the river. The local levees in this area have less than a 1% AEP level of protection. It is conceivable that a major failure of the left bank levee from RM 6-6.2 could send flowing water through the town of Crocker and into Voight Creek and on through to the Carbon at RM 3.9. A large lake is located on the right bank behind the levee at RM 6.1 that is capable of absorbing some degree of right bank flooding. Preliminary modeling has shown that South Prairie Creek is capable of flooding across Hwy 162 into the lake. Little change is expected from deposition on the Lower Carbon River.

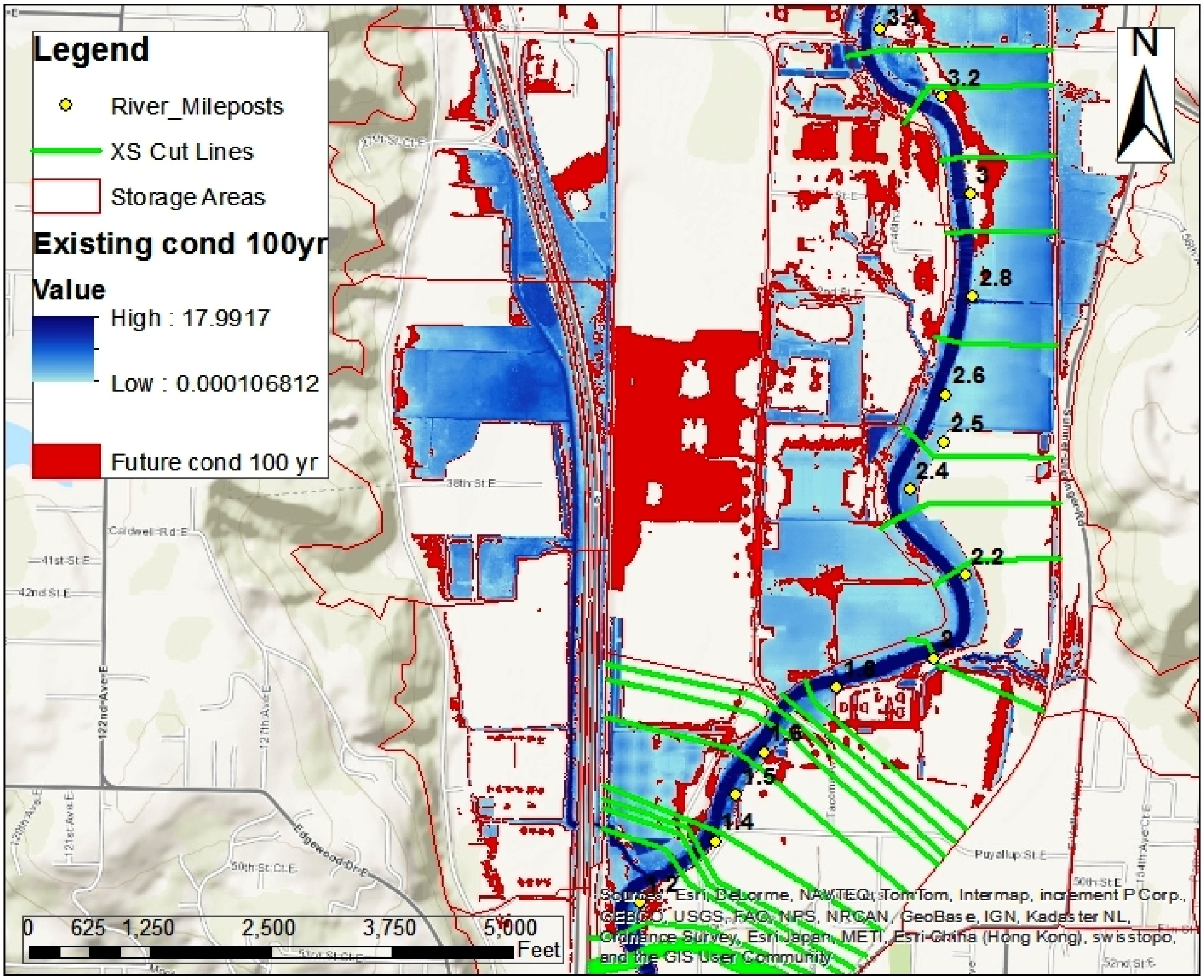

Figure 23: White River from RM 1 to 3.4

The Lower White River is heavily backwatered from the Puyallup and is a depositional zone for sand and fine sediment. The left bank is high terrain well above the 1% AEP floodplain up to RM 2.5. The left bank from RM 2.5-4.2 is agricultural land and a golf course. The right bank is commercial development and prone to flooding. Backwater occurs along the west side of Hwy 167 up the drainage ditch a great distance. Flooding is increased due to estimated deposition in the entire lower river through the area of the ditch outlet.

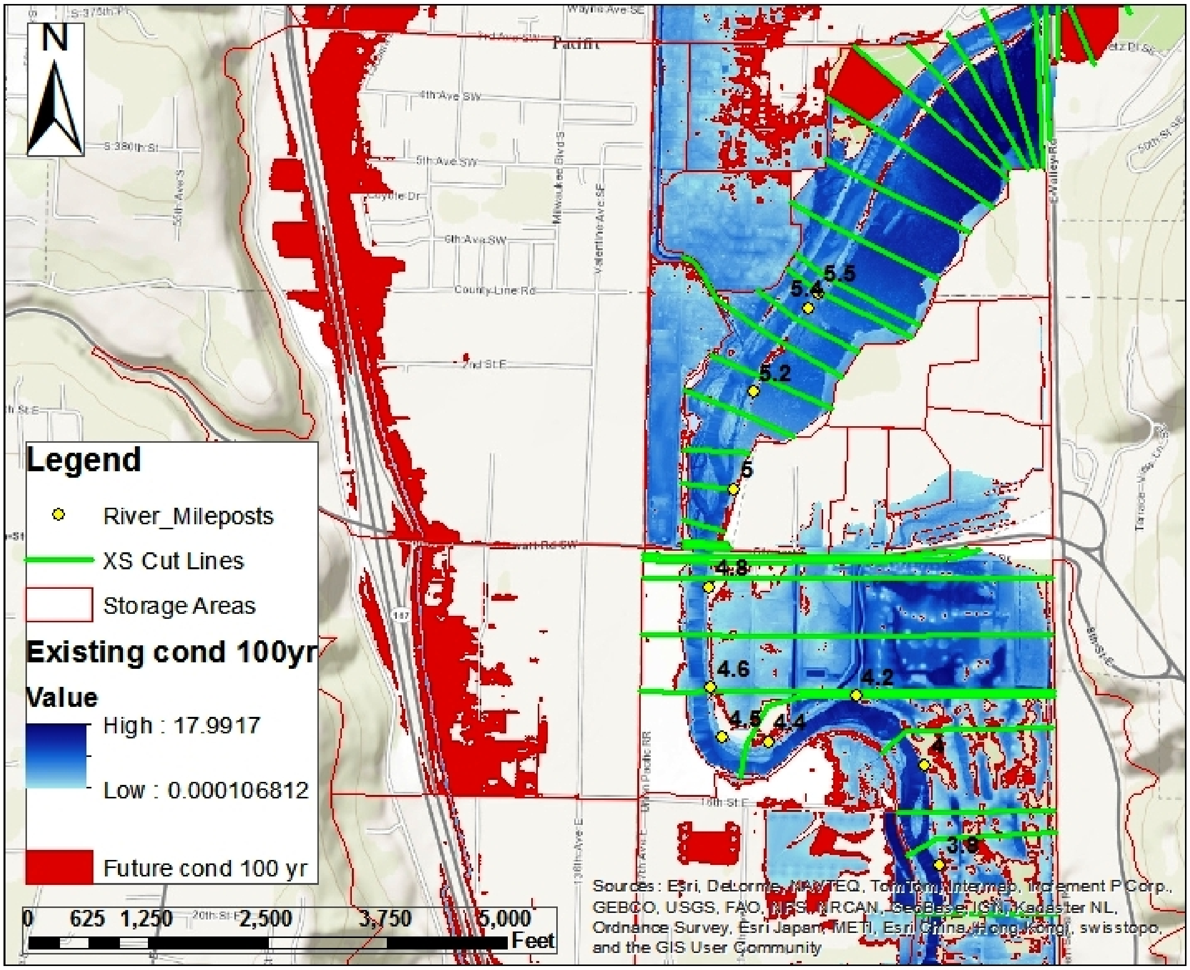

Figure 24: White River from RM 3.8-6

The White River from RM 4.8-6 is a depositional zone that has aggraded significantly since 2006 and has lost channel capacity. Stewart St Bridge has also lost capacity. The left bank from RM 4.6-4.8 is high terrain but prone to flooding from a 1% AEP event. The right bank from RM 4.9-5.2 is an old spoils pile from channel excavation over 1984-1987 and is high terrain but also prone to flooding. The left bank levee from RM 4.9-6 is scheduled to be setback beyond the historical meander belt over the next few years and is incorporated into the model as the County Line setback. The Government ditch enters the White River on the right bank at RM 5.3 and is prone to backwatering at frequent events through a 4-barrel culvert road crossing a great distance up into Pacific. This area is presently being analyzed in great detail by King County to reduce backwatering. Flooding seen on the far west side of the valley is backwatering from farther downstream at RM 1.2.

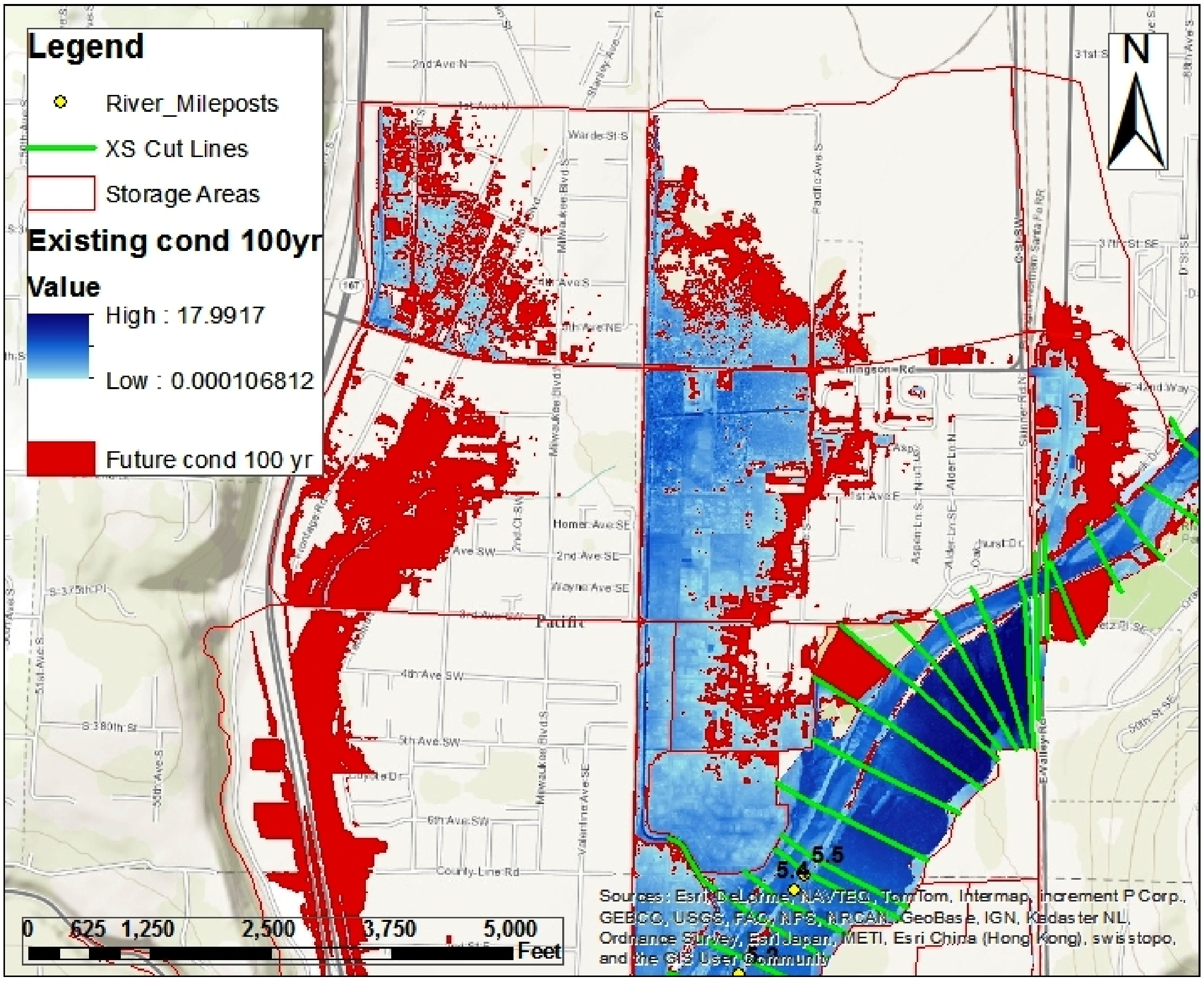

Figure 25: White River near the town of Pacific

The Right bank around RM 6 just upstream and downstream of the A St. Bridge has seen significant aggradation in recent years and is prone to flooding. The flood extent shown in central Pacific is from the Government ditch backwatering as well as right bank overtopping at RM 6 moving west to the lowest point in the storage area. Flooding seen on the far west side of the valley is backwatering from farther downstream at RM 1.2, as well as from floodwater moving west through storage areas from RM 6.

5.0 Future With-project Conditions Hydraulic Modeling

The base calibrated existing conditions hydraulic model was modified to incorporate the levee and the dredge alternatives. The geometry of the model was updated with future bed elevations as well as sea level rise discussed previously. Each alternative was developed separately to allow for comparison of effect on water surface profiles. The alternatives were designed to give the same benefit in flood risk reduction over coincidental areas to allow for a relative comparison of cost, environmental factors, and other criteria for alternative comparison. Ultimately the levee alternative was selected as the Tentatively Selected plan (TSP). This was communicated to Division at the 6/30/2014 IPR meeting. The reader is referred to the read ahead document (USACE, June 2014) for details of the screening process and scoring of alternatives.

5.1 Levee Alternative Modeling (Selected Alternative)

The levee alternative was selected as the TSP during the alternative selection process. Levee measures for this alternative consisted of raising existing levees, building new levees, and setting back levees. Measures are further explained in the FR/EIS document and the reader is referred to the project map in that document for locations. The TSP levee design used the 1% water surface plus 3 feet of residual levee height. Use of the hydraulic model was to provide water surface profiles for the 1% AEP exceedance event regardless of which levee measure was used at a given location. It should be noted that final feasibility design will be based on the upcoming NED analysis which will determine the optimal levee heights to maximize benefits. Raising of existing levees did not require any additional pre-processing in GIS. Actual design of levee profiles was done externally in GIS and/or CAD. The typical design process for a levee change was streamlined into several steps:

1. Create draft horizontal levee alignment in GIS from design team feedback and sponsor recommendations. Vertical alignment profile created from existing LiDAR terrain along the draft levee footprint.
2. Process levee alignment into HEC RAS import file (using HEC Geo RAS) consisting of reconfigured lateral structures, storage areas, and storage areas connections.
3. Delete existing model features and import/ reconfigure model with new levee measure.
4. Run the model to de-bug and access transferred risk to surrounding levees and basin.
5. Obtain feedback from the PDT on transferred risk implications and adjust model to contain flood water to the channel in desired areas.
6. Run the model (presumably with channel flow contained in areas where measures are developed) to determine water surface profiles for design event. Flow was contained by setting lateral structure coefficients to zero.
7. Export water surface profiles to GIS to process with estimated residual levee height (3 ft.) into shapefiles with horizontal and vertical alignments. This step gives the top-of-levee profiles that are used for actual design and for FDA analysis.

8. Hand off shapefiles to Civil Design Section to process into CAD and develop levee cross sections.
9. Update hydraulic model lateral structures with design levee profile elevations and run for 50-0.2% AEP events once all levee measures for the basin are incorporated.

This process was repeated for all levee measures to create the entire basin-wide alternative. A typical lateral structure feature was incorporated into the model as shown in Figure 25 for The North Levee Road levee set-back.

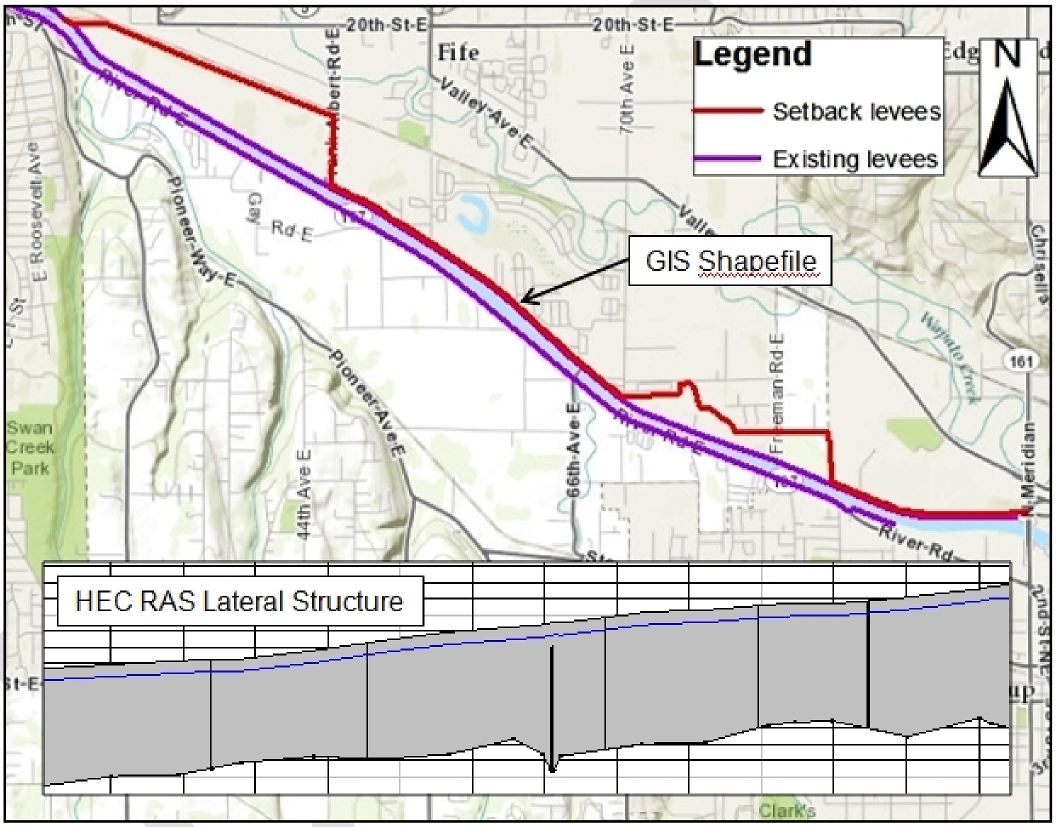

Figure 26: Development of typical levee design as Shapefile in GIS and Lateral Structure in HEC RAS (North Levee Road shown)

5.2 Dredge Alternative Modeling (Screened Alternative)

It should be noted here that the dredge alternative was screened out of the TSP. This section is included only for reference and to document the analysis conducted for alternative selection which led to selection of the levee alternative for TSP. Dredging for this alternative consists of removing a significant quantity of riverbed material from the main channel (often called mainstem dredging) in an effort to reduce water surface profiles along affected areas and provide a relative comparison with the levee alternative. In the hydraulic model this translated to modifying each cross section in the base geometry using the channel modification/design tools within HEC RAS. To achieve the most effective result, the channel bed was flattened out longitudinally to become close to parallel to the water surface, and each cross section in the model was cut into a trapezoidal shape by projecting the side slope down. This was done adjacent to existing levees and areas where a new levee measure was being considered for the levee alternative. Iteration was necessary to achieve 3 ft. of residual levee height along the existing levee or bank line. A typical reach was dredged as shown in Figure 26 for the Lower Puyallup River.

