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Lower Raging River
Hydrologic and Hydraulic Analysis

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DISCLAIMER

CREDITS AND ACKNOWLEDGEMENTS

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APPENDICES

1 INTRODUCTION

King County (County) retained a team led by Shannon & Wilson to provide engineering services for the Levee Breach Analysis Mapping and Risk Assessment Project (Project). The Project is based on recommendations presented in the Levee Breach Analysis for King County Rivers final report (Watershed Science and Engineering, 2019). The scope of work includes collecting data, developing hydraulic models and simulating levee breaches, mapping the resulting inundation, and conducting risk analysis for six levee containment systems in three rivers (Lower Raging River, Lower Tolt River, and South Fork Snoqualmie River). The findings from the Project will be used to update capital project planning strategies and emergency planning efforts.

1.1 Purpose of Report

Northwest Hydraulic Consultants (NHC) is the hydraulics lead for the Project and is leading the hydraulic modeling of the Lower Raging and Lower Tolt rivers, as well as providing review and oversight of the South Fork Snoqualmie River model (which is being developed by Tetra Tech, another Project team member). This report documents the development of hydrologic inputs, including a future conditions analysis, and a hydraulic model of the Lower Raging River and comprises the technical memorandum deliverable under King County contract E00670E20, Task 300, Subtasks 5 and 6. In this document, all references to return period floods are based on existing or historical conditions, unless otherwise indicated as a future condition (FC). The levee breach analysis and resulting risk mapping will be documented in a future report.

The study area for the Lower Raging River (LRR) includes existing levee and revetment systems from the 328th Way SE bridge to the mouth of the Raging River (confluence with Snoqualmie River), as depicted in Figure 1.1. The King County levee and revetment facilities are approximately 1.5 miles long and are present along the left and right banks of the river. The levees protect residential, commercial, and agricultural properties and infrastructure from flood hazards within the unincorporated community of Fall City.

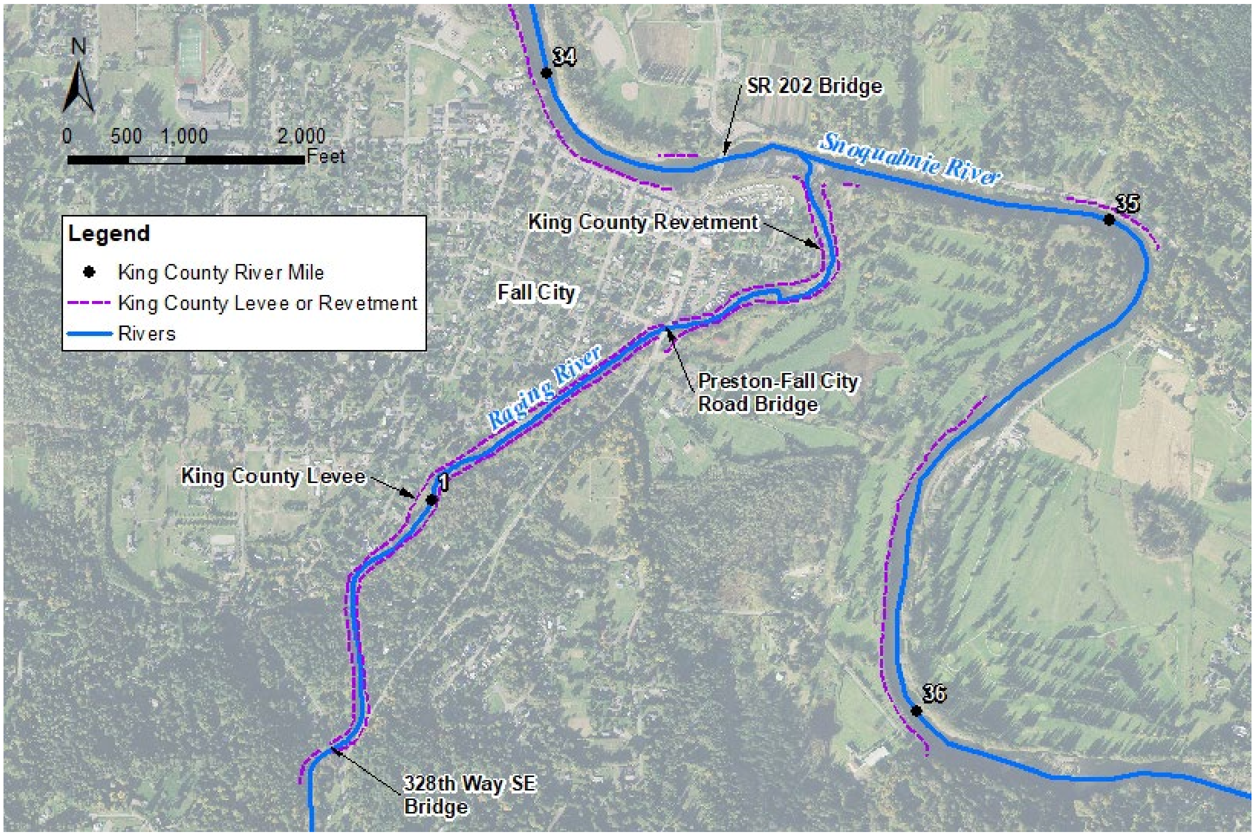


Figure 1.1 King County river facilities along the Lower Raging River

1.2 Prior Studies

NHC reviewed existing studies and models of the LRR and Snoqualmie River for their applicability to the Project. Studies reviewed and relevant findings are described below:

- The FEMA Flood Insurance Study (FIS) (FEMA, 2020c) documented hydraulic modeling of the Lower Raging River (Harper Righellis, Inc., 1993) using the U.S. Army Corps of Engineers (USACE) HEC-2 backwater computer program. The study limits extended from the confluence with the Snoqualmie River to approximately 0.6 miles upstream of Interstate 90. Cross sections in the HEC-2 model were based on a topographic field survey of the main channel conducted in 1993; the main channel elevations were supplemented with planimetric/topographic data to capture the entire study area. The river was mapped as a detailed Zone AE with a regulatory floodway (FEMA, 2020a, 2020b). NHC obtained cross-section information from the HEC-2 model and compared elevation data to recent topobathymetry, discussed in Section 3.1.
- The FEMA FIS (FEMA, 2020c) also documented the effective analysis of the Snoqualmie River (NHC, 2006), performed in 2006 using a one-dimensional (1D) HEC-RAS model. The limits of the FIS study cover the Snoqualmie River from the Snoqualmie River near Snoqualmie USGS gage (12144500) at the upstream end to the confluence with the Snohomish River at the downstream

end. NHC used the upper portion of this model for construction of the joint-coincidence model, discussed in Section 2.4.2 of this report.

- The Snoqualmie River Hydraulic and Hydrologic Study (Watershed Science and Engineering, 2016) was conducted to analyze impacts of recent flood mitigation projects upstream of Snoqualmie Falls. The study used the effective FIS model to examine changes at the falls without modification to downstream portions of the model. Results from the analysis showed minimal change downstream of Snoqualmie Falls – an increase of 0.1 feet to the peak 100-year water level. As no changes were made to the effective model in the LRR Project area, NHC did not need to incorporate information from this source.
- Watershed Science and Engineering performed hydraulic modeling of the Snoqualmie River for the Fall City Restoration Project (Watershed Science and Engineering, 2020). WSE used RiverFlow2D to analyze the Snoqualmie River from river mile (RM) 28 to 37. The model domain also included the lower one mile of the Raging River. NHC obtained geospatial data from the model and used breakline and landcover coverages to partially develop the 2021 Lower Raging River model. Section 3 discusses the hydraulic model development in detail.

Following review of these studies and models, NHC determined that the existing hydrologic and hydraulic analyses for the LRR were limited and outdated. While some information was available from historic cross-sections and the 2020 Fall City Restoration model, it was determined through consultation with King County that new hydrologic and hydraulic analyses would be required for the LRR study area. This report documents the development of balanced hydrographs for the LRR for existing and future conditions, and the development of an up-to-date hydraulic model of the LRR. Future work will examine the impact and risks associated with hypothetical levee breaches in the LRR system.

2 HYDROLOGY

The hydrologic approach for the LRR assessment requires significant new analyses, as existing studies on the Raging River are outdated. This section documents the development of updated hydrology, including watershed conditions; landcover considerations; flood frequency analysis of the USGS Raging River gage (12145500); historic event considerations; balanced flood hydrographs for various return intervals; a joint-coincidence analysis of floods from the LRR and Snoqualmie River systems; and the application of climate change projections to develop future conditions balanced hydrographs for the LRR.