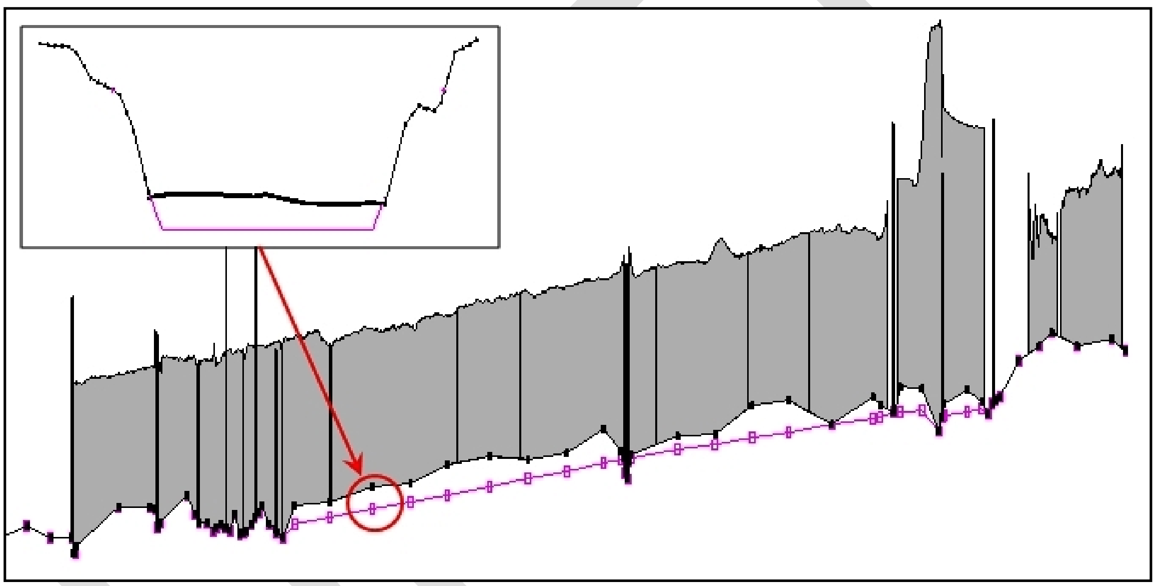

Figure 27: Development of typical dredged reach in the model using HEC RAS channel design/modification tool (Lower Puyallup River shown)

Dredged quantities were given directly by the channel design/ modification tool in HEC RAS and ultimately used for cost analysis of this alternative. Depth of dredging varied with the channel bottom profile along each reach. Average depths are given in Table 4 below for each location.

Table 4: Average depth of dredging

Location	Average Dredge depth (ft)	Area (acres)
Lower Puyallup RM 3.1-7.4	3.3	98
White- Pacific RM 4.9-6.2	3.2	30
Lower White RM 2.1-4.5	7.5	29
Upper Puyallup- Jones levee RM 21.3-22.7	2.5	36

The overall approach to dredging was to provide a 1% AEP level of protection (LOP) by the initial dredge effort, then sustain that LOP through maintenance dredging. It was necessary to estimate the volume of sediment accumulation over the project life and maintenance dredging intervals. This was determined for each reach from the historical cross section analysis used to estimate future conditions described previously. Maintenance dredging was presumed to occur at a trigger of LOP reduction to 2% AEP plus 3 ft of residual levee height. The volume of deposition allowed to accumulate for this LOP reduction was determined numerically by assuming that the ratio of average water surface (WS) change from initial dredge to initial dredge volume is equal to the ratio of average water surface change from deposition to depositional volume:

$$\frac{Average\ \Delta WS_{initial\ dredge}}{Volume_{initial\ dredge}} = \frac{Average\ \Delta WS_{deposition}}{Volume_{deposition}}$$

Where:

$\Delta WS_{initial\ dredge} =$ Determined directly from modeling

$Volume_{initial\ dredge} =$ Determined directly from channel mod design tool

$\Delta WS_{deposition} =$ Diff. between the modeled 50 and 100 yr WS (assummed)

$Volume_{deposition} =$ Unknown

When the volume of deposition is allowed to accumulate, the 2% AEP WS then becomes equal to what the 1% AEP WS was after the initial dredge, and the 1% AEP WS increases to provide something less than 3 ft. of residual levee height. Both of the variables on the right side of the above equation are actually unknown, but we can assume that the 2% and 1% AEP WS both increase by the same amount due to deposition. Which, given the wide width of the channel and relatively small dredge depth, is sufficiently accurate. From this assumption it follows that the change in WS due to deposition is roughly equal to the difference between the 2% and 1% AEP WS after the initial dredge. This relationship was then solved for volume of deposition that can be allowed to accumulate before dredging is necessary. The number of dredge events was then determined as the total volume of estimated deposition over the 50 year project life divided by the volume of deposition:

$$\frac{Volume_{50\ years}}{Volume_{deposition}} = \text{Number of dredge events}$$

Dredge quantities are given in Table 5 for applicable locations where the comparison is being made with corresponding measures in the levee alternative.

Table 5: Dredge Volumes

Location	Initial dredge vol. (cu. yd.)	Maintenance dredge vol. (cu. yd.)	Number of maintenance dredge events
Lower Puyallup	1,073,337	484,726	1
Lower White	544,931	409,579	1
White- Pacific	443,743	251,640	3
Upper Puyallup- Jones levee	461,545	281,341	2

In addition to dredging, a levee component was necessary in areas where backwater or tidal influence reduced the effectiveness of dredging. This included the lower 2-3 miles of the Puyallup River near Commencement Bay and the area of the White and Puyallup Rivers near their confluence. Measures and their locations are further explained in the alternative description section.

5.3 Impacts to Infrastructure

With the estimated future deposition throughout the basin and increased containment of flows by levee modifications, there are bridges and floodplain areas that could be impacted by higher river stages. Bridge decks that show possible flow impingement by 1% AEP future flow conditions are listed in Table 6. Several of the listed bridges are very close to impingement and all require further evaluation during later phases of future conditions modeling. Impacted sites will require either raising the bridge or dredging the bridge opening. The feasibility of dredging at several sites has not yet been determined at this phase of the study and are indicated in the table, and several of the sites are heavily backwatered from downstream sources.

Table 6: Bridges impinged upon by future conditions

Bridge	Location	With-out project	Levee Alternative	Dredge Alternative
66th Ave E / Clark St Bridge RM 5.7	Lower Puyallup		X*	X*[1]
142nd Ave Bridge RM 1.4	White	X	X	X[2]
8th St / Stewart St. Bridge RM 5	White		X*	X*[1]
SR 162/ Railroad Bridge RM 17.7	Upper Puyallup	X*	X*	X*[1]
SR 162/ foot bridge RM 5.8	Carbon	X	X	X[1]

* Flow impingement is critically close but does not contact the bridge deck in the modeling (within 1 ft.); [1]Dredging feasibility near the bridge requires evaluation at impacted site; [2]Dredging ineffective due to backwater conditions

5.4 Estimation of Future With-Project Flood Extents

Measures described above were modeled using HEC RAS as described previously for existing conditions. Measures were implemented in the model for each alternative. Because a relative comparison was intended between alternatives, modeled flood extents were identical for the levee and the dredge alternatives. Flood extents are shown below by area along with existing and future with-out project conditions for a 1% AEP event. Essentially, the difference between the Future with-project and the future without-project is the induced flooding from project measures.