2.1 Watershed Characteristics

The Raging River is a tributary to the Snoqualmie River, with a total watershed area of 32.9 square miles at the mouth, per the FEMA FIS (FEMA, 2020c). NHC obtained the LRR drainage basin boundary from the King County GIS Center (King County, 2021), as depicted in Figure 2.1. The watershed originates at Rattlesnake Mountain, approximately eight miles southeast of Fall City. Major tributaries to the Raging River include Canyon Creek, Deep Creek, and Icy Creek. The overall basin contains relatively steep gradients with a maximum elevation of 3,500 feet in the uplands that transitions to an elevation of 100 feet near Fall City. On average, the watershed receives 75.8 inches of precipitation annually (PRISM Climate Group, 2019). A majority of the upper watershed consists of forested land as displayed in

Figure 2.1 (mostly owned by the Department of Natural Resources). Landcover in the LRR floodplain study area is dominated by urban development in Fall City and mixed forest/herbaceous coverage along the river corridor.

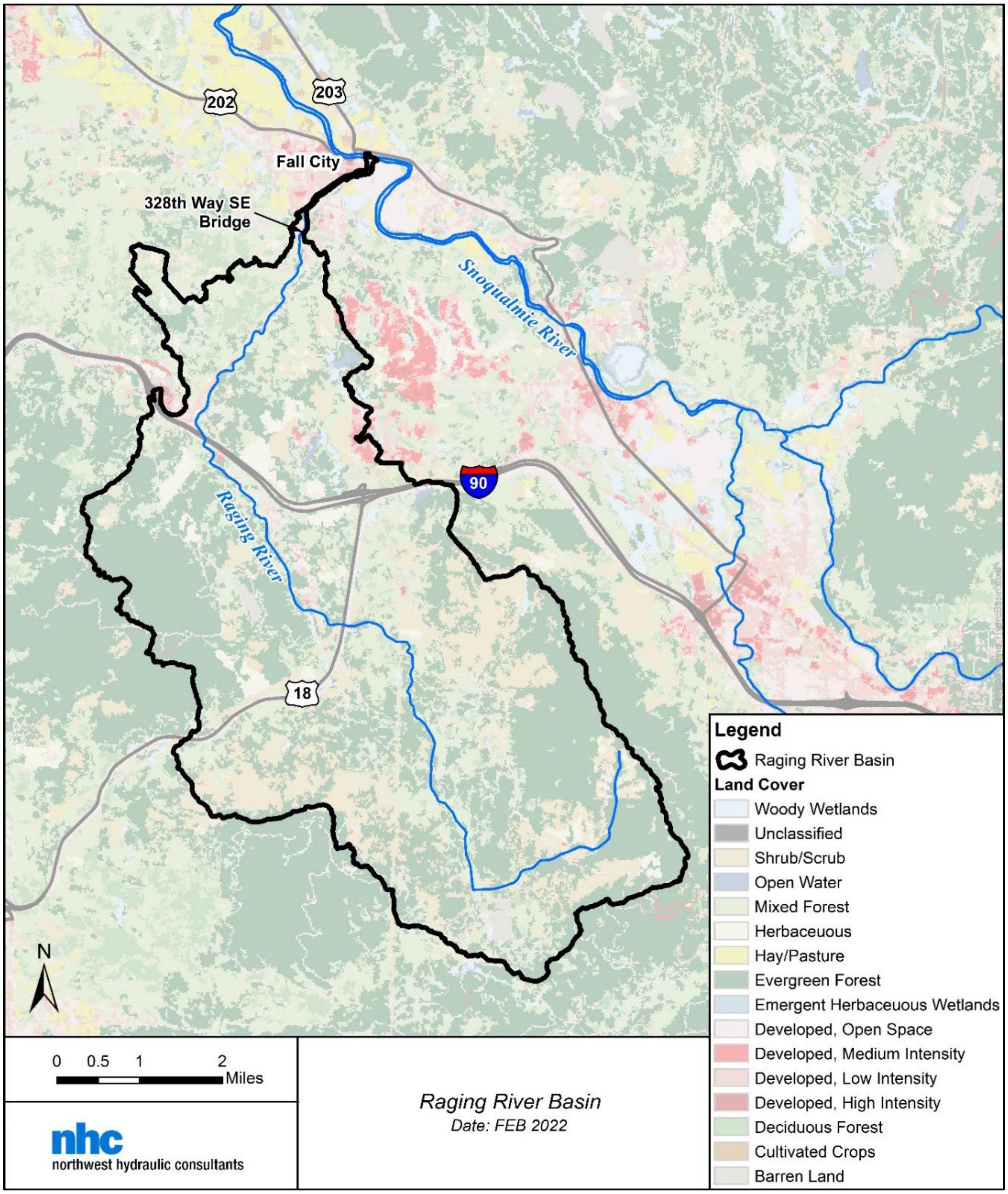


Figure 2.1 Raging River watershed and land cover.

2.2 Gage Data and Hydrologic Studies

The USGS operates two gages relevant to this study on the Raging and Snoqualmie rivers, as depicted in Figure 2.2. The Raging River near Fall City gage (12145500) is approximately 2.5 miles upstream of the confluence. The gage has a contributing basin area of approximately 30.6 square miles. The Snoqualmie River near Snoqualmie gage (12144500) is located approximately four miles upstream of the confluence, with a contributing area of 375 square miles. Both gages have continuous hourly flow records dating from the late 1980s to present, with longer records of annual peak data, as detailed in Table 2.1.

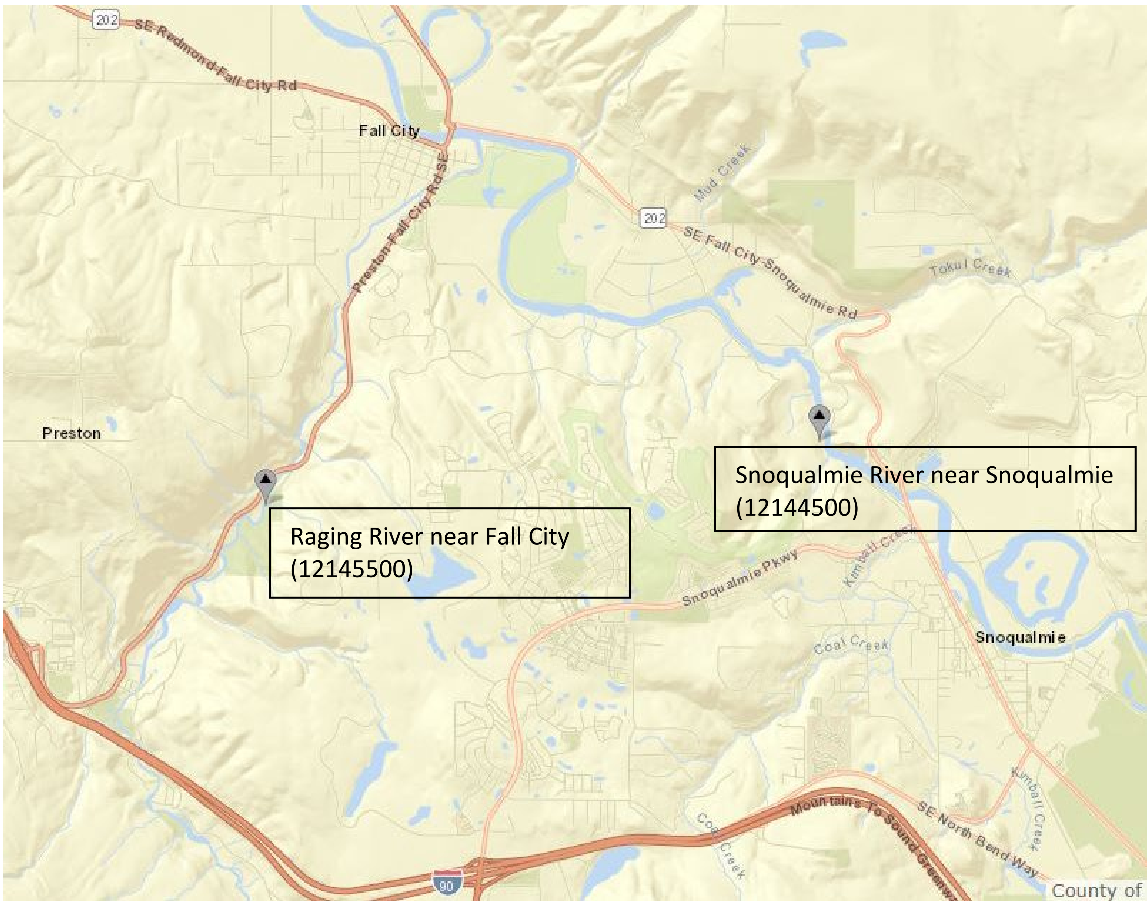


Figure 2.2 USGS gage locations

The Raging River flow regime follows a seasonal pattern, with winter high flow periods and summer low flow periods. As a generally lower elevation watershed with limited snow depths, there is no spring freshet evident in the historic monthly average flows. Annual peak flows have ranged from about 670 to 6,220 cfs (Figure 2.3). Mean annual flow is 133 cfs.