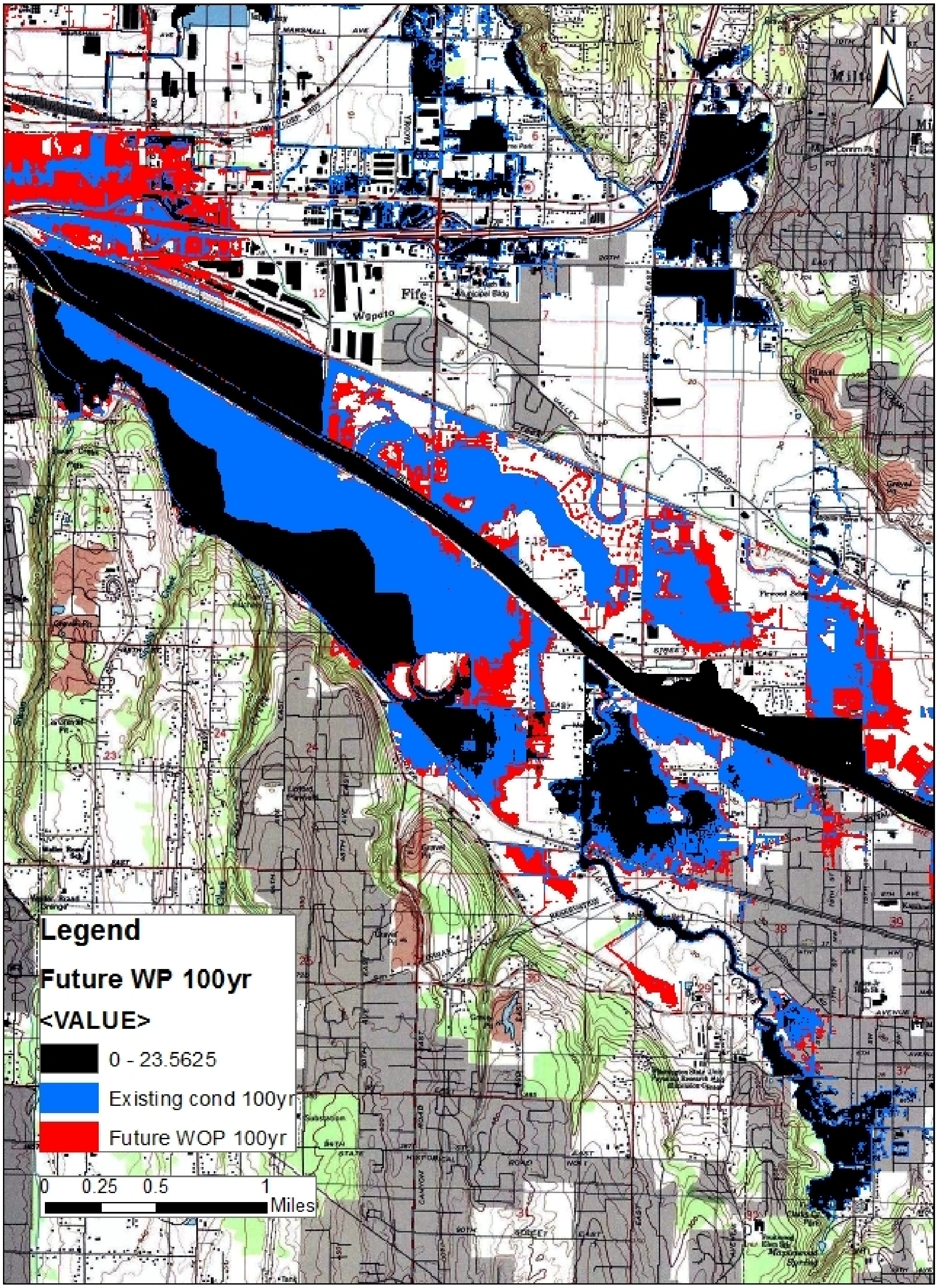

Figure 28: Lower Puyallup River

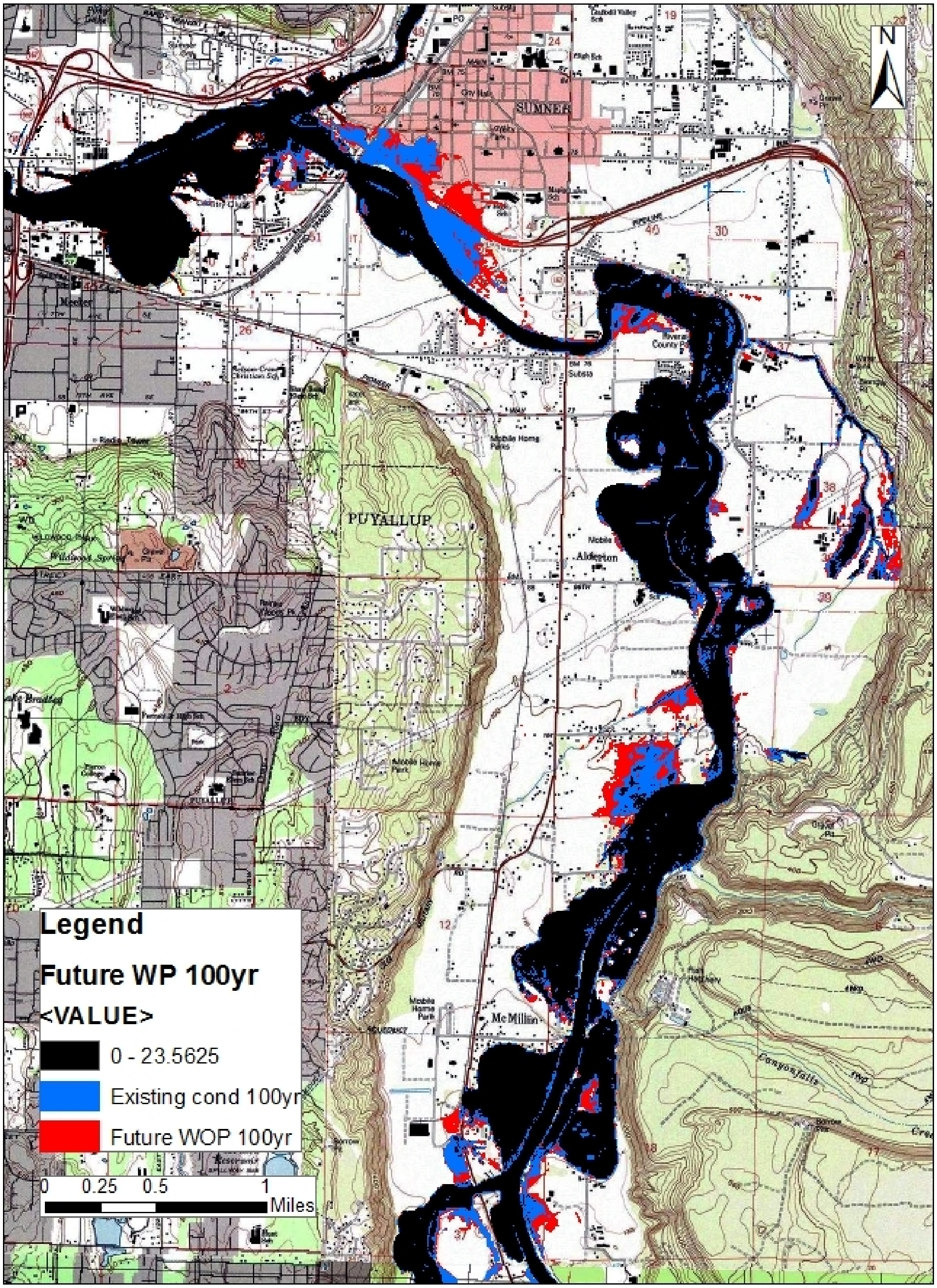

Figure 29: Middle Puyallup River

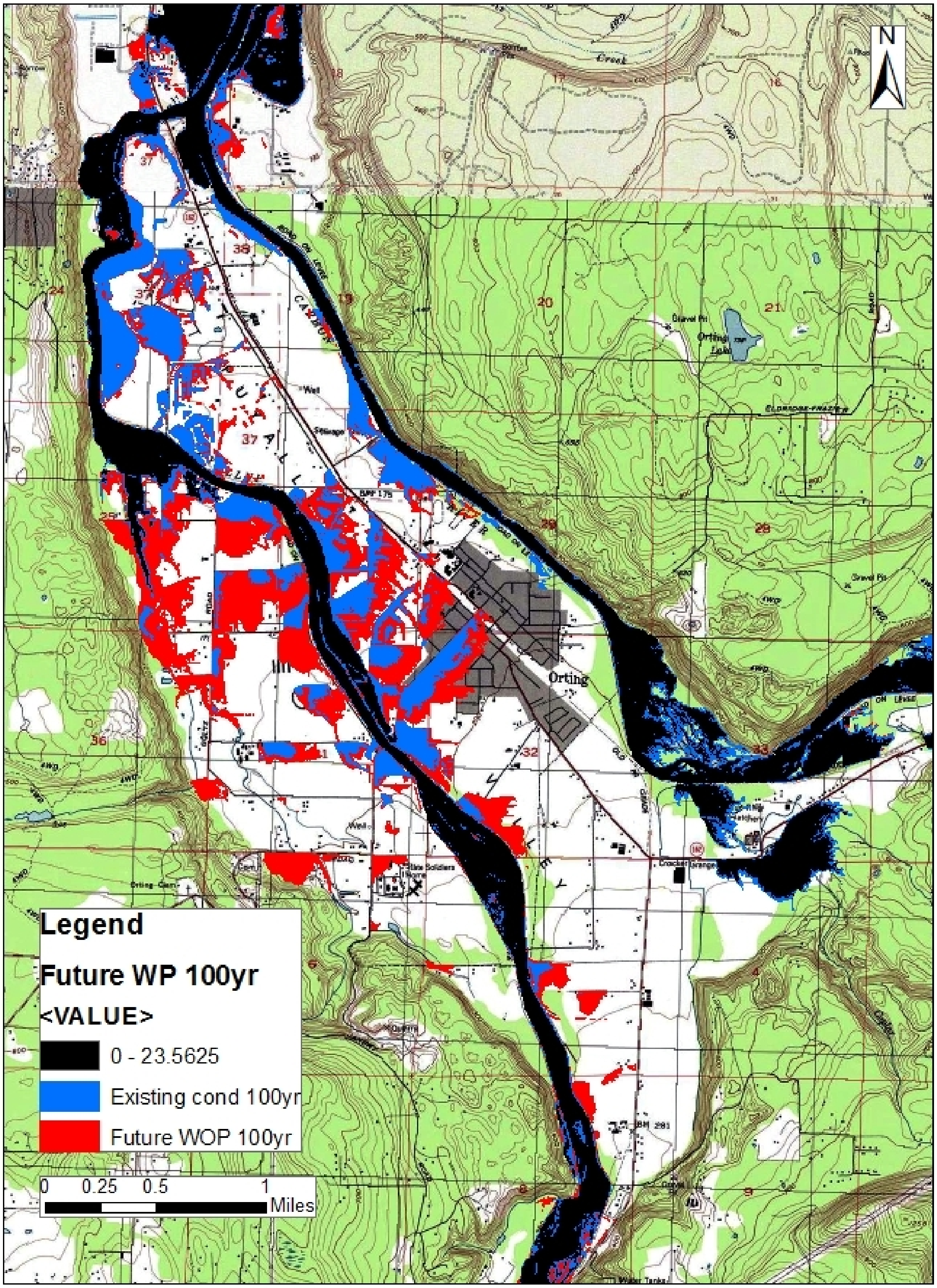

Figure 30: Orting area between the Upper Puyallup and Carbon Rivers

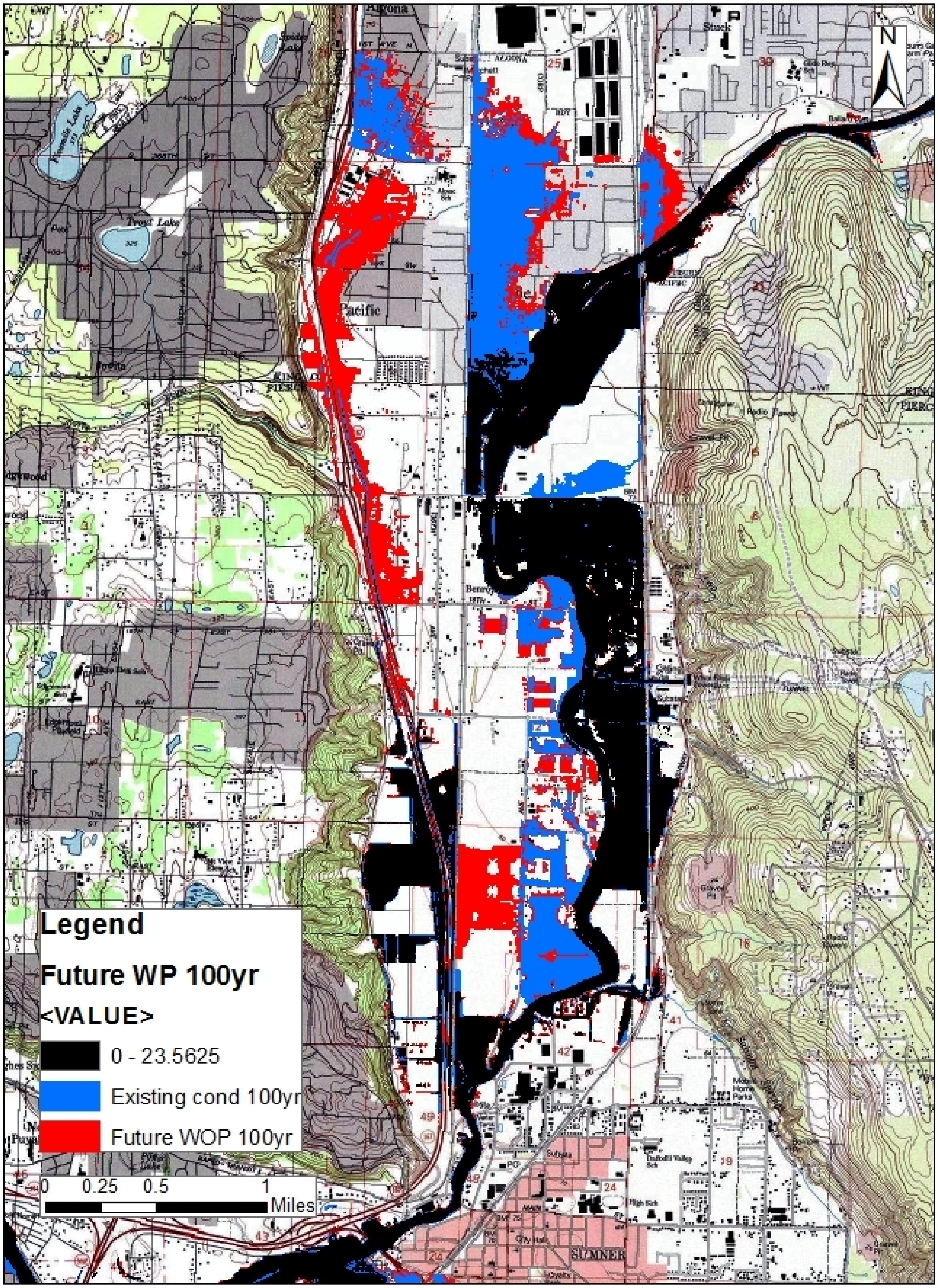

Figure 31: White River

5.5 Future With-Project Contingency Areas

The majority of sites identified for levee modification or dredging experience flooding under existing 1% AEP conditions or have questionable channel capacity within the existing levee system. These sites are discussed in the existing conditions H&H appendix. Several additional sites were identified for increased impact by 1% AEP flooding due to future conditions channel aggradation. The final answer for these areas, as to whether or not they actually need measures, will be determined based upon the result of sediment modeling which will occur after TSP to inform NED (optimization of the TSP). The cost contingency for these areas was determined to be within 10% of the total levee alternative length, or within 20% of the dredged volume, and was factored into the alternative selection process. As of the TSP milestone, discussion was occurring to determine how to incorporate these areas and how to address uncertainty in sediment deposition.

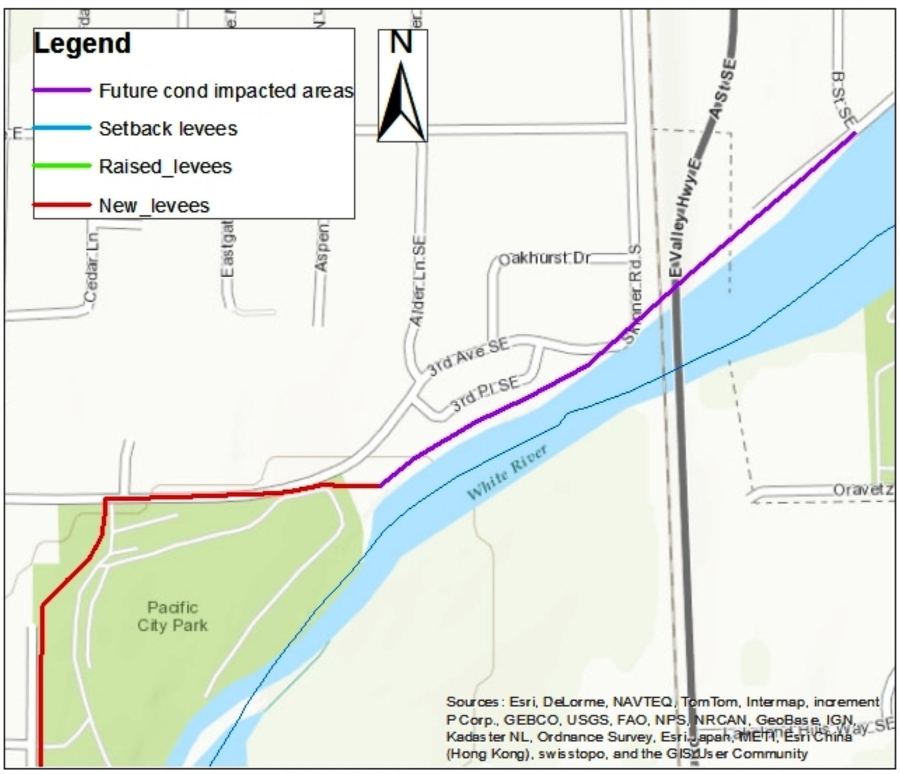

Figure 32: White River RM 6.1-6.5

At A St. on either side of the bridge, the right bank experiences increased overtopping from deposition pushing water over the existing bank. In modeling, this created a significant amount of flooding at Pacific. Note that flooding at this area is not due to backwater up the Government Ditch (which enters at RM 5.3). This area may need a short levee segment.

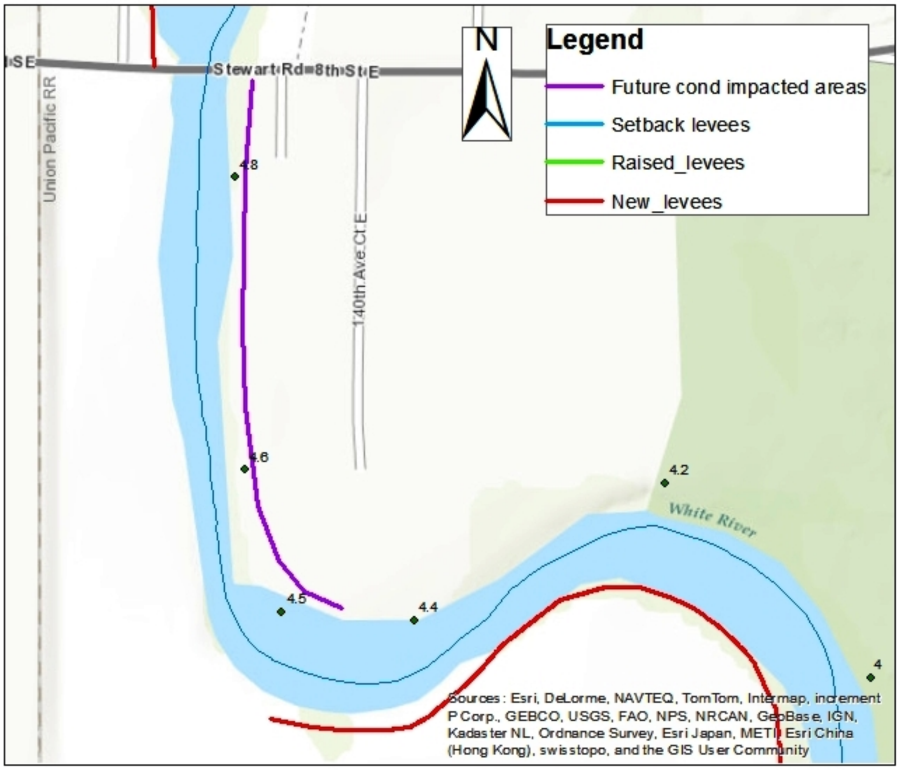

Figure 33: White River RM 4.9

At Stewart Rd. (RM 4.85) the left bank experiences increased overtopping from deposition. This area is being developed into a residential area by the City of Sumner as of TSP. The area will need to be re-examined at later design phases and a levee measure developed if needed.

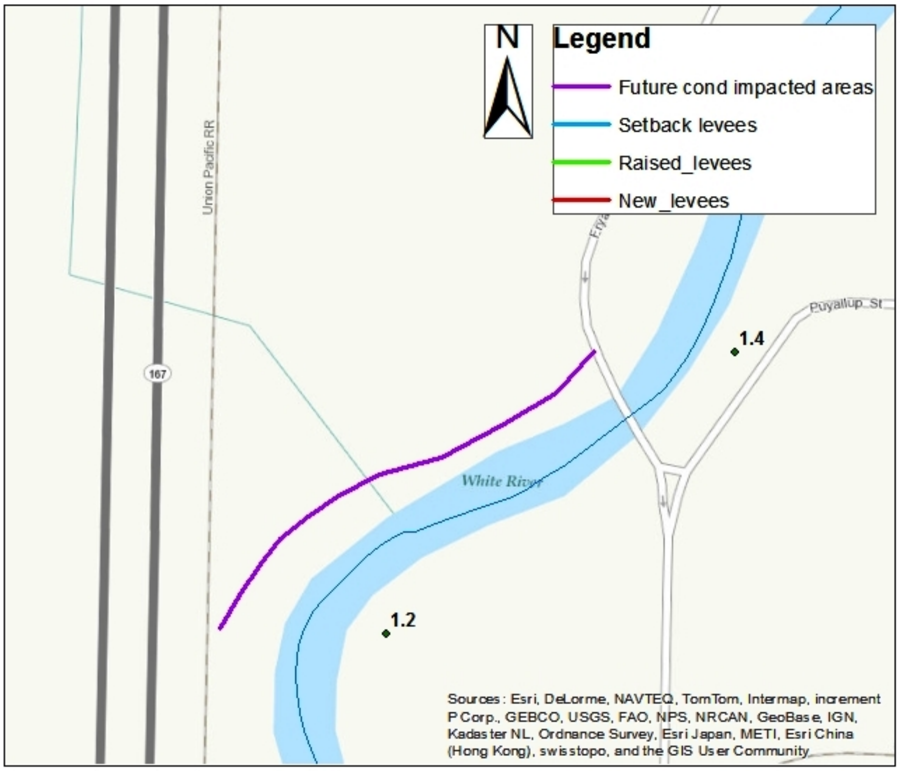

Figure 34: White River RM 1.3 near 142 Ave. bridge

The drainage canal experiences increased backwater of the interior area along Hwy 167 from deposition. Possible measures may include a gated outlet and levee.

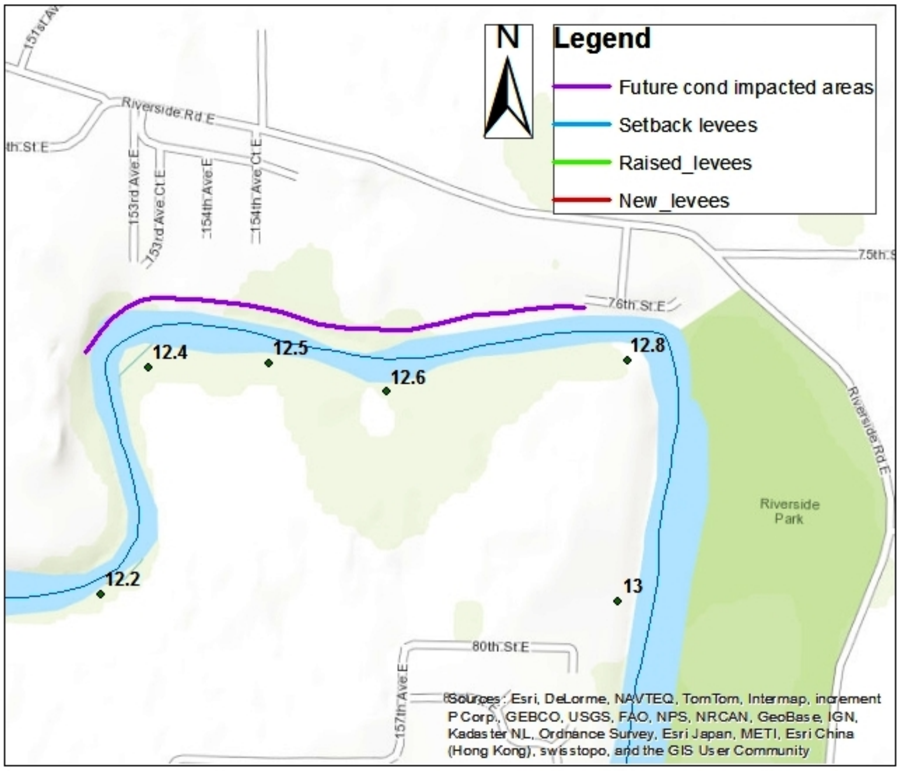

Figure 35: Middle Puyallup RM 12.5

The Riverside levee experiences increased right bank flooding due to deposition.

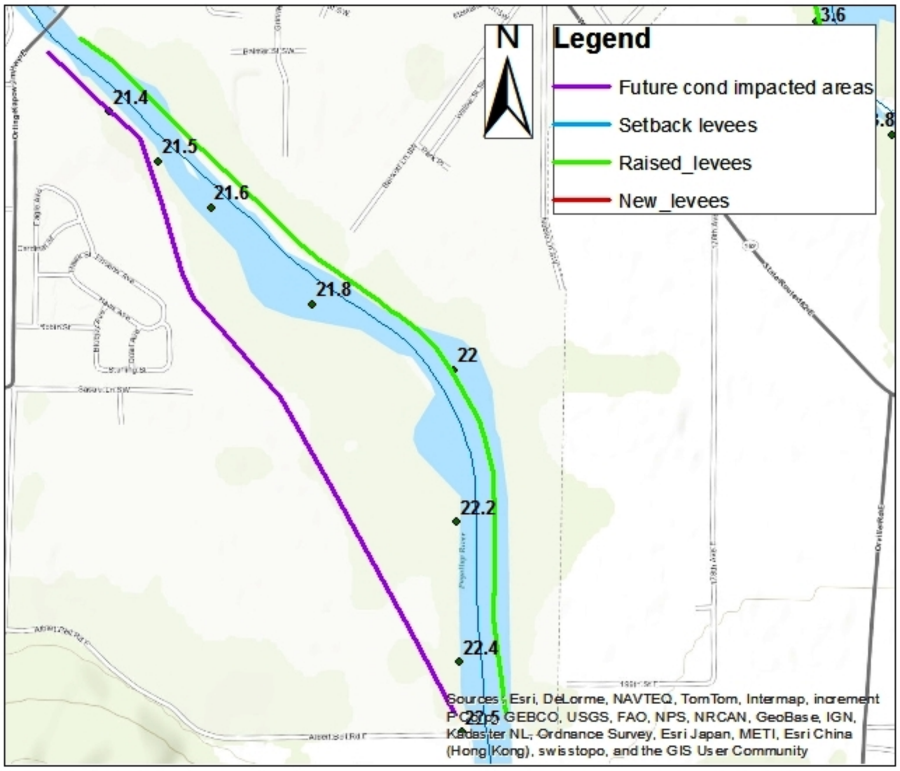

Figure 36: Left bank of Upper Puyallup RM 22

Orting on the left bank may experience increased flooding due to the right bank Jones levee measure containing more water in the channel and future deposition.

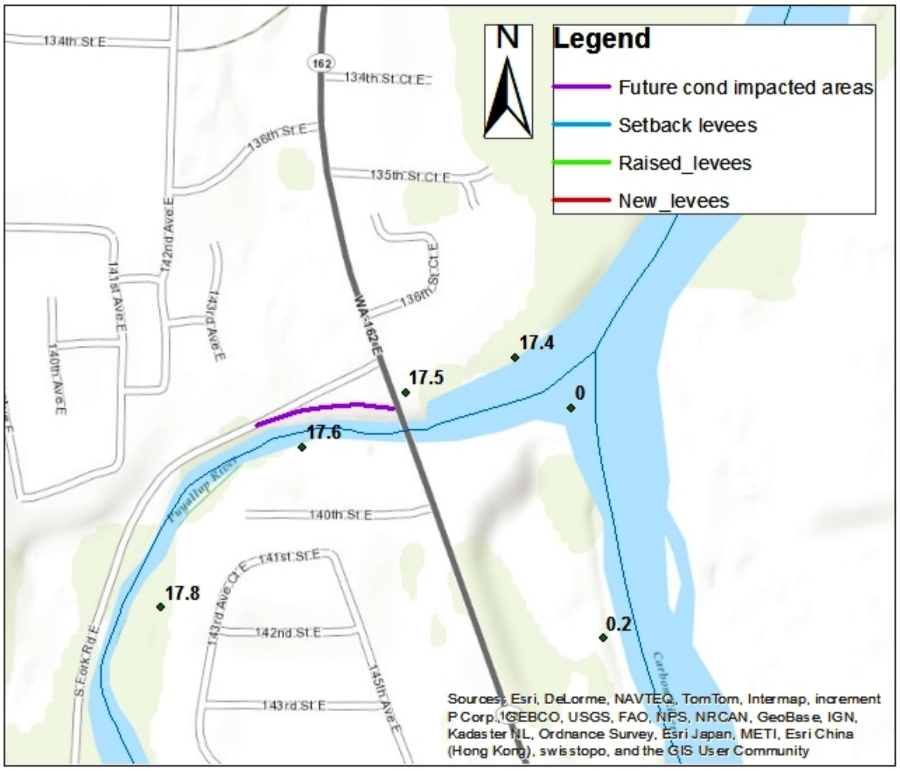

Figure 37: Left bank Upper Puyallup RM 17.6

The left bank area near the SR 162 Bridge at Orting may experience increased flooding from future deposition.