Table 2.1 Nearby gages and periods of record

Gage Number	Gage Name	Hourly (or finer) Data	Annual Peak Data
12145500	Raging River near Fall City	May 1988 - Present	1945-Present
12144500	Snoqualmie River near Snoqualmie	October 1987 - Present	1958-Present

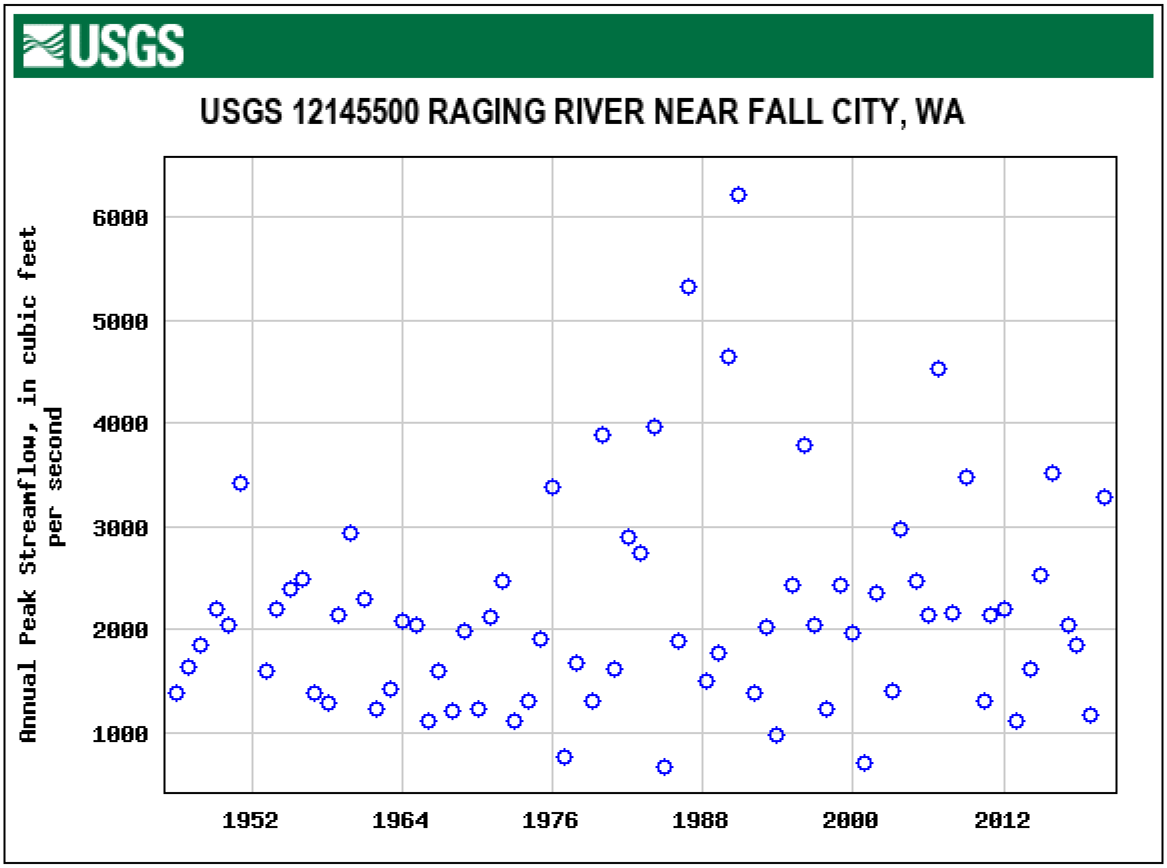


Figure 2.3 Annual peak flows for USGS 12145500 Raging River near Fall City gage

2.3 Major Historic Floods

Levees were constructed along the downstream-most 1.5 miles of the Raging River after a large flood in 1932 damaged houses and infrastructure in Fall City (King County, 2019). This flood event has been estimated to be approximately the same magnitude (6,300 cfs) as the November 1990 flood event (Harper Righellis, Inc., 1993).

Recent large flood events occurred in 1986 and 1990 and significantly impacted the Preston-Fall City Road bridge in addition to numerous properties in Fall City (FEMA, 2020c). The largest event on record occurred in November 1990 with a peak flow of 6,220 cfs (approximately a 200-year event). The second largest event occurred in November 1986, with a peak flow of 5,330 cfs (approximately a 75-year event). Post-1990, flows in excess of 3,000 cfs were recorded in water years 1996, 2007, and 2016 (with recurrence intervals ranging from 15 to 35 years). NHC obtained aerial flood photographs from King County that depict flooding during these recent events.

2.4 Existing Conditions Hydrology

2.4.1 Methods Used

A Bulletin 17C flow-frequency analysis (England Jr et al., 2019) was performed on the Raging River gage data using the USACE Statistical Software Package (HEC-SSP). The flow-frequency analysis was performed to estimate 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year recurrence interval flows. A summary of the results is provided in Table 2.2. Table 2.2 also includes the FEMA FIS flows at the USGS gage for comparison. The discharge values from the revised hydrologic analysis are lower than discharge values calculated in the FEMA FIS study (FEMA, 2020c). The decrease is due to the longer period of record and relative lack of large floods in recent years. As this revised analysis contains a longer period of record and uses updated Bulletin 17C methods, the 2021 calculated values were selected for use in this study.

Table 2.2 Flow frequency analysis for Raging River at Fall City

Recurrence Interval	USGS Gage 12145500 (NHC, 2021)[1] Peak Flow and 95%, 5 % confidence limits (cfs)	USGS Gage 12145500 (FEMA, 2020c)[2] Peak Flow (cfs)
2-year	2,020 (1810, 2240)	-
5-year	2,980 (2690, 3330)	-
10-year	3,650 (3270, 4220)	3,790
25-year	4,510 (3860, 5570)	-
50-year	5,160 (4210, 6780)	5,910
100-year	5,830 (4520, 8210)	6,970
250-year	6,730 (4860, 10500)	-
500-year	7,440 (5100, 12590)	9,840

Notes:
1. NHC 2021 statistical analysis period spanning 1945 through 2020 plus a historical event in 1932.
2. FEMA FIS statistical analysis used a period of record from 1945 to 1992 plus a historical event in 1932.

In addition to the peak flow-frequency analysis, a volume-frequency analysis was performed on the 3-hour, 24-hour, and 72-hour volumes using HEC-SSP. The volume-frequency analysis was performed using available hourly data, with records from 1988 to 2020. Peak volumes for each of the durations were calculated manually on an annual basis, then the Bulletin 17C methods (England Jr et al., 2019) in HEC-SSP were used to calculate the volume quantiles for select return periods. Results from the volume-frequency analysis are provided in Table 2.3.

Table 2.3 Volume-frequency analysis for Raging River at Fall City

Recurrence Interval	3-hour Volume (ac-ft)	24-hour Volume (ac-ft)	72-hour Volume (ac-ft)
2-year	495	3,053	6,221
5-year	724	4,401	8,425
10-year	880	5,312	9,850
25-year	1,083	6,479	11,614
50-year	1,236	7,357	12,905
100-year	1,391	8,241	14,180
250-year	1,600	9,429	15,856
500-year	1,763	10,347	17,126

2.4.2 Coincident Peak Analysis

NHC performed a joint-coincidence analysis with Snoqualmie River floods to determine effects the Snoqualmie River flows have on flood levels and levee performance in the LRR. To carry out the analysis, NHC used the USACE 1D HEC-RAS computer program (version 6.0). The model domain extends from the respective river gages, Snoqualmie River near Snoqualmie gage (12144500) and Raging River near Fall City gage (12145500), to approximately 1.4 miles downstream of the confluence. The Snoqualmie portion of this model was adopted from the existing FIS model (NHC, 2006), while the LRR portion was newly-created from recent topobathymetry (Quantum Spatial, 2020). The study area for this model is the confluence region and the lower portion of the Raging River. The downstream boundary of the model is a Snoqualmie River (FIS) cross-section rating curve approximately 1.4 miles below the confluence. The location was selected so that boundary condition assumptions will have a minimal effect on hydraulics in the confluence region. Roughness values for the Raging River were adopted from the FEMA FIS HEC-2 model (FEMA, 2020c). The selected main channel roughness varied from 0.035 to 0.040. Overbank roughness in dense riparian areas varied from 0.06 to 0.07, while overbank grass/shrub coverage varied from 0.04 to 0.05. A calibration effort was not included with the joint-coincidence model, as this model was created exclusively to route floodwaves a relatively short distance; this approach was agreed upon with the County. This 1D model is unique to the joint-coincidence analysis; a detailed two-dimensional (2D) model of the study area was developed for the main hydraulics tasks and is described in Section 3 of this report.

NHC used the joint-coincidence model to simulate the entire period in which hourly flow data was available (May 1988 to present) for the two gages. To account for flow contribution from Tokul Creek, the Snoqualmie River gage values were scaled by 1.03. This scaling factor was also applied in the Fall City Restoration analysis (Watershed Science and Engineering, 2020). Small gaps in the gage records were filled by linear interpolation. The Snoqualmie River gage went offline during the annual peak for water year 1991 (largest flood on record, occurring in November 1990). To account for this missing period of record, NHC approximated the timing of the peak based on available record data and applied the USGS' estimated peak flow of 74,300 cfs.

NHC reviewed results from the joint-coincidence model to approximate lag time from the USGS gage locations to the confluence. The lag time for the Snoqualmie River was equal to one hour while the Raging River lag time was less than one hour (the output mapping interval). A scatterplot was created to compare Snoqualmie and Raging rivers hourly flows (greater than the LRR 2-year event). The scatterplot accounted for lag time determined from the 1D HEC-RAS model. Figure 2.4 depicts the flow trends. The wide data band observed for Snoqualmie River flows when the LRR is generally less than 4,500 cfs (~30-year event) is due to differences in hydrograph timing and duration. The LRR peaks approximately six hours before the Snoqualmie River. Therefore, the falling limb of the LRR flood hydrographs typically overlaps with the Snoqualmie River peak, while the rising limb of the LRR flood hydrograph occurs prior to the Snoqualmie River's crest. During larger events (greater than the 30-year), there is apparently more coincidence with the Snoqualmie and LRR peaks, although this may be due to limited data from the few floods that have exceeded 4,500 cfs on the Raging River since 1988.

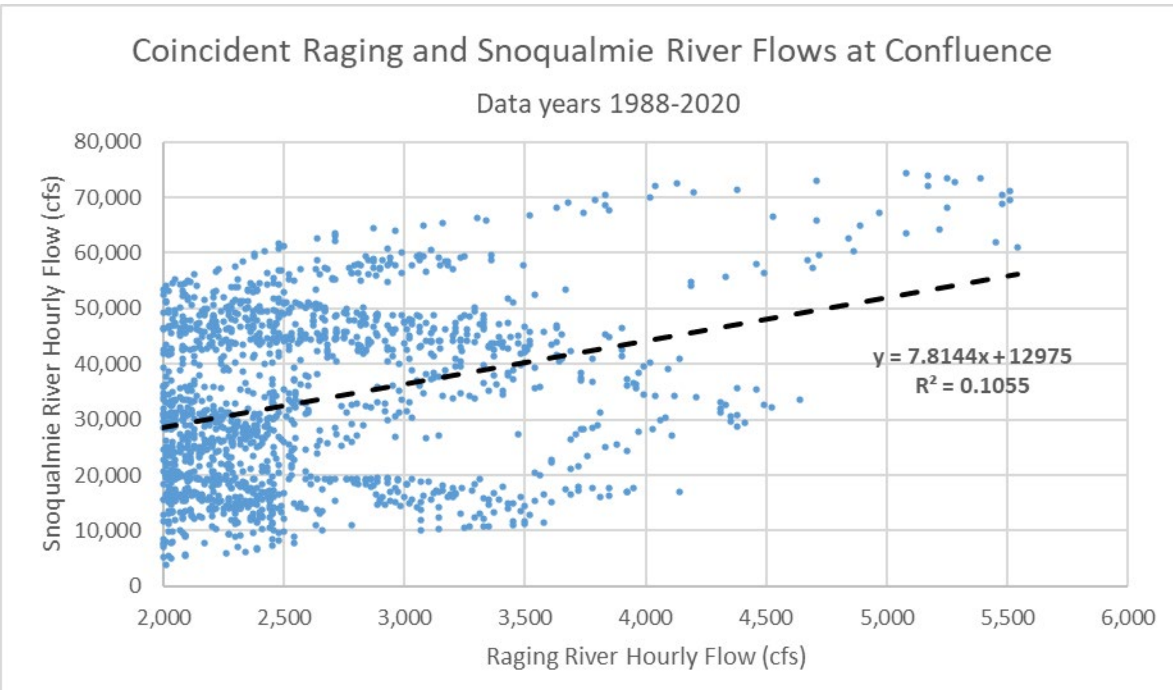


Figure 2.4 Raging River vs Snoqualmie River lagged hourly flow

Annual instantaneous peak stages were extracted from the model in the Snoqualmie River directly upstream of the confluence (at Snoqualmie River mainstem 7, section 34.36). This cross-section is clearly hydraulically influenced by the Snoqualmie River based on model results. A USGS Bulletin 17C stage-frequency analysis (England Jr et al., 2019) was performed on the annual peak stage data using HEC-SSP. The stage-frequency analysis was performed to estimate recurrence intervals for modeling discrete events and levee breaches as part of the hydraulics task. The period of analysis spanned from water year 1989 to 2020. Figure 2.5 depicts the stage trends at section 34.36 (directly upstream from the confluence with the Raging River) for Raging River flow rates at and above 2,000 cfs. A summary of calculated exceedance probability stages is provided in Table 2.4, and plotted in Figure 2.5.

Results from the coincident-peak analysis were used to develop Snoqualmie River inflows to pair with each Raging River flood simulation to be performed using the detailed 2D hydraulic model (Section 3). The Snoqualmie River flow corresponding to each stage in the stage-frequency results was calculated using the 2D model. Note that the purpose of the Snoqualmie River flow is to create the target stage at the Raging River confluence; this flow does not have a statistical probability assigned to it. The result is a stage in the confluence region that has a return period equivalent to the Raging River flood flow return interval for each simulation.

In a later phase of the Project, the joint-coincidence results will be examined at additional locations further up the Raging River to inform the stage-uncertainty function for the economic model.

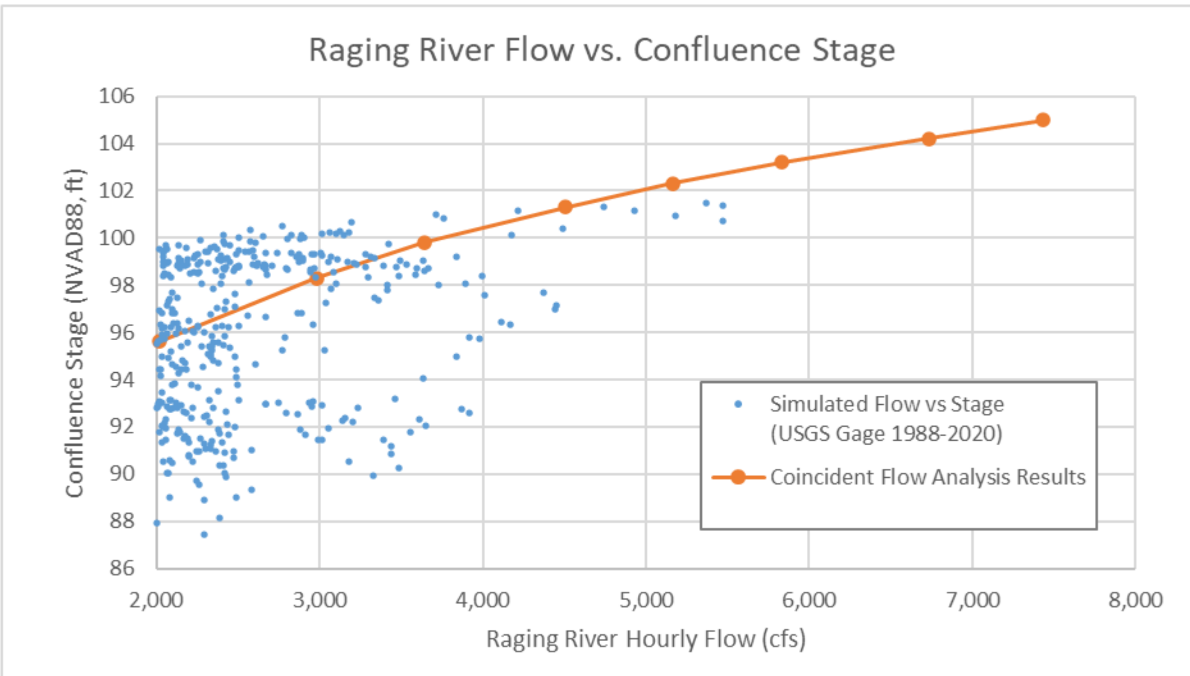


Figure 2.5 Raging River hourly flow and stage at confluence with Snoqualmie River