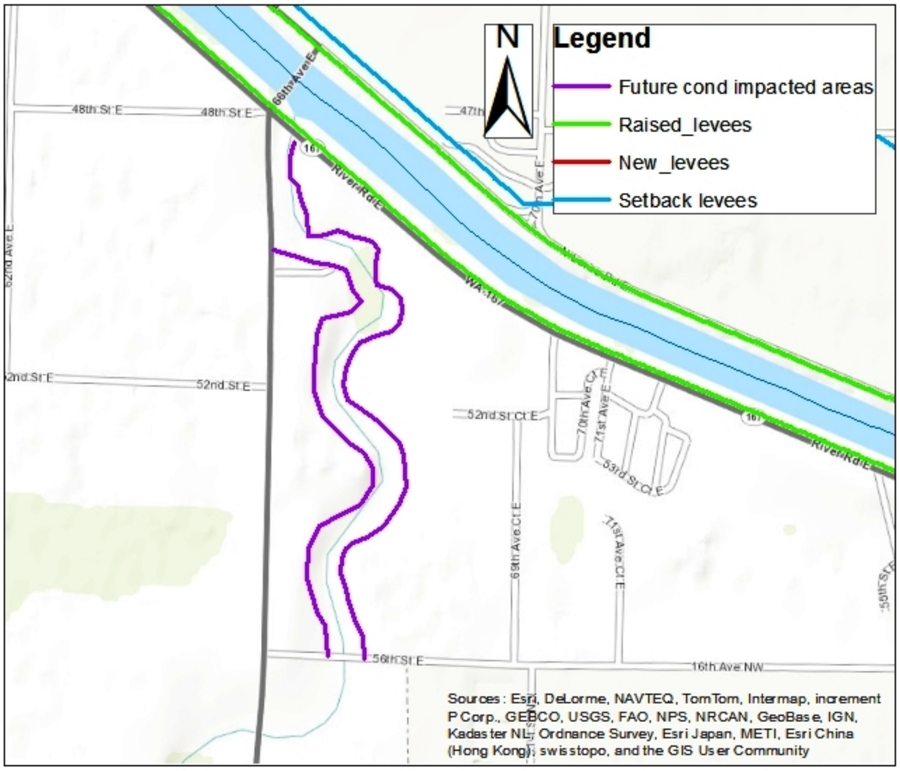

Figure 38: Clarks Creek area adjacent to the Lower Puyallup River at RM 5.8.

The Clarks Creek outlet is not gated and flows freely through a gap in the River Road Levee. During flood events the creek backwaters, which will be exacerbated by future conditions deposition. Possible measures include constructing a levee on either side of the creek, buyouts, or addition of a gate at the outlet. The proposed River Road Floodwall may influence selection of a measure.

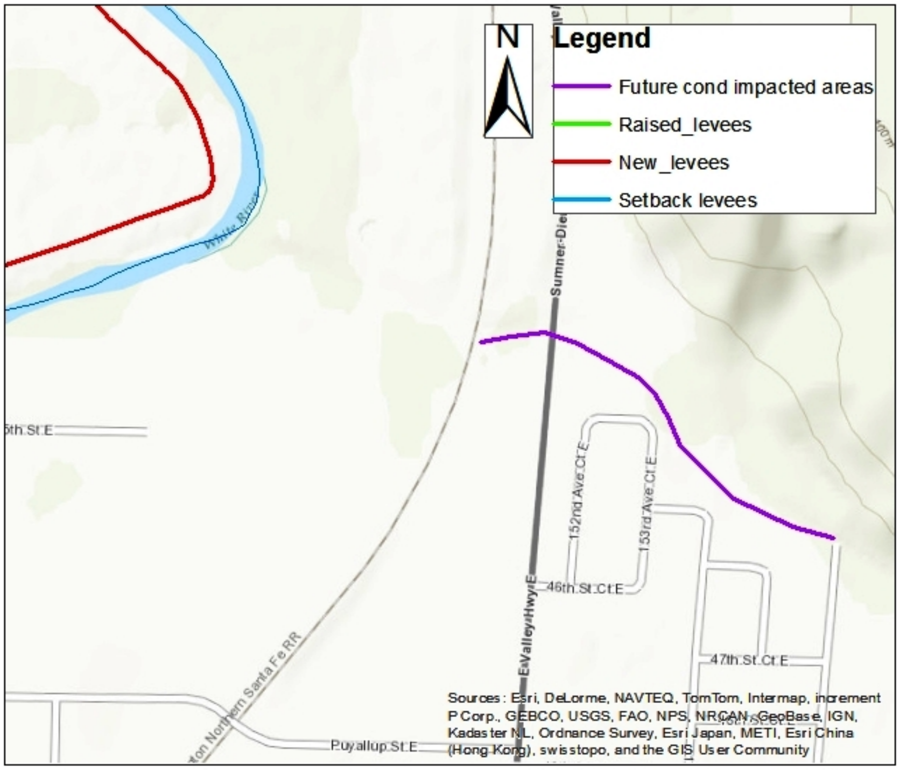

Figure 39: Local unidentified creek in Sumner adjacent to RM 2 on the White River

This area backwaters from the White River. Local development is on high ground above the 0.2% AEP water surface for existing conditions, but may need further evaluation for future conditions.

6.0 Recommendations from the USACE Committee on Channel Stabilization

In 2014, the Seattle District asked the Committee on Channel Stabilization (Committee) to provide recommendations as to various risk management strategies associated with proposed flood control improvements for this flood risk management study (USACE, 2014). The committee is comprised of subject matter experts from around the COE. The Committee was presented several issues and provided recommendations. Issues relevant to the Puyallup GI are listed below along with the district plan to address the issues for feasibility level design.

Issue 3: Future Sediment Yield and Transport. As the climate changes and warmer temperatures uncover more sediment sources on Mt. Rainier as glacial ice retreats, how can the District best assess the fate and transport of the newly uncovered sediments?

A significant part of what was recommended by the Committee for this issue has already been accomplished to date and significant research has been done by the USGS on this topic (USGS 2010 and 2012). This issue feeds into issue 4 and is being incorporated for this feasibility study. A formal Sediment Impacts Assessment Methods (SIAM) analysis may not be accomplished by feasibility; however the equivalent information has largely been determined and data gaps will be filled as necessary for incorporation into issue 4 sediment modeling. This is presently being discussed with the sedimentation subject matter expert on the Committee.

Issue 4: Future Sediment Modeling and Data Collection. The District anticipates reaching the TSP milestone before detailed sediment modeling can be completed. After that, there will be very limited time and funding to complete the feasibility study in accordance with SMART planning guidelines. The District is seeking the committee's opinion on how much risk they are taking by advancing the study forward without additional sediment data collection and modeling, and a recommendation as to what analyses must be completed given the short time and resource constrictions.

The Committee's recommendation to incorporate reach specific sediment modeling is being incorporated into this feasibility study after the TSP milestone during feasibility level design when the NED analysis occurs The risk of this significantly affecting the TSP has been communicated and documented in the risk register. It is not expected that the results of sediment modeling will substantially alter the TSP.

Issue 5: Effectiveness of Dredging. The District needs to better understand the effectiveness of gravel removal (dredging) from designated channel reaches in the basin. What is the best way for the District to determine the long and short-term effectiveness of dredging operations?

It was found that dredging alone was not sufficient solve the flood risk reduction goals of the study, and some levee component was needed. For this and other reasons the dredge alternative was eliminated and communicated at the 6/30/2015 IPR (USACE, 2014). The committee concurred with this decision. This left some questions about the long-term feasibility of not dredging and extents to which the channels may become perched as a result of not dredging. The model developed for issue 4 is needed to evaluate both of these questions. Depending upon the answers we get from addressing Issue 4, it may be necessary to answer the question of how perched of a channel is acceptable from a flood risk perspective. The information developed will be communicated to the sponsor for this feasibility study. No pilot *dredging projects are scoped for the GI study. The sponsor is independently perusing a pilot dredging project on the Upper Puyallup River.*

Issue 6: Levee Setbacks. The District is interested in the impacts of levee setbacks on sediment transport, and given the present knowledge, how can the District best determine the long and short-term effectiveness of levee setbacks relative to flood risk reduction?

It should be noted that no levee setbacks are included in the TSP for areas known to be highly depositional (the White River near Pacific and the Upper Puyallup River near Orting). The sponsor has independently pursued levee setbacks in these areas. These designs from the sponsor were incorporated into the existing conditions hydraulic model (including land use changes) and will be treated as such for sediment modeling of issue 4. The TSP contains raised levees in these two areas. The only levee setback in the TSP is on the Lower Puyallup River and is expected to only be accessible to high flows that exceed the capacity of the existing bank. The area will not be subject to channel migration due to existing concrete panels that line the entire reach underneath the existing levee prism. The model developed for issue 4 will be used to assess what deposition is possible in this set-back area. The answer is not expected to significantly affect the height of this levee. Sediment modeling efforts by the sponsor for their levee set-backs have fallen short of providing predictions for the GI project life of 50 years. So a final answer (to be provided by issue 4) is needed for future condition bed elevations where we are raising levees near setbacks and in all areas where we are raising levees. Sediment modeling for issue 4 will also inform the question of a perched channel forming in depositional areas. Depending upon the results we get from Issue 4, it may be necessary to answer the question of how perched is acceptable from a flood risk perspective.

7.0 Reference List:

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Motsenbocker, 2013. Personal Communication with Pierce County Flood Control District.

West, 2013. Personal Communication with West Consultants.

Appendix B-3

Hydrology and Hydraulics

Mapping of Hydraulic Model Runs

Puyallup River basin

Flood Risk Reduction Feasibility Study

The following runs were accomplished using the HEC RAS modeling described in the H&H Appendices. Combinations of events/conditions for which flood extent maps were produced were based upon PDT input and the needs of each discipline. For the FDA analysis the hydraulic model output for all 2-1000 yr events was used. Only flood extent maps were created for the conditions listed below. A total of 64 reach level maps were created and are included as a map appendix to the Feasibility report. Note that for consistency with organization of the hydraulic model the terminology used below is given n-year rather than AEP for hydraulic model plan names. The term "existing" or "exist" is used in the HEC RAS model for the "base" condition, and refers to the point in time when the project starts accruing benefits.

Flood extents were produced for the following conditions:

- **50%, 5% AEP-** WOP base conditions (with breaches), future W/P conditions
- **2%, 1%, 0.2% AEP-** WOP base conditions (with breaches), WP base conditions, future WOP conditions (with breaches), future WP conditions

50% AEP WOP base condition plans (short ID's):
Exist 2 yr WOP Up
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\UPPER_WO_EX_FUT
2 Yr Apr2015BR9944
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
2 Yr Apr2015BR10752
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
2 Yr Apr2015BR42525
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT

50% AEP future WP condition plans (short ID's):
2 Yr Fut WP UP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT
2 Yr Fut WP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT

5% AEP WOP base condition plans (short ID's):
Exist WOP 20 yr Up
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\UPPER_WO_EX_FUT

Exist WOP 20 Yr Br 9944
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Exist WOP 20 Yr Br 10752
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Exist WOP 20 Yr Br 42525A
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT

5% AEP future WP condition plans (short ID's):
20 Yr Fut WP UP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT
20 Yr Fut WP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT

2% AEP WOP base condition plans (short ID's):
Exist WOP UP 50 Yr
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\UPPER_WO_EX_FUT
Exist WOP Lwr 50 YrBr9944
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Exist WOP Lwr 50 YrBr10752
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Exist WOP Lwr 50 YrBr42525
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT

2% AEP future WOP condition plans (short ID's):
Fut WOP 50yr Upper
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\UPPER_WO_EX_FUT
Fut WOP 50 10752BR
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Fut WOP 50 9944BR
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Fut WOP 50 42525BR

O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT

2% AEP WP base condition plans (short ID's):
50 Year Existing WP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT
50 Yr Exist WP UP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT

2% AEP future WP condition plans (short ID's):
50 Yr Fut WP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT
50 YR FUT UP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT

1% AEP WOP base condition plans (short ID's):
100 Yr Exist Up
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\UPPER_WO_EX_FUT
Exist WOP Lwr 50 YrBr9944
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100_lower_10752BR
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
100_lower_42525BR
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT

1% AEP future WOP condition plans (short ID's):
Fut WOP 100 yr Upper
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Fut WOP 100 9944Br
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Fut WOP 100 10752Br

O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Fut WOP 100 42525Br
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT

1% AEP WP base condition plans (short ID's):
100 Year Existing WP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT
100 Yr Exist WP UP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT

1% AEP future WP condition plans (short ID's):
100 Yr Fut WP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT
100 Yr FUT WP UP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT

0.2% AEP WOP base condition plans (short ID's):
500_UP Exist WOP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\UPPER_WO_EX_FUT
500_lower_BR9944
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
500_lower_BR10752
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
500_lower_BR42525
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT

0.2% AEP future WOP condition plans (short ID's):
Fut WOP 500 yr Upper
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\UPPER_WO_EX_FUT
Fut WOP 500 9944BR

O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Fut WOP 500 10752BR
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT
Fut WOP 500 42525BR
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\LP_WO_EX_FUT

0.2% AEP WP base condition plans (short ID's):
500 Year Existing UP WP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT
500 Year Existing WP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT

0.2% AEP future WP condition plans (short ID's):
500 Yr Fut WP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT
500 YR FUT WP UP
O:\BASINS\WestWA\PUYALLUP\Puyallup_GI\GI_TSP_Modeling_Sept15\Puyallup GI Sept 2015 TSP\WP_EX_FUT

Runs were mapped using RAS Mapper to create depth grid files (Geo-TIFF's) for 2, 20, 50, 100, 200 year events. Three Breach scenario grids (only for the lower basin runs) were merged to create the greatest possible flood extents that could exist given present levee reliability estimations. These breaches were at XS 42525, 10725, and 9944 on the Lower Puyallup. Grids for upper and lower basin runs (these have slightly differing hydrology) were merged also. This gave four grids that needed to be merged for WOP conditions. For WP conditions there are no breach runs and only upper and lower basin grids needed to be merged.

Appendix B-4

Hydrology and Hydraulics

Support of FDA Analysis

Puyallup River Basin

Flood Risk Reduction Feasibility Study

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1.0 Hydraulic Model Considerations

1.1 Hydrology

Flood statistics for the Upper Puyallup and Carbon River reaches are based on stream gages with long records located upstream of the study area. For damage reaches along these two reaches, the flow-frequency curve in FDA was based on the Carbon River at Fairfax USGS gage 12094000 and the Puyallup River near Orting USGS gage 12093500. The frequency curve used the analytical option in FDA based on HEC-SSP analysis per Bulletin 17B (1982). A transform function was used to capture the local inflow downstream of the stream gages and any hydraulic factors affecting flow such as levee overtopping and local inflows. The "outflow" part of the transform function was based on the routed flows computed by the hydraulic model.

The White River and Lower Puyallup River are a little more complicated. One factor complicating the approach was flow regulation during floods from Mud Mountain Dam. Another was the roughly 50,000 cfs channel capacity of the Lower Puyallup reach. There is a stream gage in the middle of the Lower Puyallup reach (USGS gage 12101500). An analysis of the unregulated local inflow (the portion of the basin excluding that above MMD) indicated that flows well in excess of 50,000 cfs are statistically possible. Since releases from MMD are based in large part on the local inflow (to try to keep the Lower Puyallup below 50,000 if possible) they may have to go to zero outflow during peak. An analytical local inflow frequency curve was used for the White and Lower Puyallup reaches with a transform function to capture the added flows from MMD regulation and any hydraulic factors like levee overtopping and local inflows.

The purpose of using the transform function, which is described in greater detail in the sections that follow, was to assure consistency in the hydrologic uncertainty between the without project conditions and with project conditions. The routed flows could have simply been used from the hydraulic model in a graphical frequency curve for each condition. The problem with this however is that as features of alternatives were included in the model, the routed flows changed. The graphical frequency curve approach did not let us parse out hydrological uncertainty in a statistical sense (that which would be the same from condition to condition) from that of a particular alternative or condition (which would vary).

1.2 Hydraulics

The HEC-RAS unsteady flow hydraulic model uses cross sections to model the riverine portions of the study area and a network of storage areas to model the floodplain areas. The model contains approximately 185 storage areas. The riverine and floodplain regions are connected by lateral structures representing levees, roads, railroads, etc. adjacent to the rivers. The storage areas are connected to each other where appropriate using storage area connections. As with the lateral structures, these connections represent features such are roads, railroads etc. that define a 'bathtub' area in the floodplain. Water can flow from storage area to storage area via the storage area connections as appropriate. The main purpose of this approach is to allow for the computation of floodplain water surface elevations which can be different from that of the river. The approach also allows for the movement of water in directions perpendicular to the main channel. Modeling the floodplain as a parallel streams was tried as an alternate approach. While this method allowed for differing river/floodplain water surface elevations, it did not account for different water surfaces in the floodplain normal to the river as well as the storage area approach.

A couple of issues inherent to the storage area approach used in the hydraulic model should be explained and are described in greater detail in the Existing Conditions Modeling Appendix. The first is with the physical movement of water through the floodplain. The storage area regions represent where the water ultimately pools in a given storage area. The flow path through the storage area to the pooling area is not captured. This is not ideal but in reality much of this flowing water would likely flow through existing low lying areas which may be typically void of structures en route to the final pooling locations. The second issue is the modeling depicts very fast travel times through the floodplain. As with the first issue this is related to the simplification of no overland flow process component. To partly compensate for this, the weir coefficients used with the storage area connections were set to low values-0.5, a value recommended in discussions with HEC.

Calibration of the model is based on comparisons to stream gages within the study area and some available high water marks. Calibration is difficult due to the large ungaged contribution at various locations and inconsistent flow-stage ratings from flood event to flood event due to sedimentation. The large ungaged inflows make it difficult to accurately estimate observed event flow values away from stream gages. Changing flow-stage relationships due to sediment make it difficult to use older flood events to verify the hydraulic model. In addition, the type of event that would produce significant damages is one where levee overtopping/breaching would occur and a lot of water would be moving around in the floodplain. This type of observed event has not occurred for use in verification of the hydraulic model.