Table 2.4 Coincident flow analysis results (existing conditions)

Raging River		Snoqualmie River	
Raging River Flow Recurrence Interval	**Raging River Flow (cfs)**	**Coincident Peak Stage at Confluence (ft) [1]**	**Coincident Snoqualmie River Flow (cfs) [2]**
2-year	2,020	95.6	20,000
5-year	2,980	98.3	25,000
10-year	3,650	99.8	34,000
25-year	4,510	101.3	47,000
50-year	5,160	102.3	60,000

Raging River		Snoqualmie River	
Raging River Flow Recurrence Interval	**Raging River Flow (cfs)**	**Coincident Peak Stage at Confluence (ft) [1]**	**Coincident Snoqualmie River Flow (cfs) [2]**
100-year	5,830	103.2	75,000
250-year	6,730	104.2	94,000
500-year	7,440	105.0	110,000

Notes:

1. Snoqualmie River peak stage analyzed 280 feet upstream of the confluence with the Raging River.
2. Snoqualmie River inflow required to achieve peak stage target; no recurrence interval is attached to this flow.

2.4.3 Balanced Hydrographs

NHC developed balanced hydrographs by scaling a historical flood hydrograph to match the frequency results for both peak and volume. The analysis was performed for eight return periods (2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year) and three durations (1-, 24-, and 72-hours). Because the period of hourly flow record was significantly shorter than the annual peaks record, the frequency analysis of the volumes used a shorter period of record than the analysis of the peaks. In selecting the historical flood for scaling, consideration was given to selecting an event with a well-shaped hydrograph amenable to scaling. A review of notable floods indicated that the November 2008 event was suitable for this purpose, as its 3-, 24-, and 72-hour volume ratios were similar to the frequency analysis ratios.

NHC used the following procedure to develop the balanced hydrographs:

1. The entire November 2008 historic hydrograph was initially scaled to the peak flow determined from the frequency analysis (Table 2.2). A uniform scaling factor was applied to each discharge interval (15-minute) along the historic hydrograph.
2. The 3-hour volume was then scaled by applying a uniform scaling factor to each incremental volume (15-minute) inside of the 3-hour time period and outside of the 15-minute peak. The 3-hour period was centered on the peak. Following scaling, the total volume within the 3-hour interval (including the peak) was equal to the 3-hour volume derived from the frequency analysis reported in Table 2.3.
3. The 24-hour volume was scaled in a similar manner to the 3-hour approach. A uniform scaling factor was applied to values within the 24-hour period but outside of the previously scaled 3-hour period. These values were scaled such that the total volume within the 24-hour interval matched the 24-hour frequency volume reported in Table 2.3.
4. Lastly, the 72-hour volume was scaled to match the frequency analysis volume reported in Table 2.3. A uniform scaling factor was applied to incremental values within the 72-hour period and outside of the 24-hour period. In some cases, the scaling factor for the 72-hour period was larger than that for the 24-hour period, creating an abrupt transition in the balanced hydrograph. To smooth these irregularities and produce a more natural hydrograph, the discharge value at the boundary of the 24-hour period was held at a constant value (horizontal slope) until the applied scaling factor created a smooth transition while maintaining the target frequency volume. This is evident in the flat portions of the hydrographs shown in Figure 2.6.

The total volume within the 72-hour was maintained to meet frequency values reported in Table 2.3.

Figure 2.6 presents the final hydrographs used for the upstream boundary condition of LRR.

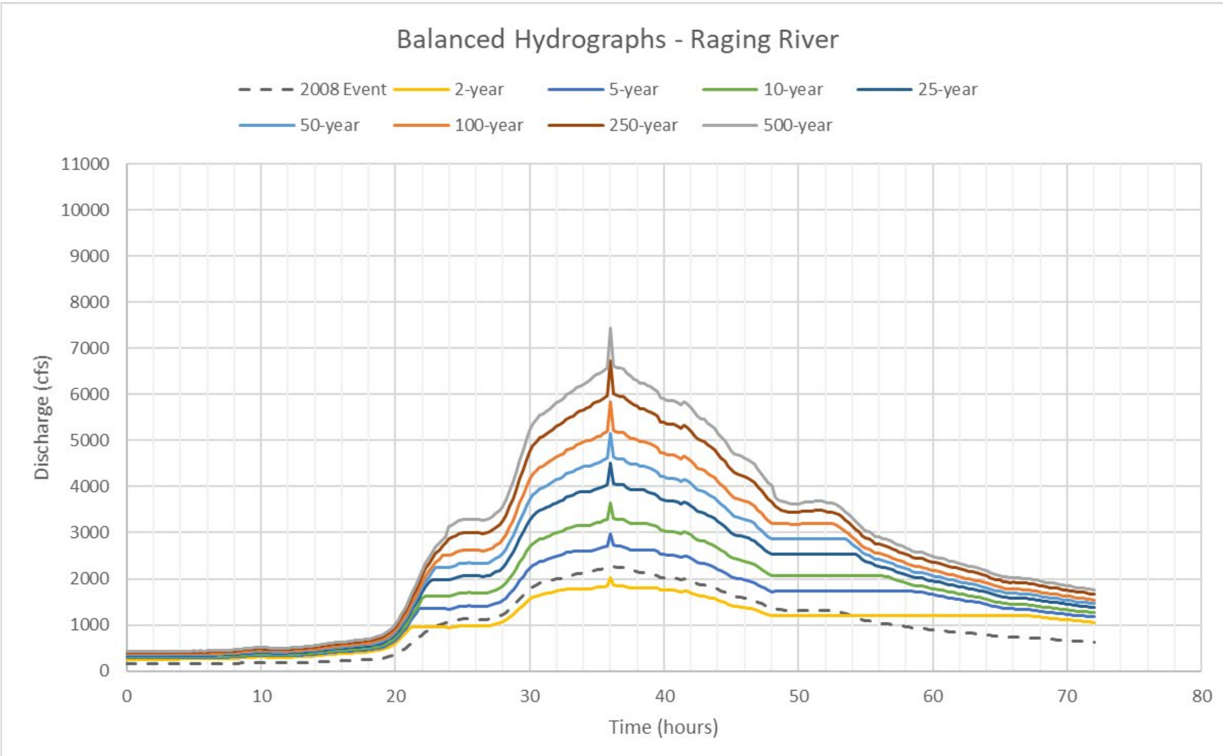


Figure 2.6 Balanced hydrographs for Raging River

2.5 Future Conditions Hydrology

Future conditions hydrographs were developed by scaling the existing conditions balanced hydrographs (Section 2.4.3) using multiplication factors derived from hydrologic projections for the Raging River by the Climate Impacts Group (CIG) at the University of Washington (CIG, 2020). Appendix A contains the future climate projections technical memorandum, which documents the analysis in detail. No consideration was given in the future conditions analysis for landuse changes that could impact flows such as development related to forest conversion or impervious surface increases.

Per Appendix A, NHC calculated percent change in peak flow for each general circulation model (GCM) and each return period up to the 500-year event for the 1-hour, 24-hour, and 72-hour durations. The calculated percent change was a ratio of CIG's 2050s flow quantile divided by the matching 1990s flow quantile. The median future conditions scaling factors, used to develop future conditions balanced hydrographs, are provided in Table 2.5. These factors are from Table 1.1 in Appendix A.

NHC created the future conditions hydrographs by applying the median scaling factors to the balanced hydrographs (documented in Section 2.4.3). The percent change in flow was computed for five return periods (2-, 10-, 50-, 100-, and 500-year) and three durations (1-, 24-, and 72-hour). The following procedure was used to scale the hydrograph data:

1. The 1-hour duration of each balanced hydrograph (Figure 2.6) was initially scaled by applying the median future conditions scaling factor (Table 2.5) to each incremental discharge value (15-minute) inside of the 1-hour time period. The 1-hour period was centered on the peak.
2. The 24-hour duration was then scaled by applying a uniform scaling factor to values within the 24-hour period but outside of the previously scaled 1-hour period. These values were scaled using a lower 24-hour scaling factor than reported in Table 2.5 to ensure the average scale within the entire 24-hour interval, including the 1-hour period, matched the 24-hour scaling factor in Table 2.5.
3. Lastly, the 72-hour duration was scaled by applying a uniform scaling factor to values within the 72-hour period but outside of the previously scaled 24-hour period. These values were scaled such that the average scale within the entire 72-hour interval matched the 72-hour scaling factor in Table 2.5.