Calibration/verification of the hydraulic model was based on using information at discreet points (such as high water marks or USGS gage data) and making model adjustments based on a similar reach approach. For instance, along the Lower Puyallup reach there is a USGS stream gage. The assumption with this reach is that Manning's n values are the same (unless there is some defensible reason to deviate) throughout the reach. The reality is however, that an observed water surface elevation to work with only exists at the gage and at the downstream boundary (Commencement Bay) and the assumption is that the model parameters are appropriate elsewhere in the reach.

2.0 FDA Analysis Support

The study area streams were divided up into five logical reaches-the Lower Puyallup, the Middle Puyallup the Upper Puyallup, the White River and the Carbon River. These reaches are shown in Figure 1 below. Along most areas of the Study Rivers there is a levee, or other structure acting like a levee, adjacent to the stream. As discussed earlier, the floodplains in these locations are modeled with a network of storage areas or cells that represent "bathtub" areas where water would pool. For the five reaches, it is reasonable to expect more or less consistent flood statistics within a reach for a given flood (i.e. if a 1% Annual Exceedance Probability, or AEP, flood occurs at one spot along the Lower Puyallup reach it is reasonable to expect that conditions everywhere else in this reach would be representative of 1% AEP conditions). The basic structure and rational for the FDA model approach are described below for topics that required significant H&H input. The H&H FDA inputs include flow-frequency curves, discharge-stage rating curves, water surface profiles, and associated uncertainties. The reader is referred to the Economic Appendix for further detail on the FDA model and data.

2.1 Damage Reaches

For the purposes of the FDA modeling, separate damage reaches were designated for each reach for the riverine portion of the study area (essentially capturing water surfaces computed at model cross sections) and for the right and left floodplain portion (the areas where water surfaces are based on a storage area computation). This results in three FDA damage reaches for each of the five reaches plus an additional damage reach to represent Commencement Bay as a flooding source. Figure 1 shows the study area, the river reaches and the floodplain (based on a group of storage areas) damage reaches.

Index points were generally selected at locations of incipient overtopping that caused the greatest flooding for a given damage reach. The exception to this was on the Lower Puyallup where levee fragility was used in as a failure mechanism. If failure before overtopping was determined to be most likely, then that location was used. Three locations met that criteria, and are at cross sections 42525, 10752, and 9944 on the Lower Puyallup.

Typically in FDA if a damage reach has a levee or other feature separating it from the flooding source (river), the elevation of the levee is entered (the top-of-levee input) in the "levee" editor in FDA. This allows for the program to not compute damages for structures on the landward side of the levee when river stages are greater than the floor elevation of the structure but are below the top elevation of the levee for the particular river condition the model is sampling during its Monte Carlo routine. Additionally, for conditions where river conditions would cause levee overtopping but the landward water surface elevations would be different than that of the river, FDA allows for the inclusion of an interior-exterior relationship to reflect the water level that structures would see. This configuration also allows for the computation of structure performance values such as conditional non-exceedance probability by event.

Based on the physical system and the configuration of the damage reaches as shown in Figure 1 the “typical” approach described above would allow for the capture of the varying floodplain water surface elevations computed in the floodplain by the hydraulic model. With the “typical” approach the entire area on the landward side of levees would see the same water surface elevations as the river during overtopping or apply an interior-exterior relationship and assume the entire damage reach would use the same offset. The results of the hydraulic modeling have shown varying water surfaces through the floodplain and suggest that neither of these are good assumptions. Another approach was developed by treating each damage reach as a “pseudo” stream.

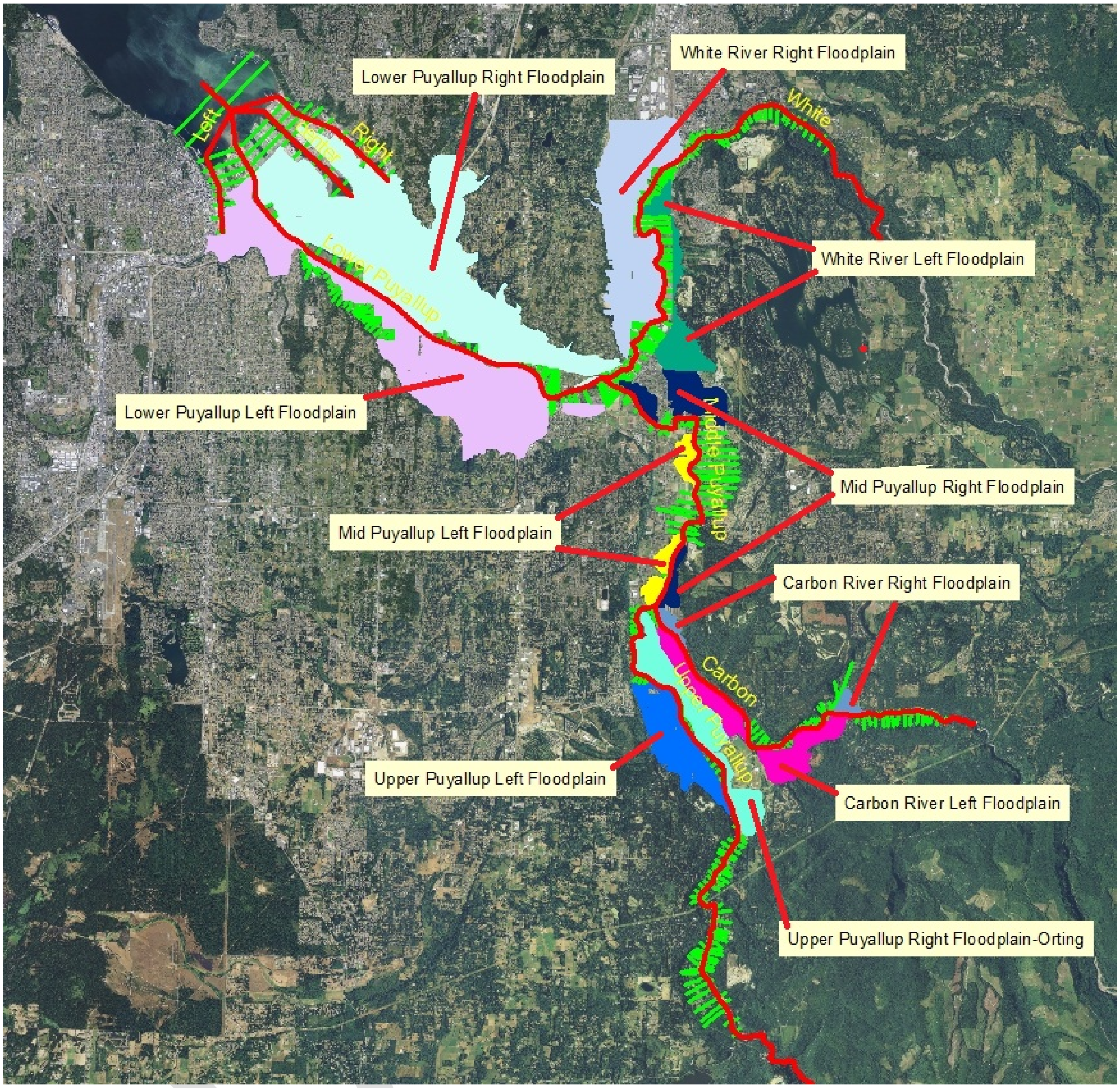


Figure 1: Floodplain Damage Reaches

2.2 Pseudo Streams

The concerns stated above were mitigated by breaking the damage reaches up into smaller units. Initially, each storage area in the hydraulic model was treated as a separate damage reach. This resulted in a very time consuming data entry exercise to input the various interior-exterior relationships (~185 of them) as there is not an import mechanism in FDA. As a work around, an approach was adopted similar to that used for cases where a two-dimensional hydraulic model is used to characterize the hydraulics. Each floodplain damage reach (for example Lower Puyallup Left Floodplain in Figure 1) was treated as a “pseudo” stream. Each storage area in the damage reach became a station on the pseudo streams water surface profile (WSP). In addition to the storage area stations, another station was added from cross sections on the riverine flooding source stream. In the case of the Lower Puyallup Left Floodplain damage reach, for example, this additional station would be a hydraulic model cross section in the Puyallup River adjacent to the damage reach boundary. This point at the river cross section was used as a damage reach index point.

2.3 Water Surface profiles

Water surface elevations for each of the eight return interval profiles were retrieved from the hydraulic model results. Stations corresponding to storage areas were populated with the appropriate storage area water elevation and the riverine station was populated with flow and elevations from the corresponding hydraulic model cross section. Since a discharge-frequency based approach is used, flow values are needed at every point on the Water Surface Profile (WSP). At the index point the flow values are physically based as they come from the hydraulic model. For the storage areas there is not a flow value per se to go along with the elevation. Arbitrary increasing values of 100, 200...800 were used as placeholders. As long as an index point does not occur at one of the storage area locations, FDA does not use the flow values. All of the index points were at a riverine location where the flow values have physical meaning. Since storage areas do not have a true sloping water surface profile in the physical sense, it is critical that all structures were assigned to a WSP station so FDA did not interpolate between storage area stations.

The last item to address in the approach was empty storage areas. For many of the eight statistical events simulated, some storage areas remained dry or were not flooded from the river. During these conditions they still needed a water surface to include on the WSP. Storage area inverts or the lowest physical ground elevations could have just been included. The concern with doing this was that the river or index point locations are never dry and FDA would be translating the uncertainty associated with this location along the WSP when it aggregates the individual structure damages back to the index point even when the storage area is dry. This uncertainty could result in the computation of frequent event damages. At frequent events we are confident in the hydraulic model results indicating a storage area is dry. For instance, our model results may show a storage area as dry for the 50%, 20%, 10%, and 5% AEP events and then show increasing amounts of water for the 2% and less frequent conditions. If the uncertainty band placed on the flow-frequency and flow-stage relationships at the index point were translated to a particular structure station (i.e. storage area), FDA could interpret that damages could occur because it would be applying the uncertainty to the ground elevation which may result in a water surface range high enough to reach structures. To address this, for conditions where modeling indicated storage areas were dry, a value of -19.99 was entered which was well below any structure ground elevation. When modeling indicated water in a storage area, the computed storage area elevation was entered (entering -19.99 for a dry condition instead of the actual invert is a way to turn off the uncertainties and the possibility of FDA computing a damage value for a dry condition).

Figure 2 below shows the 1% AEP WSP for the Lower Puyallup Left Floodplain Damage reach noted in Figure 1. The first point at Station 500000 is the index point and is based on the hydraulic model 1% AEP water surface elevation (WSE) computed at the Puyallup River cross section corresponding to this point.

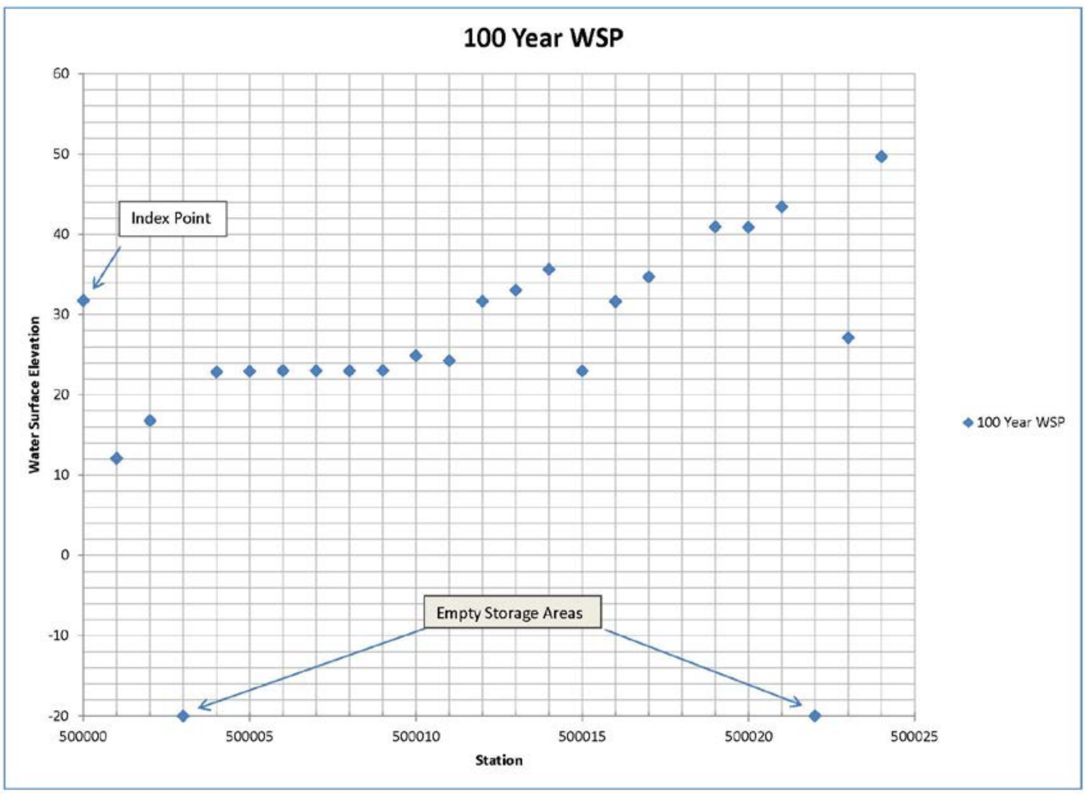


Figure 2: Example without Project WSP (1% AEP) for the Lower Puyallup Left Floodplain

The rest of the points are WSE values computed in storage areas within the damage reach. Note there are two storage areas that are dry and are noted such by the -19.99 WSE. The structure inventory has stations assigned so they fall on one of these points and not between. The elevations shown are based on all the features of a particular alternative being in place. The profile shown in Figure 2 is for the without project condition. The results would be different for the with-project condition with a levee. The presence of an improved levee might make the 1% AEP index point elevation higher due to more flow containment but would presumably reduce the elevations of the rest of the stations. This would likely result in more dry (-19.99) storage areas. The impact of the levee (or lack of one) in this example would be imbedded in the modeling results and the WSP. Figure 3 shows the 1% AEP profile from Figure 2 but also has the 5% AEP profile overlaid. Note there are more dry stations on this profile.

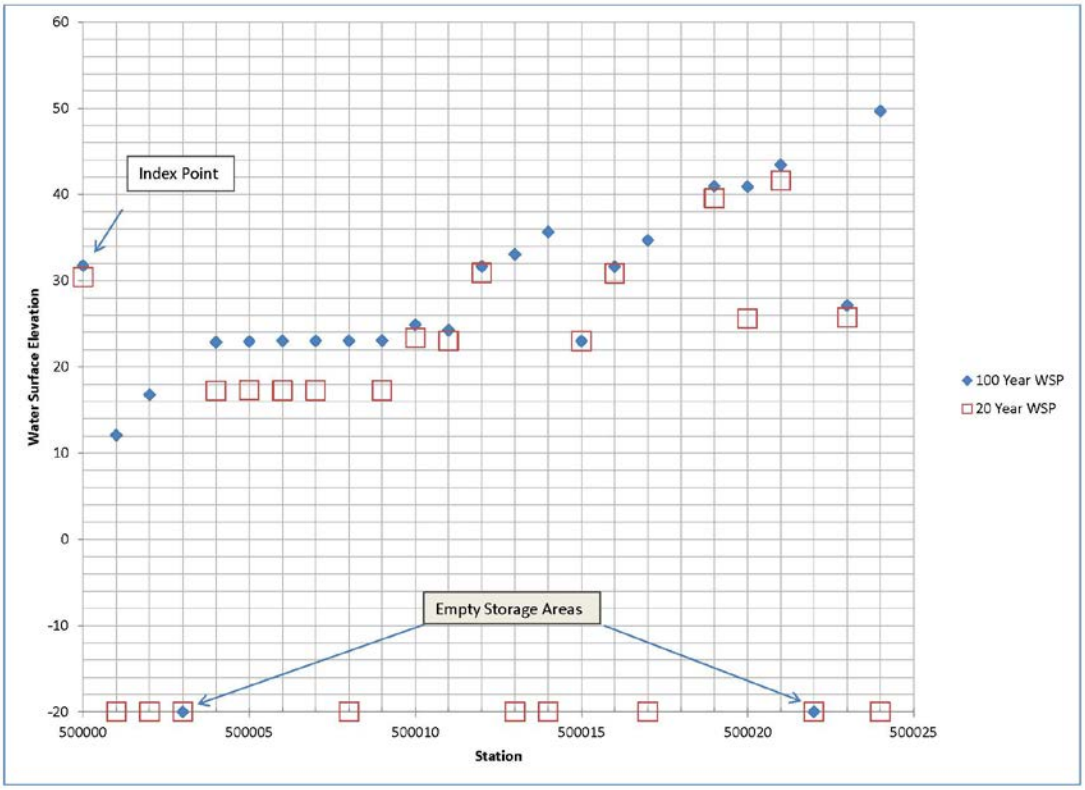


Figure 3: Example without Project WSP (5% and 1% AEP) for the Lower Puyallup Left Floodplain

2.4 Damage Curves

Damages in each damage reach are aggregated to each corresponding index point in the river. Figure 4 through Figure 6 below show aggregated stage-damage curves for the Lower Puyallup Left Floodplain damage reach (DR_63); the same reach and plan the WSP shown in Figure 2 and Figure 3 are based on. The correct aggregation of damages to the index point was a main area of concern with this scheme. Figure 7is the discharge-stage rating curve at the index point, where the 31.75 ft. elevation is the 1% AEP without project stage and 30.42 feet is the 5% AEP index point stage.

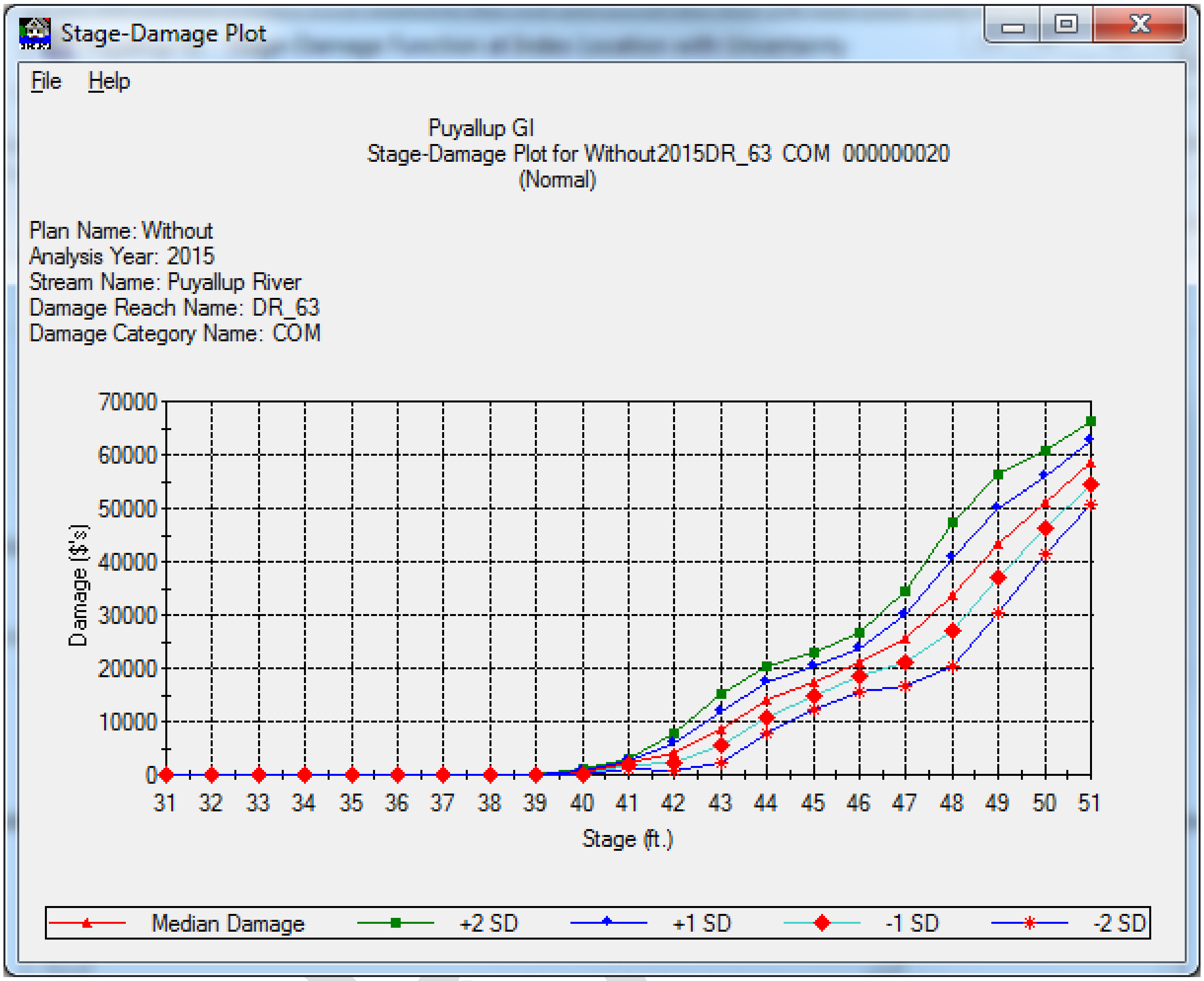


Figure 4: Aggregated Stage-Damage Curve for Lower Puyallup Left Floodplain (commercial)

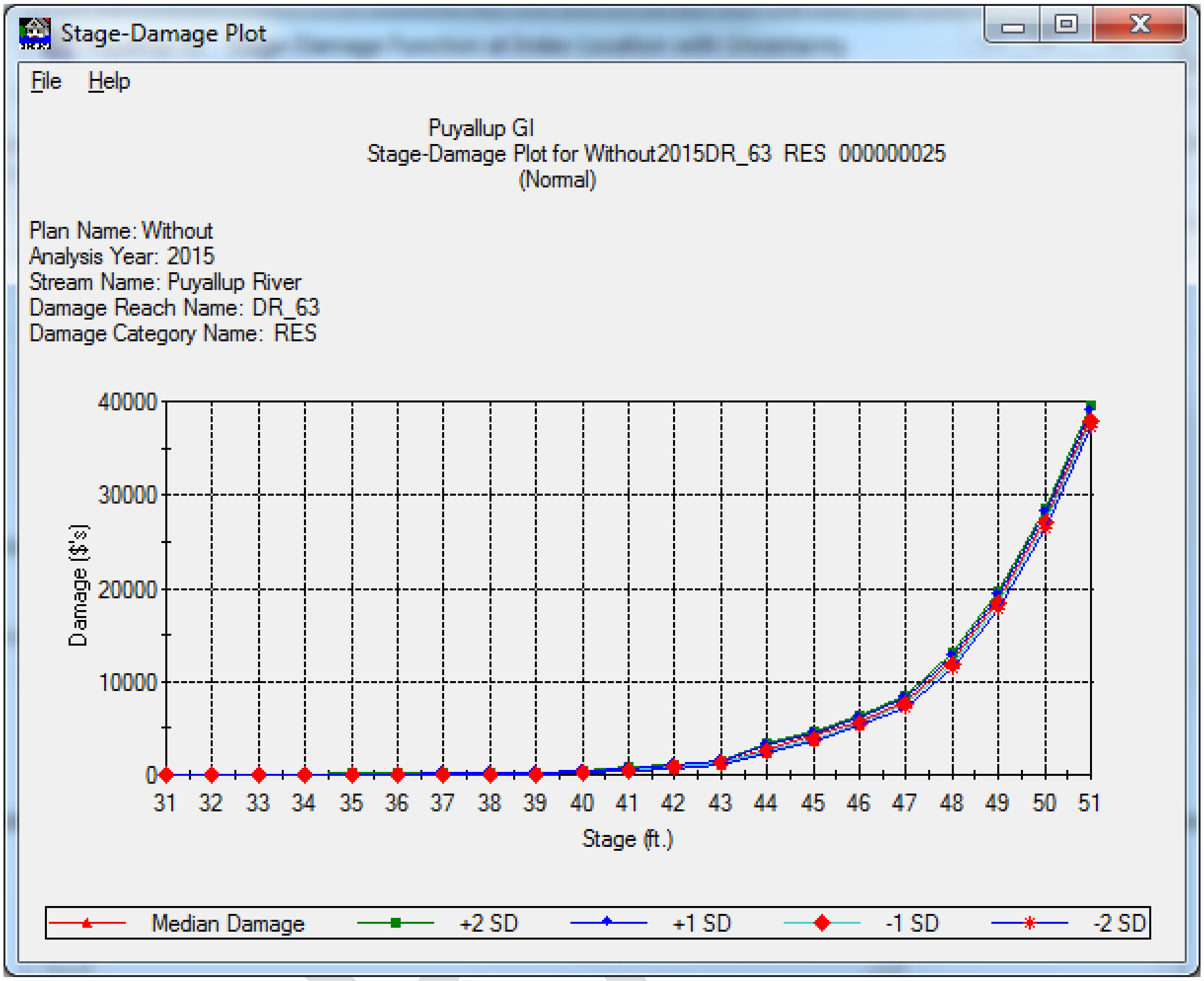


Figure 5: Aggregated Stage-Damage Curve for Lower Puyallup Left Floodplain (Residential)

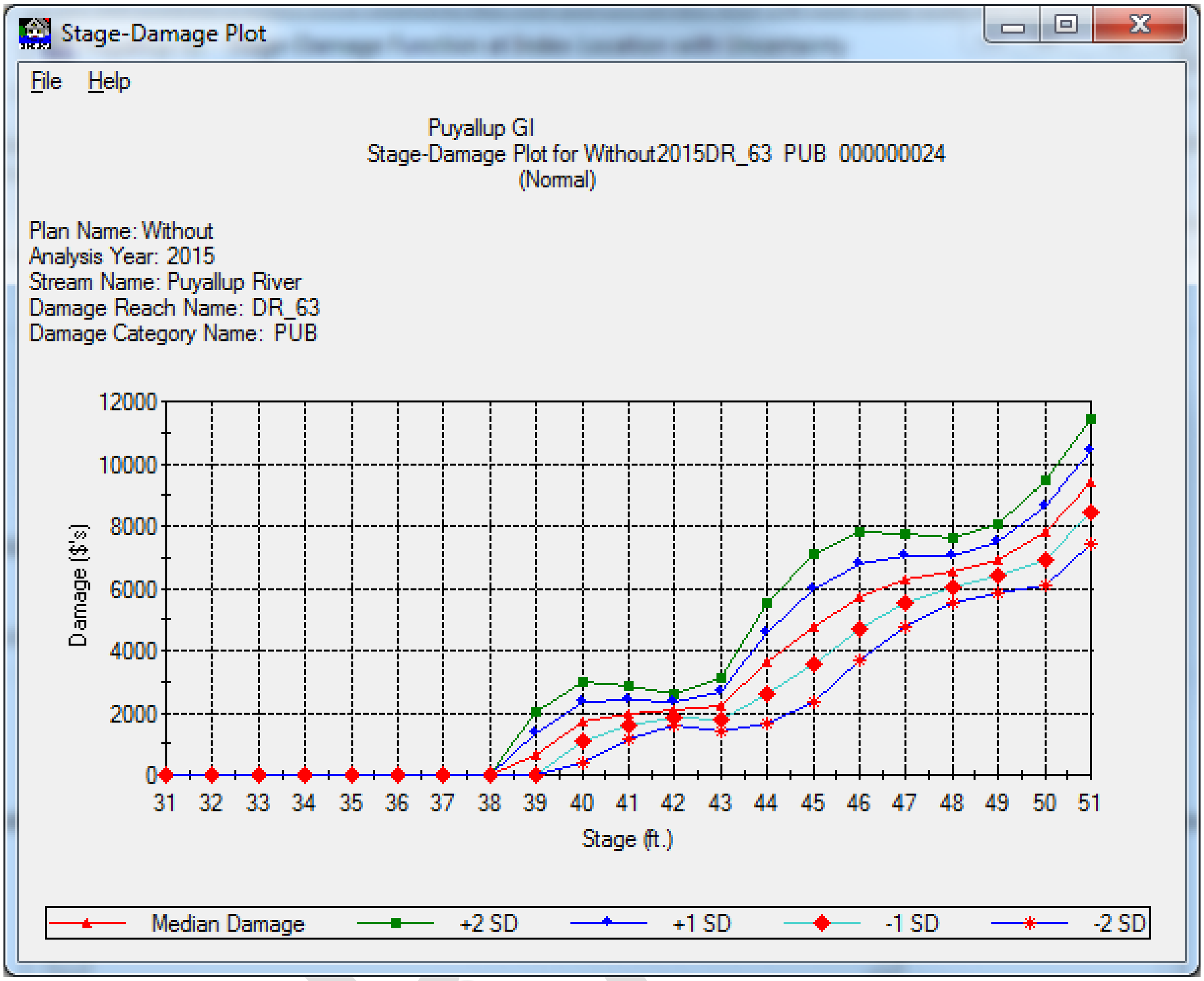


Figure 6: Aggregated Stage-Damage Curve for Lower Puyallup Left Floodplain (Public)

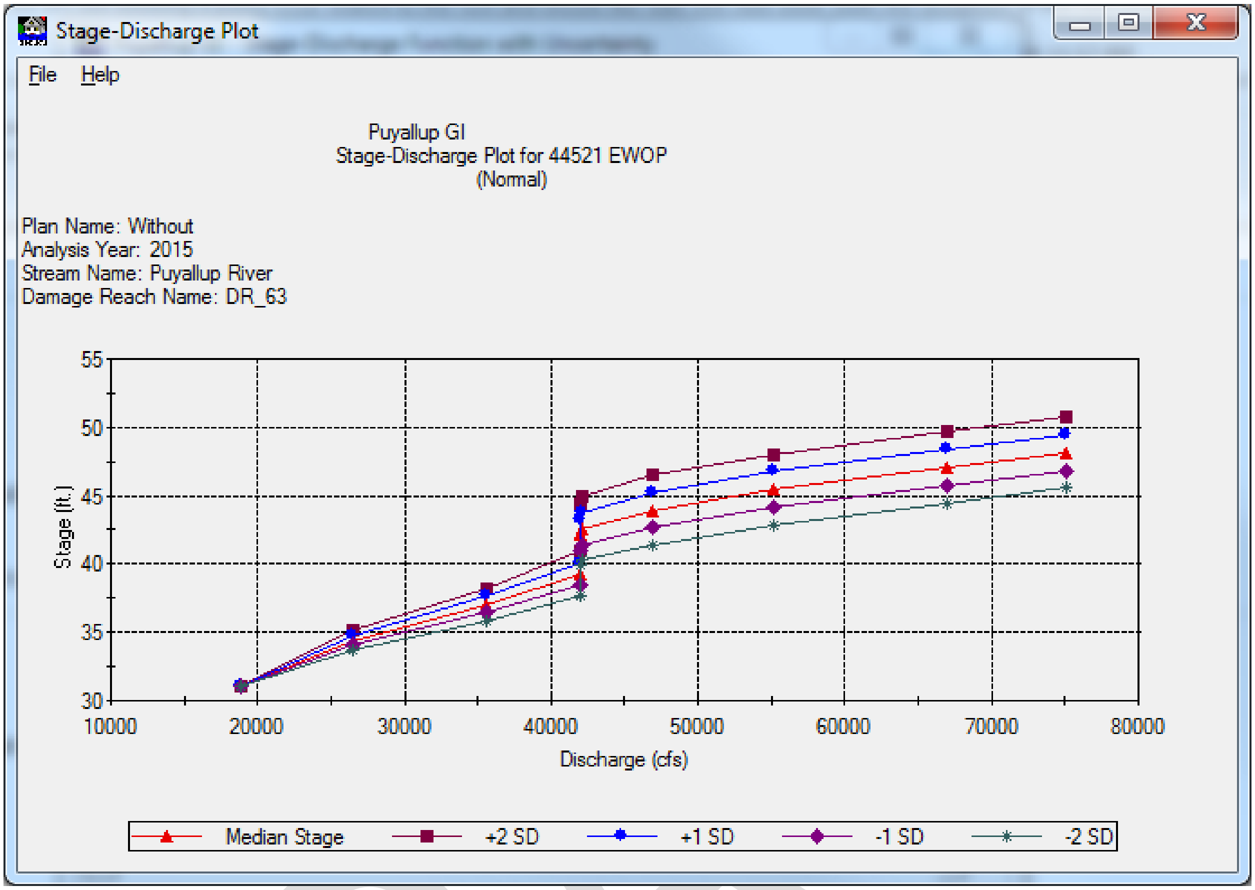
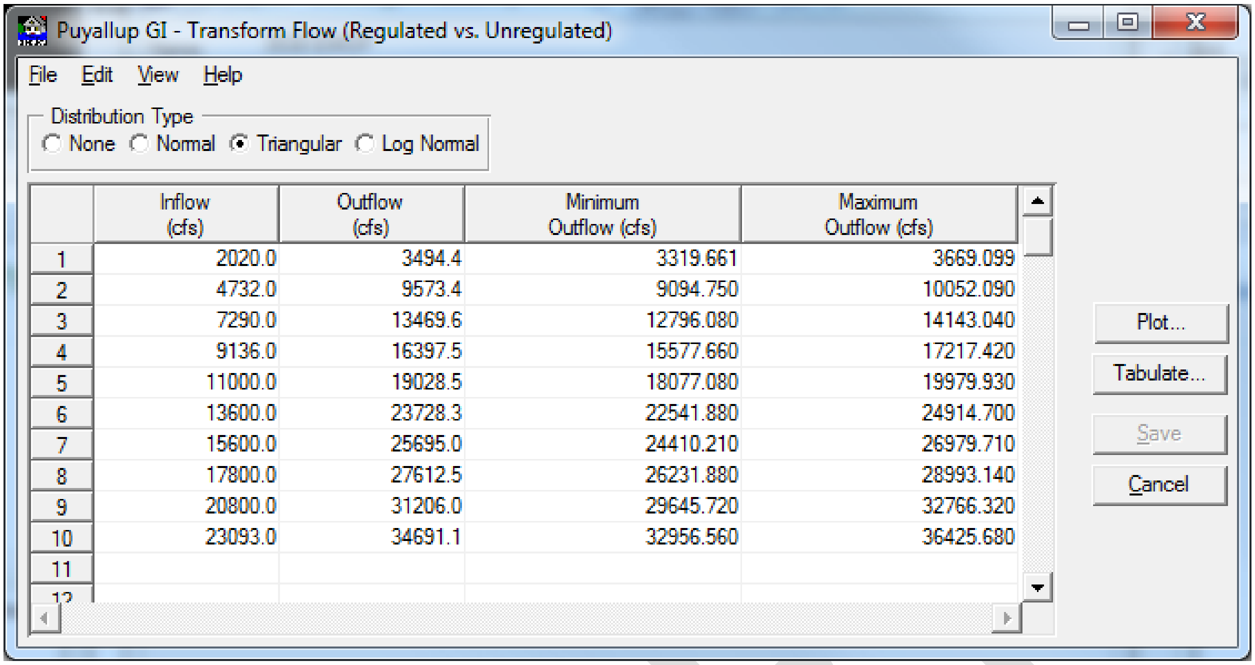


Figure 7: Discharge Stage Rating Curve for Lower Puyallup Left Floodplain DR_63 damage reach

2.5 Flow-Frequency Relationships

Incorporating the flow-frequency relationships on the unregulated portion of the study area was relatively simple. The analytical frequency curves from HEC-SSP, based on the flow record at the Carbon River at Fairfax and Puyallup River at Orting stream gages, were used as a starting point. Since damage areas (index points) were located away from the stream gages, the flow values from the analytical curves were paired with the routed flow values computed by the hydraulic model from the same flood event at a given damage area. Figure 8 is an example frequency curve from the FDA model for a damage location on the Carbon River.