The computed future projections peak flows and scaled hydrographs are reported in Table 2.6 and presented in Figure 2.7, respectively.

Table 2.5 Future conditions scaling factors projected for the 2050s (median values)

Recurrence Interval	1-hour	24-hour	72-hour
2-year	11%	5%	17%
10-year	13%	10%	13%
50-year	22%	14%	5%
100-year	27%	21%	9%
500-year	41%	35%	21%

Table 2.6 Lower Raging River future conditions peak flows

Recurrence Interval	Peak Flow (cfs)
2-year	2,240
10-year	4,120
50-year	6,300
100-year	7,400
500-year	10,480

To develop future conditions coincident stages for the Snoqualmie River, the same procedure that was used for existing conditions coincident stages was repeated but with the future conditions Raging River flows as shown (Table 2.6). The future conditions flows were applied to the relationship between Raging River discharge and confluence stage as presented in Table 2.4 to determine a future conditions coincident stage. For example, the future conditions 10-year Raging River discharge is 4,119 cfs (Table 2.6). For this peak flow, the coincident peak analysis described and summarized in Table 2.4 predicts a stage at the confluence of 100.6 feet. For the 500-year future conditions event, extrapolation of the coincidence results was required. Table 2.7 presents the future conditions stage results at the confluence for each future condition Raging River flood, as well as the corresponding Snoqualmie River flow required to achieve that stage.

Table 2.7 Coincident flow analysis results (future conditions)

Raging River		Snoqualmie River	
Future Raging River Flow Recurrence Interval	**Raging River Flow FC (cfs)**	**Confluence Peak Stage (ft) [1,2]**	**Snoqualmie River Flow (cfs) [3]**
2-year	2,240	96.2	21,500
10-year	4,120	100.6	40,000
50-year	6,300	103.8	86,000
100-year	7,400	105.0	110,000
500-year	10,480	107.5	168,000

Notes:
1. Snoqualmie River peak stage analyzed 280 feet upstream of the confluence with the Raging River.
2. Interpolated/extrapolated from Table 2.4.
3. Snoqualmie River inflow to required achieve peak stage target, no recurrence interval is attached to this flow.

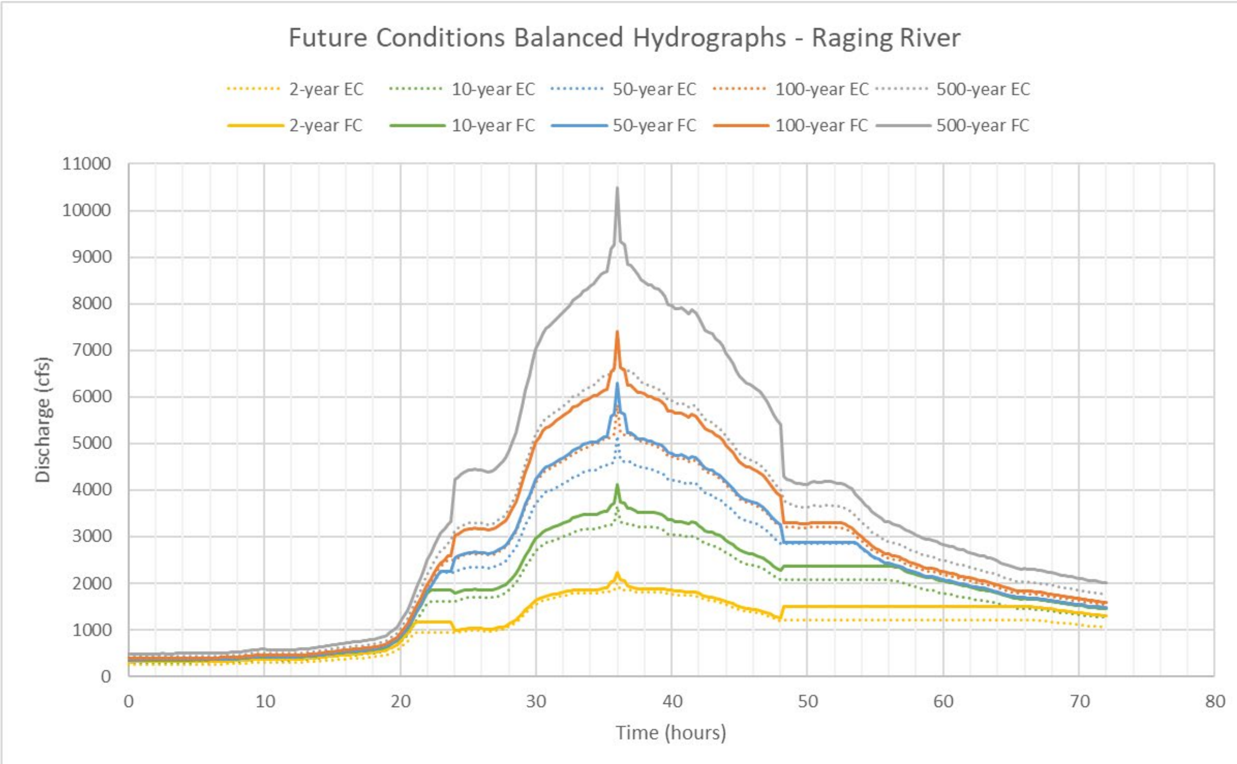


Figure 2.7 Lower Raging River future conditions hydrographs

3 HYDRAULICS

3.1 Summary of Geomorphology

A full geomorphic assessment of the LRR is beyond the scope of the current study; however, NHC reviewed information from the Raging River Channel Migration Study (King County, 2019) and documented general field observations. The geomorphology of the Raging River results from the most recent (Vashon) glacial maximum (King County, 2019). During glacial retreat, the current alignment of the LRR served as an outlet for glacial Lake Snoqualmie, flowing from north to south, before turning west and following the present-day East Fork Issaquah Creek alignment into the Lake Sammamish basin. As glacial retreat continued, this flowpath was abandoned and a basin divide developed between present day Preston and Fall City. Eventually, head-cutting in the Fall City side of the divide progressed to the point that the Raging River was captured and diverted from the Sammamish basin to the Snoqualmie basin. The increased flow caused the LRR to incise and create the relatively steep and confined segment between Preston and Fall City.

The modern Raging River is characterized as a single thread channel with a pattern of small-amplitude meanders. In the Project area, the river exhibits plane bed morphology with limited large woody material observed. Riffles were also noted in a number of reach segments. The upper river profile, RM 8.5 to RM 14.6, is characterized by relatively steep gradients that result in high velocities, causing significant bank erosion upstream (FEMA, 2020c). Where the lower reach flows into the confluence with the Snoqualmie River, an alluvial fan deposit was formed that extends from RM 1.5 to RM 0.0 and results in large gravel bars near the confluence. In vicinity of the confluence, NHC compared historic cross-sections from the FEMA effective study (surveyed in 1993) with recent topobathymetric data and observed the bed elevations to be slightly higher in current conditions, reflecting aggradation.

3.2 Flood Control Structures and Other Key Structures

Following a major flood event in 1932, the LRR levee and revetment system was constructed from 1938-1940 to protect infrastructure and allow for urban development in Fall City. The flood protection system consists of four King County levee/revetment facilities located along the downstream-most 1.5 miles of the Raging River. The facilities are present on the left and right banks of the river and account for a total of 2.85 miles of facilities. The system protects residential, commercial, and agricultural properties and infrastructure from flood hazards within the unincorporated town of Fall City. The Raging River levee system does not currently meet FEMA freeboard requirements, per the FIS (FEMA, 2020c).

Two bridges cross the Raging River within the study area: (1) the Preston-Fall City Road SE bridge near Fall City; and (2) the 328th Way SE bridge at the upstream end of the levee study reach. The Preston-Fall City Road bridge is located at RM 0.4 and contains three piers. The 328th Way SE bridge is located at RM 1.7 and is a clear span.

Stormwater infrastructure in Fall City is limited; in the study area, two storm outfalls are located along the levee system. A stormwater detention pond with a flapgate outfall to the Raging River was observed along the left bank at RM 0.9. A culvert outfall was observed at RM 0.6 along the right bank.