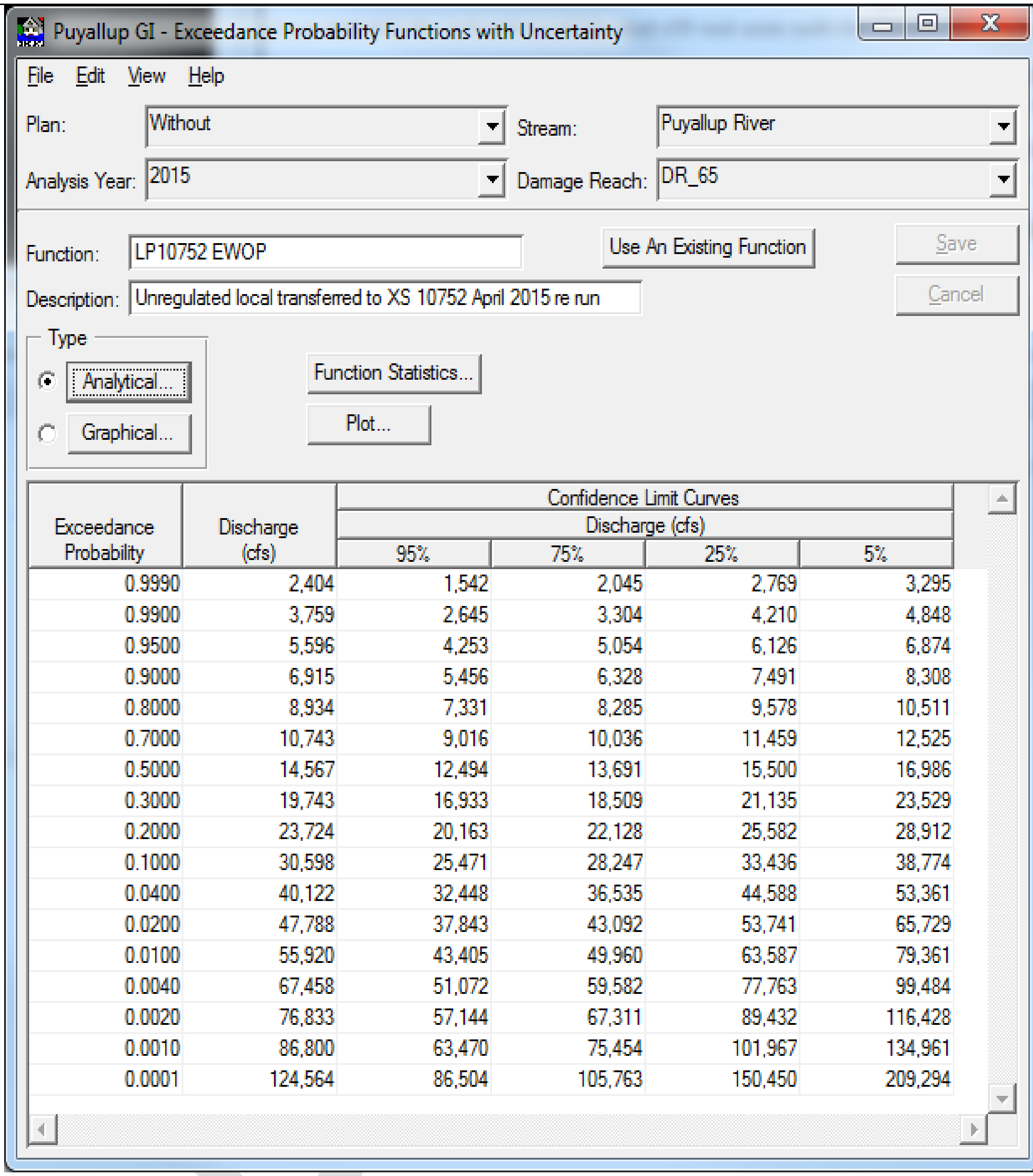


	Inflow (cfs)	Outflow (cfs)	Minimum Outflow (cfs)	Maximum Outflow (cfs)
1	2020.0	3494.4	3319.661	3669.099
2	4732.0	9573.4	9094.750	10052.090
3	7290.0	13469.6	12796.080	14143.040
4	9136.0	16397.5	15577.660	17217.420
5	11000.0	19028.5	18077.080	19979.930
6	13600.0	23728.3	22541.880	24914.700
7	15600.0	25695.0	24410.210	26979.710
8	17800.0	27612.5	26231.880	28993.140
9	20800.0	31206.0	29645.720	32766.320
10	23093.0	34691.1	32956.560	36425.680
11				
12				

Figure 8: Example Frequency Curve for a Location on Carbon River without Project

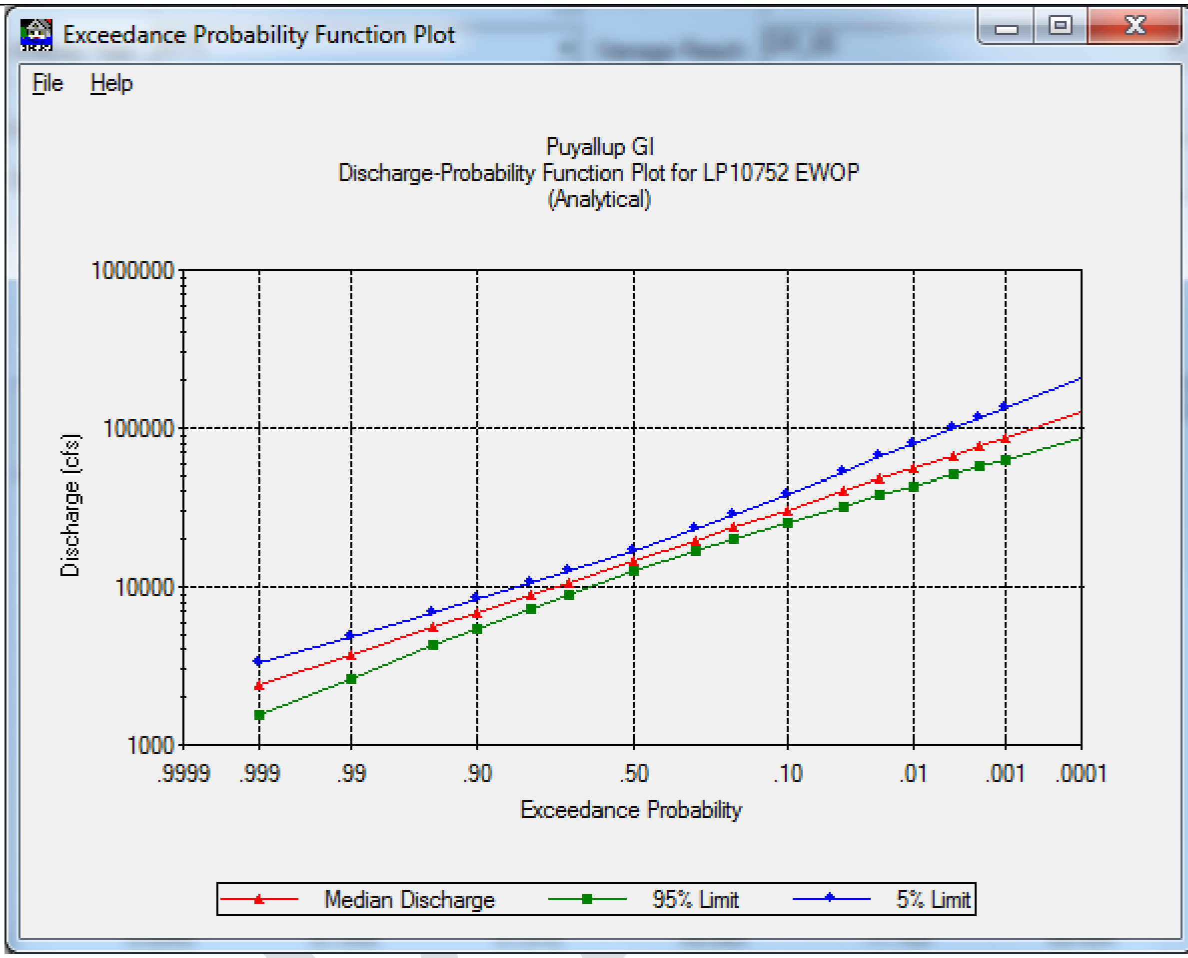
The left hand column in Figure 8 is the analytical peak flow frequency curve computed at the Carbon River at Fairfax streamgage. Each of the eight flow values represent the peak flows for the 50% through 0.2% AEP floods from the analytical frequency curve. These eight floods are each simulated individually in the hydraulic model. The right hand column in Figure 8 are the peak flow values as computed by the hydraulic model and incorporate incremental local inflow between the gage and the index point as well as any hydraulic effects such as floodplain storage, levee overtopping and channel form influences. Internal to FDA is also an uncertainty band on the frequency curve based on Bulletin 17B methodology.

Since the Lower Puyallup and White River portions of the study area are regulated by Mud Mountain Dam, the approach was a little different. For these locations the analytical portion of the frequency input is based on the unregulated local inflow. This is the aggregate inflow from the basin into the Lower Puyallup minus Mud Mountain Dam discharge (i.e. assumes the portion of the basin above Mud Mountain does not exist), which was explained in the hydrology discussion above. Figure 9 below is an example frequency curve from the FDA model for a damage location in the Lower Puyallup.



Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs)			
		95%	75%	25%	5%
0.9990	2,404	1,542	2,045	2,769	3,295
0.9900	3,759	2,645	3,304	4,210	4,848
0.9500	5,596	4,253	5,054	6,126	6,874
0.9000	6,915	5,456	6,328	7,491	8,308
0.8000	8,934	7,331	8,285	9,578	10,511
0.7000	10,743	9,016	10,036	11,459	12,525
0.5000	14,567	12,494	13,691	15,500	16,986
0.3000	19,743	16,933	18,509	21,135	23,529
0.2000	23,724	20,163	22,128	25,582	28,912
0.1000	30,598	25,471	28,247	33,436	38,774
0.0400	40,122	32,448	36,535	44,588	53,361
0.0200	47,788	37,843	43,092	53,741	65,729
0.0100	55,920	43,405	49,960	63,587	79,361
0.0040	67,458	51,072	59,582	77,763	99,484
0.0020	76,833	57,144	67,311	89,432	116,428
0.0010	86,800	63,470	75,454	101,967	134,961
0.0001	124,564	86,504	105,763	150,450	209,294

a. Exceedance Probability Table



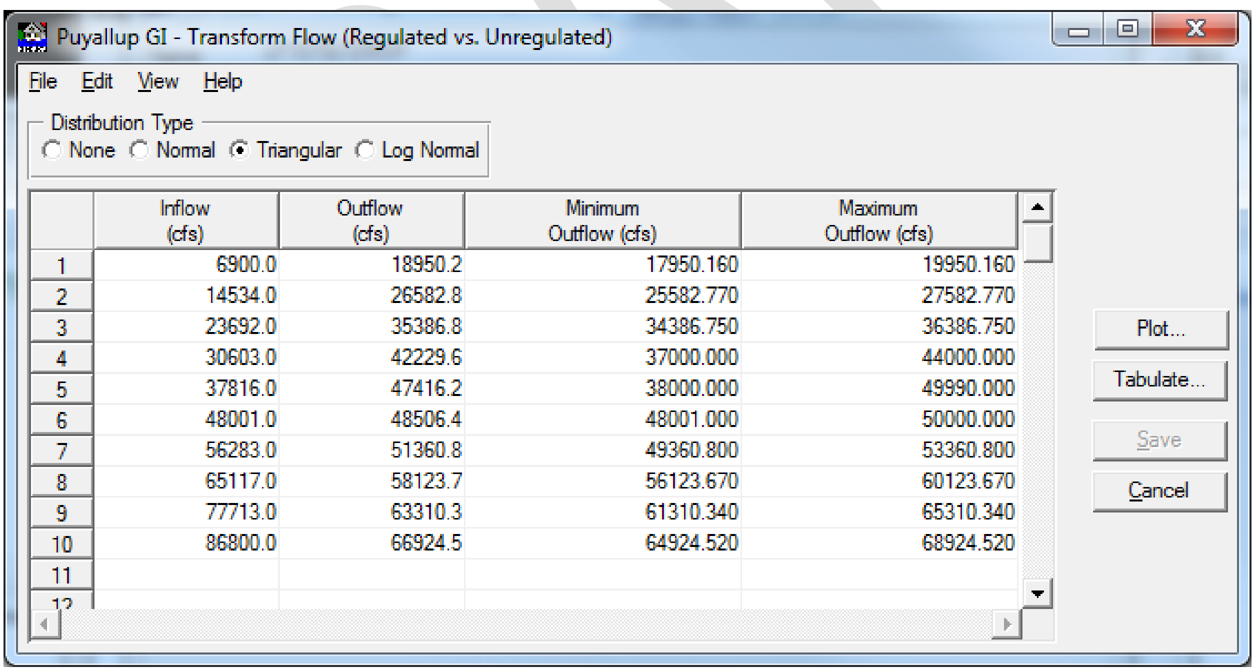
b. Exceedance Probability Plot

Figure 9: Example Frequency Curve for a Lower Puyallup Location without Project

2.6 Flow Transforms

For this study the hydrologic inputs to the HEC-FDA model used the transform feature of the program. This allows us to base the hydrologic probability for all conditions on a consistent set of analytical frequency curves described previously to ensure that hydrologic probability and uncertainty is correct for each condition. The alternative is to use the hydraulic model output directly and use a graphical approach to the uncertainty. Past experience with the graphical approach has indicated that there can be inconsistencies in the uncertainty from condition to condition. The transform approach lets us specify a peak flow value from an analytical frequency curve (this stays same for all conditions) and then specify another corresponding flow value from the hydraulic model (this can vary from condition to condition) that is based on factors like local inflow, the influence of levee overtopping and sediment deposition (this can vary from condition to condition). Figure 10 is the transform function used for the existing without project condition at a location in the Lower Puyallup Reach. The “Inflow” column represents the 50% through 0.2% AEP values from an analytical frequency curve.

In Figure 10 the far left column represents peak flow values from the unregulated local inflow frequency curve. As with the Carbon River example (Figure 8) the ‘Outflow’ values are the peak flow values as computed by the hydraulic model for the same return interval flood event. The Outflow column is equivalent to the regulated values in the Lower Puyallup frequency curve. The values track well up to and including the 2% AEP (row 6 in Figure 10). Above the 2% AEP event the values in Figure 9 are significantly Lower than the corresponding values in the regulated values in the Lower Puyallup frequency curve. This is because the channel capacity of the Lower Puyallup is generally 50,000 cfs completely full. For the 1% AEP event (row 7) the model has approximately 61,000 cfs entering the reach but not all of this flow makes it down to the damage location Figure 10 is depicting (at the lower end of the reach close to the I-5 Bridge). The hydraulic model has the excess water overtopping levees and flowing through and being stored in the floodplain. Another item of note with Figure 10 is the uncertainty band shown-the minimum and maximum outflow columns. As with the Carbon example, internal to FDA is an uncertainty band associate with the analytical frequency curve in the left column.



	Inflow (cfs)	Outflow (cfs)	Minimum Outflow (cfs)	Maximum Outflow (cfs)
1	6900.0	18950.2	17950.160	19950.160
2	14534.0	26582.8	25582.770	27582.770
3	23692.0	35386.8	34386.750	36386.750
4	30603.0	42229.6	37000.000	44000.000
5	37816.0	47416.2	38000.000	49990.000
6	48001.0	48506.4	48001.000	50000.000
7	56283.0	51360.8	49360.800	53360.800
8	65117.0	58123.7	56123.670	60123.670
9	77713.0	63310.3	61310.340	65310.340
10	86800.0	66924.5	64924.520	68924.520
11				
12				

Figure 10: Flow Transform for the Lower Puyallup Existing Without Project

The uncertainty associated with the analytical curve is input separately into FDA. Also included is an “extreme” event point as well (greater than a 0.2% AEP event) to allow FDA to sample during the Monte Carlo simulations events that might fall towards the upper end of the 0.2%

AEP event uncertainty band. The “Outflow” column is based on the flow value computed by the hydraulic model at a particular index point (cross section in the hydraulic model) for the simulation of a particular return interval and condition flood event. The “Minimum Outflow” and “Maximum Outflow” columns are estimates of the upper and lower bounds for the modeled flow values or values in the “Outflow” column. Factors driving this uncertainty include uncertainty in the incremental inflows included in the hydraulic model and the computed flow rates over levees and other structures adjacent to the river that either take water out or put it back into the river. Along the White River and Lower Puyallup River there is an added uncertainty associated with the operation of Mud Mountain Dam. For locations along the Carbon, Upper Puyallup and Middle Puyallup reaches, this uncertainty has been quantified as plus or minus 5% of the modeled flow. The analysis can be refined for later phases of this study. The single percentage approach does not take into account that, throughout the range of flood events simulated, the proportion added by each uncertainty source adds is likely different. For example, the uncertainty in computed flows over levees is really not a factor in the overall uncertainty at very low flows where water is contained in the channel but it could be a large factor at high flows where a significant amount of levee overtopping occurs. There is likely some overlap in the transform uncertainties and the uncertainty applied to the discharge-stage relationship discussed in the next section.

For the White and Lower Puyallup reaches the same uncertainty sources are present but there is an added source related to the operation of Mud Mountain Dam. Analysis of past regulation shows that during many events, flows have been reduced more than required (in hindsight) to keep flow in the Lower Puyallup from reaching 50,000 cfs-the recommended limit called for in the Water Control Manual. To capture this the uncertainty is skewed to the low side where appropriate. Note row five in Figure 10. The unregulated local inflow is approximately 38,000 cfs. Theoretically, for a flood event of this magnitude, MMD releases could be as much as 12,000 cfs and still keep the Lower Puyallup below 50,000 cfs. However to reflect factors such as conservative operation, changing forecasts, etc., the Lower bound on the uncertainty is reduced to 38,000 cfs to capture the possibility that the MMD operator might drastically reduce outflows for a number of reasons. However, for a larger flood of the magnitude depicted in row 7 of Figure 10 it is assumed that the magnitude would be apparent to a dam operator and everything possible would be done to reduce outflow from MMD during the peak of the unregulated local inflow.

In the study area, the analytical frequency curves come from three locations. Damage reach index points on the upper and middle Puyallup reaches use flow-frequency statistics from the Puyallup near Orting USGS gage (12093500) and locations along the Carbon River use the Carbon River at Fairfax USGS gage (12094000). Locations along the White and Lower Puyallup reaches, which are regulated by Mud Mountain Dam, use an analytical frequency curve based on the unregulated local flow. Inherent in this approach is the assumption that if a given return interval flood occurs at the Fairfax or Orting gages it occurs everywhere along the Carbon, Middle and Upper Puyallup Rivers. Similarly, it is assumed that a given return interval flood occurs everywhere along the White and Lower Puyallup reaches as well.

The unregulated local flow includes the entire basin except the contribution above MMD. This is a computed value, estimated by taking peak flow values at the Puyallup at Puyallup USGS gage (12101500) and subtracting the corresponding MMD outflow. Another way to think of the unregulated local flow used for FDA is to consider what the flow in the Lower Puyallup would be without the MMD discharge. The unregulated local inflow is used on the White and Lower Puyallup locations because MMD discharge releases are generally based on what the unregulated local flow at Puyallup. For example, if it is thought during a flood event that flow in the Lower Puyallup (i.e. Puyallup at Puyallup gage) is going to be about 40,000 cfs without the MMD contribution then theoretically the MMD discharge could be 10,000 cfs and the flow in the Lower Puyallup would not exceed 50,000 cfs. Alternatively if MMD regulation predicts the local unregulated inflow is going to be greater than 50,000 cfs, then if possible, the MMD outflow would be adjusted to zero to coincide with the time this peak is anticipated to occur.

2.7 Stage-Discharge Uncertainty

Figure 11 shows an example of the stage-discharge relationship used for the existing without project condition at a location in the Lower Puyallup reach. The flow values and water surface elevations are as computed by the hydraulic model. At a given location, this relationship varies between the different scenarios. The uncertainty is characterized using a standard deviation. Here a standard deviation value of 1.3 feet was initially used for the 1% AEP condition based on guidance in EM 1110-2-1619 table 5-2 for field surveyed cross sections. It could be argued this value should be lower based upon the relatively good calibration of the hydraulic model at locations of observed data. The model comparison with the USGS Puyallup at Puyallup streamgage seems to be well calibrated to the existing condition using historic data. A conservative value of 1.3 was chosen from the table, based on engineering judgment, because the type of event that causes significant damage has not been observed for the study area and it is unknown how well the model predicts water surfaces for an event where significant levee overtopping occurs. Roughness influences river stage which drives overtopping of levees. This uncertainty is transferred through the FDA analysis to the water surface profiles used for each storage area, for which there is large uncertainty supporting the conservative value of 1.3. In summary, the model appears to be well calibrated but the model performance for floodplain areas landward of levees is largely unknown for large events.

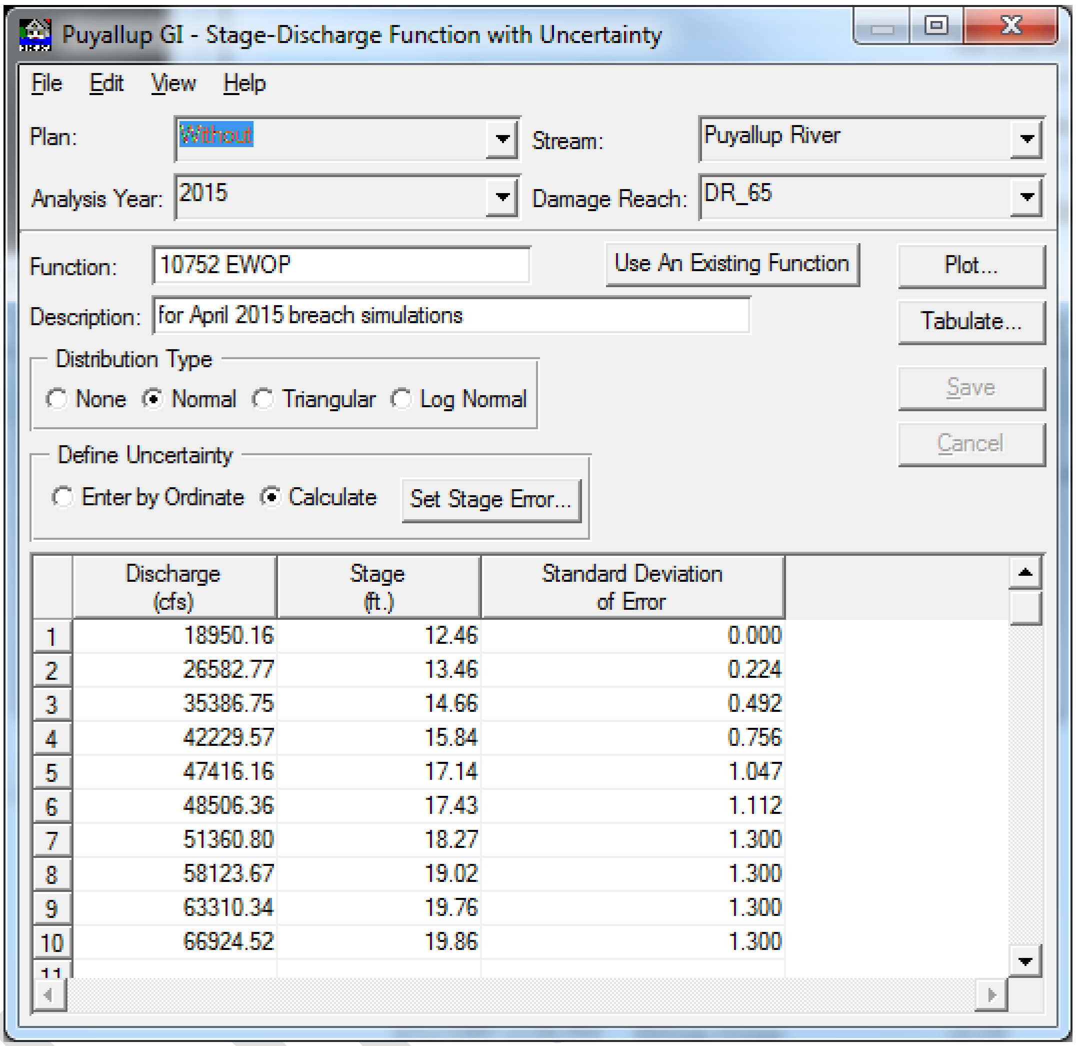


Figure 11: Example Discharge-Stage Rating Curve at Lower Puyallup reach

The largest source of uncertainty for future conditions may be changes in bed elevations from sediment deposition. For the future condition where sediment deposition is expected to occur, the standard deviation is increased to 1.6 feet to account for this additional uncertainty. This value is expected to be refined for NED optimization of the TSP when more detailed sediment modeling is completed for future conditions and a range of uncertainty can be bracketed with the sediment model.

2.8 Top-of-Levee Elevations

Upon inclusion of the top-of-levee elevations for each levee (which is used to estimate the level of protection considering uncertainty), the level of protection output for some damage reaches gave unreasonable performance numbers, for example an annual exceedance probability (AEP) of 0.01%. This appears to be due to selection of the top-of-levee elevation exactly at the cross section representing the index point. Further inspection showed that, in some cases, the incipient overtopping location actually lies upstream or downstream of the index point cross section. In these cases, the AEP of the incipient overtopping event can be markedly different between these index point location and the overtopping location. This is illustrated in Figure 12.

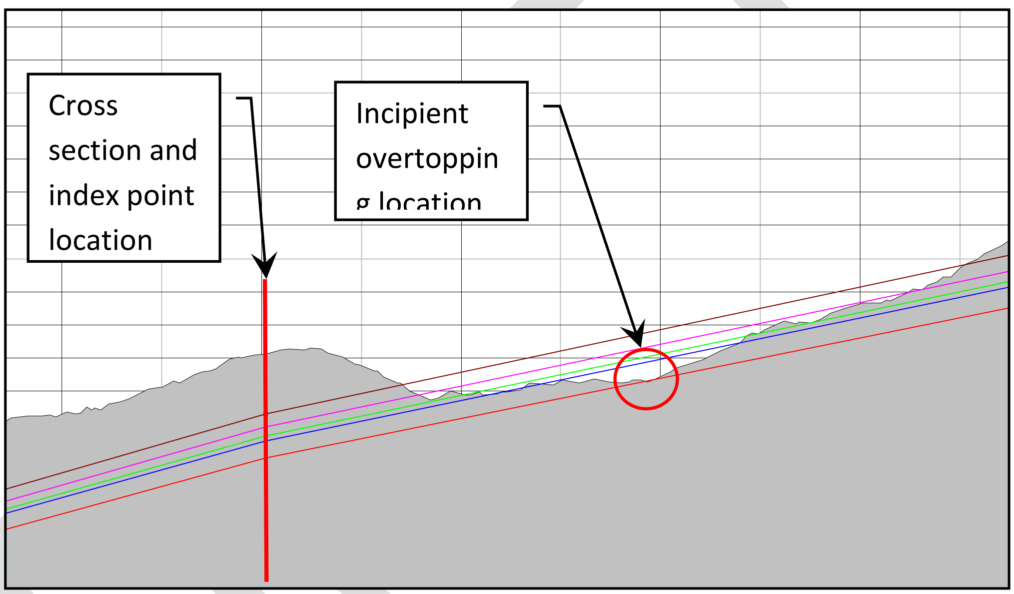


Figure 12: Example of Index Point Not Located at the Lowest Point on a Levee

If the top-of-levee elevation is higher or lower than the elevation of incipient overtopping the level of protection estimate will be skewed, as will the EAD estimate. The presence of levees, and their incipient overtopping, is reflected in the hydraulic model results. Although the EAD computation does not require a top of levee elevation to be included in FDA, the top of levee input is required to obtain performance output. Entering a top-of-levee elevation can override the hydraulic data (if it is higher than the actual incipient overtopping return interval), thus skewing the EAD computation. The top-of-levee elevation can be left out, but doing so causes the FDA program to use the target stage (which is based upon the event exceedance probability and the percent residual damages) to compute project performance. Target stage is the stage at which significant flooding begins. For a reach with a levee, the top of levee stage is the target stage. For all other reaches, the target stage is calculated from the exceedance probability and percent residual damages. By default, it is the stage that corresponds to the damage that is 5% of the total damage for the 1% AEP event. While leaving out the top-of-levee elevation does provide an indication of project performance, is the actual performance of the structure itself.

Several options were considered to remedy this situation: 1. Leave the top-of-levee elevation out and document the limitations of the computed performance values; 2. Input the actual elevation of incipient overtopping at each index point location by interpolating the water surface elevation between cross sections; or 3. Let FDA interpolate between cross sections surrounding the index points (index points would need to be changed). Each of these approaches may have other implications that need to be explored. Option number 2 was selected as a reasonable fix and was implemented for TSP. An example of project performance output as of TSP analysis is shown in Table 1 below.

Project Performance

File Help

Puyallup GI Project Performance
by Damage Reaches for the Without
(Without project condition Default) plan for Analysis Year 2015
(Stages in ft.)
Plan was calculated with Uncertainty
Version 1.4, Sep. 2014; Less Simple Method (0.010)

Without Project Base Year Performance Target Criteria:
Event Exceedance Probablility = 0.01
Residual Damage = 5.00 %

Stream Name	Stream Description	Damage Reach Name	Damage Reach Description	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
					Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
Carbon River	Stream added c	DR_79	Carbon Left Orting	135.87 L	0.0508	0.0721	0.5269	0.8941	0.9763	0.7192	0.4125	0.2597	0.1666	0.0739	0.0307
		DR_795	carbon left orting	192.11 L	0.0131	0.0196	0.1796	0.4478	0.6283	0.9954	0.8702	0.6308	0.3984	0.1695	0.0711
		DR_80	Carbon Left Voight C	305.00 L	0.9990	0.9984	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		DR_81	Carbon Right	332.63 L	0.9990	0.9984	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		DR_Carbon_River	Carbon River riverine	202.01 L	0.0036	0.0129	0.1221	0.3234	0.4786	0.9795	0.9254	0.8423	0.7179	0.4699	0.2728
Puyallup River	Stream added c	DR_60	Lower Puyallup Left t	17.75 g	0.0691	0.0733	0.5330	0.8982	0.9778	0.8842	0.7488	0.6244	0.4871	0.3308	0.2566
		DR_61	Clear Creek	27.75 L	0.0504	0.0354	0.3027	0.6609	0.8351	0.9420	0.6442	0.4195	0.2793	0.1936	0.1637
		DR_62	Puyallup River LB@	35.75 L	0.0512	0.0354	0.3027	0.6610	0.8352	0.9592	0.6558	0.3815	0.1929	0.0729	0.0365
		DR_63	Puyallup River lower	37.13 L	0.1939	0.1907	0.8795	0.9982	1.0000	0.1065	0.0332	0.0162	0.0056	0.0010	0.0002
		DR_64	Puyallup River lower	48.79 L	0.1194	0.1071	0.6779	0.9666	0.9965	0.4785	0.0796	0.0157	0.0028	0.0003	0.0001
		DR_65	LPuyallup RB below	18.50 g	0.0687	0.0736	0.5345	0.8991	0.9781	0.8709	0.7077	0.5522	0.3955	0.2365	0.1654
		DR_66	Hylebos Creek	39.59 L	0.0566	0.0405	0.3383	0.7103	0.8732	0.9436	0.5958	0.3102	0.1292	0.0334	0.0112
		DR_67	L Puyallup RB	39.59 g	0.2147	0.2021	0.8954	0.9989	1.0000	0.5701	0.3002	0.1423	0.0553	0.0137	0.0045
		DR_68	L Puyallup RB at Cor	39.13 L	0.8783	0.8822	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		DR_69	Middle Puyallup LB1	78.84 L	0.2258	0.2295	0.9262	0.9996	1.0000	0.0549	0.0053	0.0023	0.0018	0.0013	0.0010
		DR_70	Middle Puyallup LB2	101.65 L	0.4889	0.4696	0.9982	1.0000	1.0000	0.0016	0.0002	0.0001	0.0000	0.0000	0.0000
		DR_71	Middle Puyallup LB3	119.22 L	0.1410	0.1515	0.8066	0.9928	0.9997	0.3285	0.1247	0.0702	0.0420	0.0203	0.0104
		DR_72	SR 410	51.18 L	0.7626	0.7487	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		DR_73	Middle Puyallup Righ	67.01 L	0.0572	0.0703	0.5178	0.8879	0.9739	0.7837	0.2682	0.0741	0.0181	0.0041	0.0018
		DR_735	Middle Puyallup Righ	76.59 L	0.0022	0.0039	0.0379	0.1095	0.1758	1.0000	0.9975	0.9759	0.8888	0.6337	0.4052
		DR_74	Puyallup Carbon Jun	114.90 L	0.0999	0.1130	0.6986	0.9726	0.9975	0.5049	0.2015	0.0909	0.0420	0.0152	0.0061
		DR_75	U Puyallup Orting Le	217.11 L	0.0729	0.1065	0.6757	0.9659	0.9964	0.5479	0.4093	0.3356	0.2856	0.2087	0.1468
		DR_76	Upper Puyallup RB H	130.10 L	0.1609	0.1475	0.7974	0.9917	0.9997	0.2140	0.0540	0.0176	0.0068	0.0020	0.0008
		DR_77	Upper Puyallup RB F	233.94 L	0.0533	0.0844	0.5859	0.9290	0.9878	0.6275	0.4348	0.3377	0.2789	0.1980	0.1355
		DR_78	Upper Puyallup RB C	249.58 L	0.9990	0.9985	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		DR_Clear_Crk_Riv	Clear Creek riverine	26.73 L	0.0001	0.0003	0.0034	0.0103	0.0170	1.0000	0.9993	0.9954	0.9861	0.9717	0.9623
		DR_Low_Puyallup	Below White River ri	31.99 L	0.0065	0.0140	0.1317	0.3454	0.5065	0.9924	0.8996	0.7525	0.5942	0.4412	0.3739
		DR_Mid_Puyallup	Between Carbon and	91.70	0.0944	0.1044	0.6678	0.9634	0.9960	0.5248	0.1118	0.0274	0.0067	0.0012	0.0004
		DR_Mid_Puy_RR	Middle Puyallup RR	63.31	0.0328	0.0413	0.3442	0.7180	0.8787	0.9480	0.5768	0.2707	0.1076	0.0280	0.0089
		DR_Upp_Puyallup	Upper Puyallup riveri	237.24 L	0.9990	0.9985	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Tidal	Stream added c	DR_Tidal	Tidal	11.35	0.9990	0.9990	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
White River	Stream added c	DR_82	Lower White RB Vall	52.45 L	0.0523	0.0605	0.4641	0.8461	0.9558	0.7741	0.5039	0.3888	0.2511	0.0981	0.0382
		DR_83	Lower White RB	58.73 L	0.0014	0.0027	0.0266	0.0778	0.1262	0.9983	0.9933	0.9847	0.9430	0.7827	0.6037
		DR_84	Pacific RB	74.76 L	0.9990	0.9980	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		DR_85	Lower White RB abo	94.85 L	0.5485	0.4272	0.9962	1.0000	1.0000	0.3079	0.2928	0.2838	0.2562	0.1799	0.1157
		DR_86	Lower White LB Sum	45.06	0.5784	0.5716	0.9998	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		DR_87	Lower White LB	47.95 L	0.9990	0.9980	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		DR_88	White River County l	83.80 L	0.0001	0.0001	0.0010	0.0030	0.0050	1.0000	1.0000	1.0000	1.0000	0.9999	0.9999
		DR_Wht_Riverine	White Riverine	82.03 L	0.0001	0.0180	0.1664	0.4207	0.5974	0.9363	0.8958	0.8827	0.8683	0.8298	0.7909
		DR_Wht_Riv_Ind	White River industria	80.50 L	0.0001	0.0001	0.0010	0.0030	0.0050	1.0000	1.0000	1.0000	1.0000	0.9999	0.9999

***** - Computations have not been completed.
\+ - Something has changed and computations need to be redone.

Table 1: Example of Performance Output from FDA

3.0 FDA Discussion

For the without project condition, the modeling results have shown water entering the floodplain from multiple locations for large flood events. For example, a floodplain area adjacent to an existing levee (a storage area in the RAS model) can be flooded but the actual source location can be a point in the river upstream or downstream of the storage area without the adjacent levee being overtopped. The multiple overtopping source locations and the complex floodplain flow paths make it difficult to select index points. However following the methods described in section 2.2 for treating each damage reach as a pseudo-stream with a water surface profile made up of storage areas the complexities were largely reduced and the FDA model results made sense.

The different river/floodplain water surfaces, and the different floodplain water surfaces laterally from the streams, have been the biggest issues in configuring the FDA model. The FDA modeling approach adequately captured the big picture processes of the system. It allowed for different river and floodplain water surfaces and variation of water surfaces across the floodplain. It also allowed for the capture of water movement through the floodplain which is not parallel to that of the main streams. It is felt that improvement on the existing modeling approach would most likely be accomplished using a true two-dimensional model. This was not necessary for feasibility design; however it is conceivable that at later design phases such an improvement could be made if the improved accuracy in capturing damages is necessary. This has been identified in the risk register with the recommendation for follow-up in final feasibility.

Generally the FDA results based upon the discussion above reflect what would be expected for most damage reaches. The reader is referred to the Economic Appendices for further detail of FDA results. The model simulations used to derive FDA inputs are based on best estimates of individual model components to characterize flood conditions carried out using SMART Planning. Examination of study area hydrologic data from past floods indicates quite a variation in spatial magnitude, timing, and duration of flows. Furthermore, the active sediment transport in the study area makes model calibration and future conditions characterization difficult. Sediment modeling for the NED phase will help to reduce some of the uncertainty as estimations can be made for stage uncertainty from the modeling, which will likely vary by reach). To the extent possible, the uncertainties associated with these inputs have been incorporated in the risk-based (FDA) analysis and satisfactory results for EAD were determined. Refinements are expected to be made as the TSP is optimized for NED.

4.0 Next Steps

Several items were identified in the FDA analysis completed for TSP that may require further evaluation for the final Feasibility Study completion.

- Refinement of weir coefficients in the hydraulic model. The default coefficient of 2 was used for TSP modeling efforts. More accurate determination of the weir coefficient typical for most levees in this system (with gravel road surface and sloping, vegetated sides) will be considered for post-TPS modeling for feasibility. The HEC RAS manual cites 2.6-3.1 coefficient as typical for a broad crested weir. Levees with significant vegetation may have lower values. The Federal Highway Administration has produced several documents for reference in selecting coefficients.

- Evaluation of standard deviation used in estimating stage uncertainty in the FDA model. A more robust effort has been recommended defining incremental sources of uncertainty (due to roughness, sediment deposition, etc.). Sediment modeling will be completed after TSP and will add clarity to selection of appropriate standard deviation. It may be necessary to re-evaluate flow transforms (i.e. flow uncertainty) also if sedimentation significantly changes overtopping of levees and affects outflow at each index point.
- The Final Feasibility design will be based on the upcoming NED analysis. After the NED plan has been determined, the conditional non-exceedance probability (CNP, or assurance) of this plan will be computed. This CNP could be different from the CNP associated with the TSP. As part of Smart Planning, the TSP levee design profiles used a preliminary 1% AEP water surface profile plus 3 feet of residual levee height as a starting point (in the future condition). This was vetted with NWD/NWS management at study IPR's.
- For simplification of importing data from HEC RAS to HEC FDA, storage areas in the hydraulic model can be re-named to correspond to structure stationing in the FDA model.