3.3 Model Selection

Detailed hydraulic modeling of the Lower Raging River was conducted using the USACE 2D HEC-RAS computer program (version 6.1). Two-dimensional HEC-RAS is capable of simulating subcritical and supercritical unsteady flows through a full network of open channels and floodplains, including complex terrains with highly non-uniform flows. HEC-RAS is a hydraulic model approved by FEMA for riverine studies and flood analyses. Development of the hydraulic model components and results from computer simulations are described in the following sections.

3.4 Terrain Development

The model terrain was developed using topobathymetric Light Detection and Ranging (LiDAR) collected by Quantum Spatial in 2020 (Quantum Spatial, 2020). Consideration was given to using the surface from the Fall City Restoration Project (Watershed Science and Engineering, 2020), but the bathymetry in that project surface was superseded by 2020 data collected by Quantum Spatial. NHC received quarter sections of topobathymetric data from the County and used ArcGIS to merge the sections together to cover the entire 2D model domain.

3.5 Computational Mesh

3.5.1 Model Domain

The 2D model domain encompasses the Raging River from approximately 0.5 miles upstream of the 328th Way SE bridge (RM 2) to the confluence with the Snoqualmie River. NHC included the Snoqualmie River in the 2D model to account for hydraulic influence at the confluence. The 2D domain encompasses the Snoqualmie River valley from RM 31.4 to RM 37.1. The downstream boundary for the 2D model was based on two FEMA FIS section locations; one section for the Snoqualmie main channel and a second for the left overflow channel (see Section 3.7.2 and Figure 3.1). The location of the two FEMA FIS sections were selected sufficiently far from the Project area so as to avoid boundary condition effects. In addition, the FIS sections were selected due to the relatively smooth nature of their rating curves; it was noted that both curves exhibit a small hysteresis effect with a spread of less than 0.5 feet. NHC performed sensitivity testing (Section 3.10.2.3) to ensure the boundary condition locations were sufficiently downstream of the Project area and performed levee breach testing to ensure that all building structures that may be impacted by a levee breach are included in the domain. Figure 3.1 shows the complete 2D model extents.

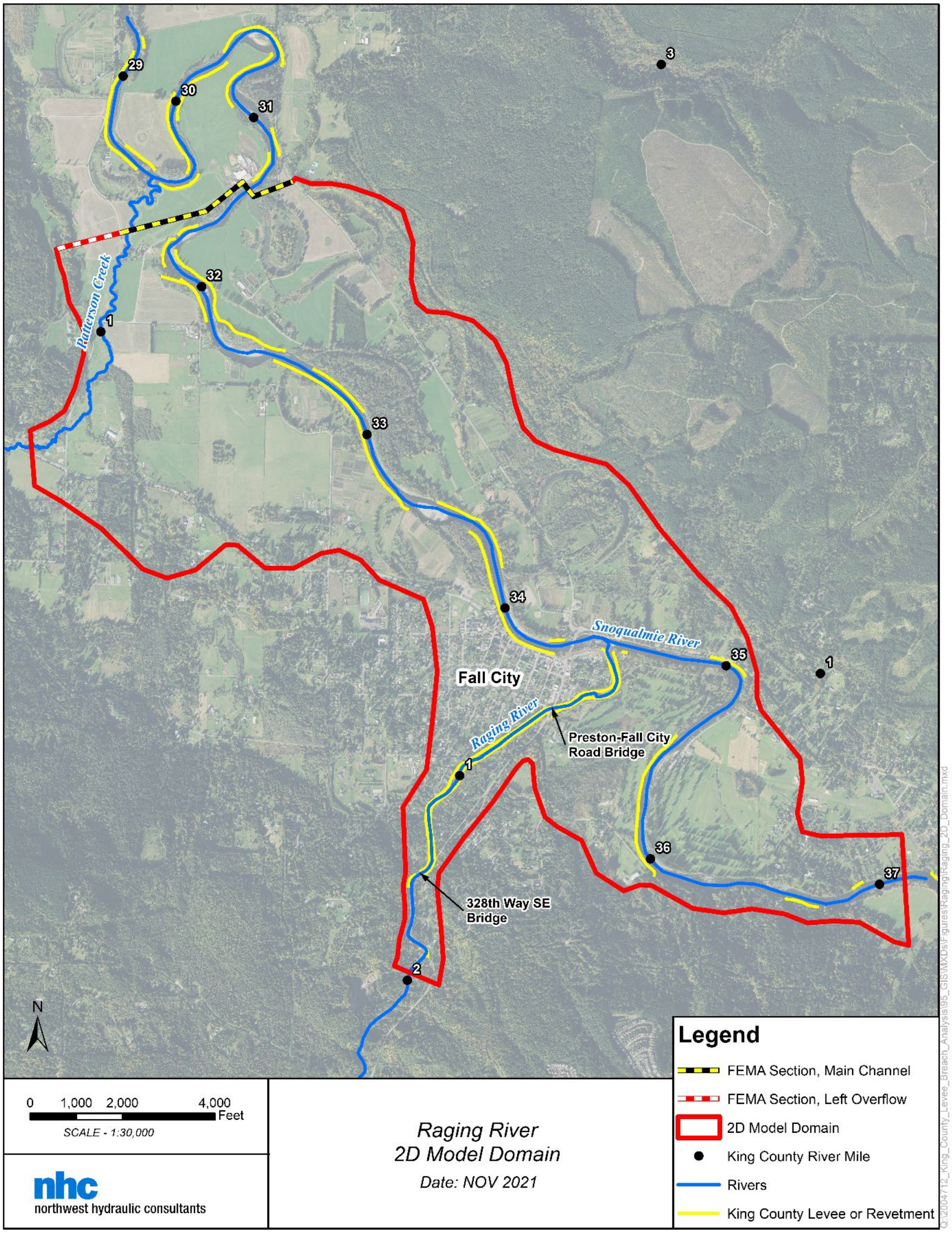


Figure 3.1 2D HEC-RAS model domain

3.5.2 Cell Size Selection

The 2D model domain consists of 15-foot computational mesh spacing within the LRR and Fall City areas. The mesh within the Snoqualmie River valley consists of 60-foot spacing along the river alignment and 100-foot spacing in overbank areas. A cell protection radius was applied between the Raging River and Snoqualmie River zones to gradually transition spacing between the two regions. The final mesh spacings were established from preliminary test simulations and provide sufficient resolution to capture landcover regions including buildings while maintaining manageable model run times.

3.5.3 Breakline Use

Breaklines were added along prominent topographic features to provide refinement and definition to terrain elements, such as high points and roadways. NHC also applied breaklines to orient mainstem channel cell faces perpendicular to the direction of flow. A majority of breaklines within the Snoqualmie valley area were obtained from the Fall City Restoration Project model (Watershed Science and Engineering, 2020). Breaklines within the Raging River corridor and Fall City region were newly created.

3.5.4 Hydraulic Structures

Bridge, levee, and stormwater culvert structures were added to the 2D HEC-RAS model. Elevations for bridges, levees, and culverts were applied from the ground survey data, completed for this project by 1 Alliance Geomatics in October 2021. The following sections document these hydraulic structures in detail.

3.5.4.1 Levees

NHC coded levees and embankments into the model as hydraulic structures (weirs). The weirs were divided into segments, ranging from approximately 500 to 1,800 feet, to allow more discretization in parameters and calculation methods. Segments were selected with consideration of structure height above the natural ground and expected tailwater conditions at the peak of the flood which dictated use of the 2D or weir flow equations at each structure. The “2D flow equation” option was used for weirs that experience high tailwater conditions. Levees that maintained a significant head difference from the river floodplain at flood peak (approximately three feet or more) used the weir equations with a weir coefficient of 2.6. The selected calculation method for each levee segment was fixed for all flows (i.e., the calculation method for each weir did not switch between the weir equation and 2D equations depending on the flow).

3.5.4.2 Bridges

Two bridges are located along the Raging River within the model domain: (1) the Preston-Fall City Road SE bridge near the confluence; and (2) the 328th Way SE bridge at the upstream end of the leveed reach of the river. As part of this project, 1 Alliance Geomatics performed a field survey of the two bridge structures in October 2021 to collect low chord elevations and embankment/pier locations. Plans for each of the bridges were also available to fill gaps in the survey.

NHC modeled the Preston-Fall City Road SE bridge as a hydraulic structure within the model due to one of the bridge piers interacting with the river. The bridge was coded as a hydraulic structure – with bridge deck and pier dimensions defined – rather than a terrain modification due to the terrain resolution being significantly larger than the pier width; use of a terrain modification would not appropriately capture hydraulics at the pier. Model results showed the 500-year future conditions flow resulted in a water surface elevation (WSEL) 0.2 feet lower than the low bridge chord. During this event, flow typically overtops the right- and left-bank levees upstream of the bridge. Results from the second- and third-largest events (100-year with future conditions and 500-year without future conditions) show 1.5 feet of freeboard from the bridge low chord. As a result, the Preston-Fall City Road SE bridge is not expected to experience pressure flow.

The 328th Way SE bridge is a clear span structure with no in-channel piers interacting with the flows. Model runs showed no pressure flow during the 500-year future conditions event with approximately 1.9 feet of freeboard. As the bridge deck is well above the water elevation with no interaction with flows, the 328th Way SE bridge was not included in the 2D model geometry.

In addition to the bridge crossings along the Raging River, NHC also included the State Route 202 bridge along the Snoqualmie River, immediately downstream of the confluence. Bridge geometry data was obtained from the effective model (NHC, 2006). The bridge deck was determined to be well above the 500-year WSEL (not pressurized), while terrain resolution was not fine enough to capture the piers. As a result, the bridge was coded into the 2D model as a hydraulic structure and modeled using the energy equation.

3.5.4.3 Culverts

One culvert was included in the model, a 30-inch corrugated metal pipe along the right bank of the Raging River near RM 0.6. The culvert does not have a flapgate. The culvert was coded into the model as a hydraulic structure. Structure input data, including inlet configuration, length, and invert elevations, was surveyed by in October 2021 by 1 Alliance Geomatics as part of this project. While an interior drainage analysis is not part of this scope, tailwater conditions may be estimated at the breach locations, if necessary, during the final analysis.

3.6 Landcover/Roughness

3.6.1 Data Sources

A landcover layer for the model domain was created using the following data sources:

1. Fall City Restoration Project landcover layer (Watershed Science and Engineering, 2020)
2. A building footprint layer (Microsoft Maps, 2020)
3. 2019 NAIP aerial photography (National Agricultural Imagery Program, 2019)
4. Estimated vegetation height derived from the difference between bare earth and first return LiDAR data (DSM minus DTM)

NHC used the Fall City Restoration Project's landcover layer (Watershed Science and Engineering, 2020) in areas occupied by that model's domain. The coverage contained the entire Snoqualmie River valley and a portion of the Raging River, near the confluence. The Fall City Restoration Project used a general "urban" classification within the Fall City limits. NHC removed the "urban" land coverage area to provide more detailed roughness coverage within the developed areas of Fall City.

The selected 2D model grid resolution of 15-feet is fine enough to capture individual buildings, so a building footprint layer was added to the landcover classification. To populate this information, NHC obtained building outlines from the Microsoft AI buildings layer (Microsoft Maps, 2020). NHC compared the aerial photography to the Microsoft buildings layer, and manually added additional buildings greater than 2,000 square feet that were not included in the Microsoft data (approximately 40 structures).

A vegetation height map was developed by subtracting the bare earth LiDAR data from the highest hit LiDAR data, provided by the County (Quantum Spatial, 2020). The estimated vegetation height was used to create different classes of vegetation in the landcover layer, using the same method of classification that was used in the Fall City Restoration Project model.

3.6.2 Selection of Final Values

A total of 14 distinct landcover types were created. Roughness values for each landcover type were determined by literature review and comparison to the calibrated Fall City Restoration Project model (Watershed Science and Engineering, 2020). Vegetation heights were grouped into discrete categories, based on the Fall City Restoration Project. Buildings were assigned an extremely high Manning roughness value of 10. The high roughness value allows for water to enter the buildings, which will be important for damage computations in later phases of the Project, but not actively flow through them.

LRR main channel values were split into upper and lower reaches during the calibration process described in Section 3.9. The final calibrated values are reported in Table 3.1 and depicted in Figure 3.2 and Figure 3.3. With exception to the “urban” designations (Section 3.6.1), all floodplain roughness values used in the calibrated LRR model were identical to those used in the calibrated Fall City Restoration Project hydraulic model. Similarly, the Snoqualmie main channel roughness value was the same as that used in the calibrated Fall City Restoration Project hydraulic model. The Raging River channel roughness values are a product of the specific model calibration for this project, and therefore differ from those used in the calibrated Fall City Restoration Project hydraulic model.

Table 3.1 Final roughness coefficients used in 2D HEC-RAS model

Type of Surface Area	Manning’s Roughness Coefficients
Open Water/Side Channels	0.03
Snoqualmie Main Channel	0.025
Raging River Channel – Lower (RM 0.0 to RM 1.1)	0.044
Raging River Channel – Upper (RM 1.1 to RM 2.0)	0.054
Building	10
Road	0.015
Mow/Till <1' Tall	0.045
Tall Grass/Small Shrubs 1-4' Tall	0.05
Shrubs 4-6' Tall	0.075
Young Woodland	0.09
Young Mixed Forest	0.095
Forest	0.1
Vegetated Gravel Bar	0.06
Unvegetated Gravel Bar	0.037

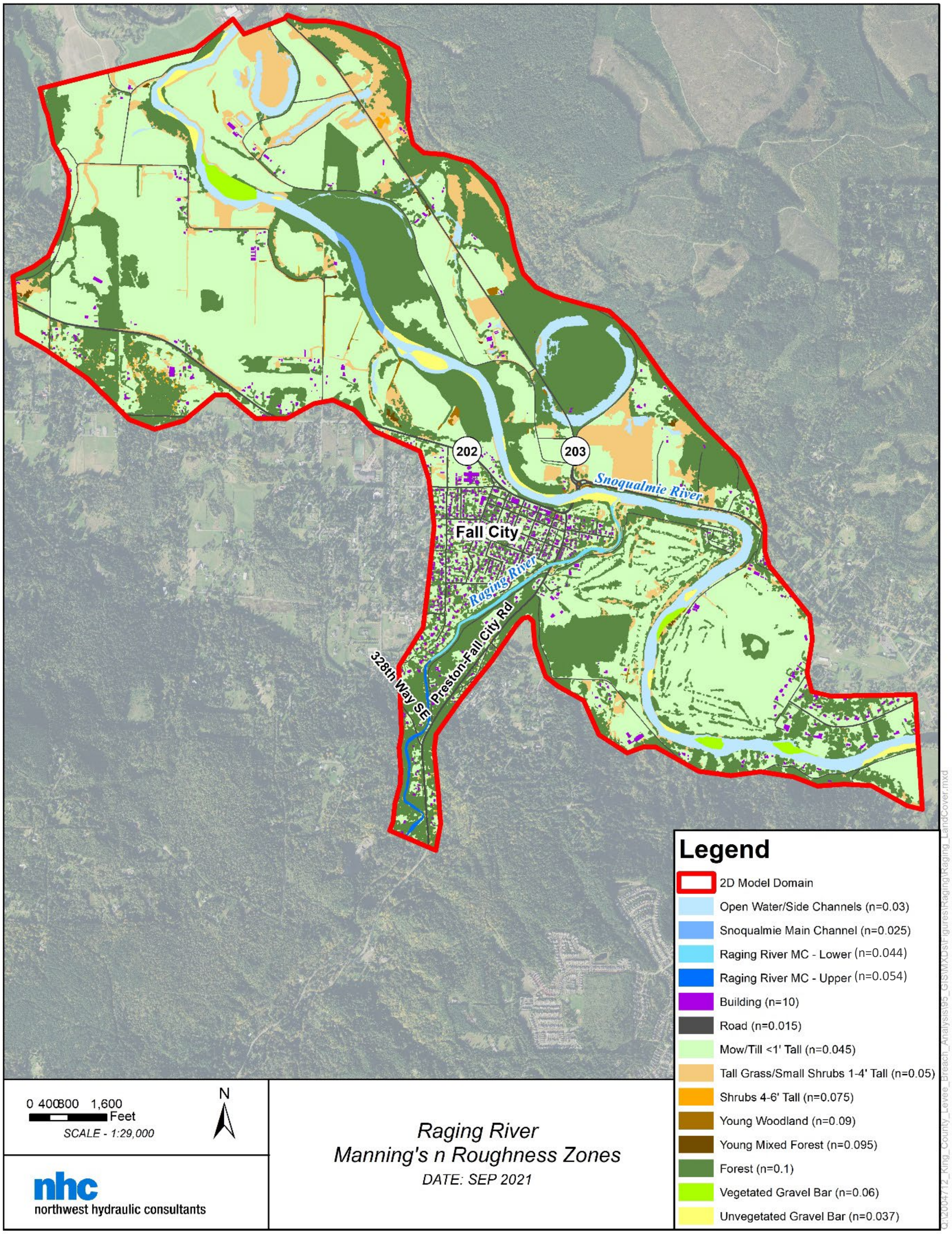


Figure 3.2 Overview of final Manning's n roughness zones

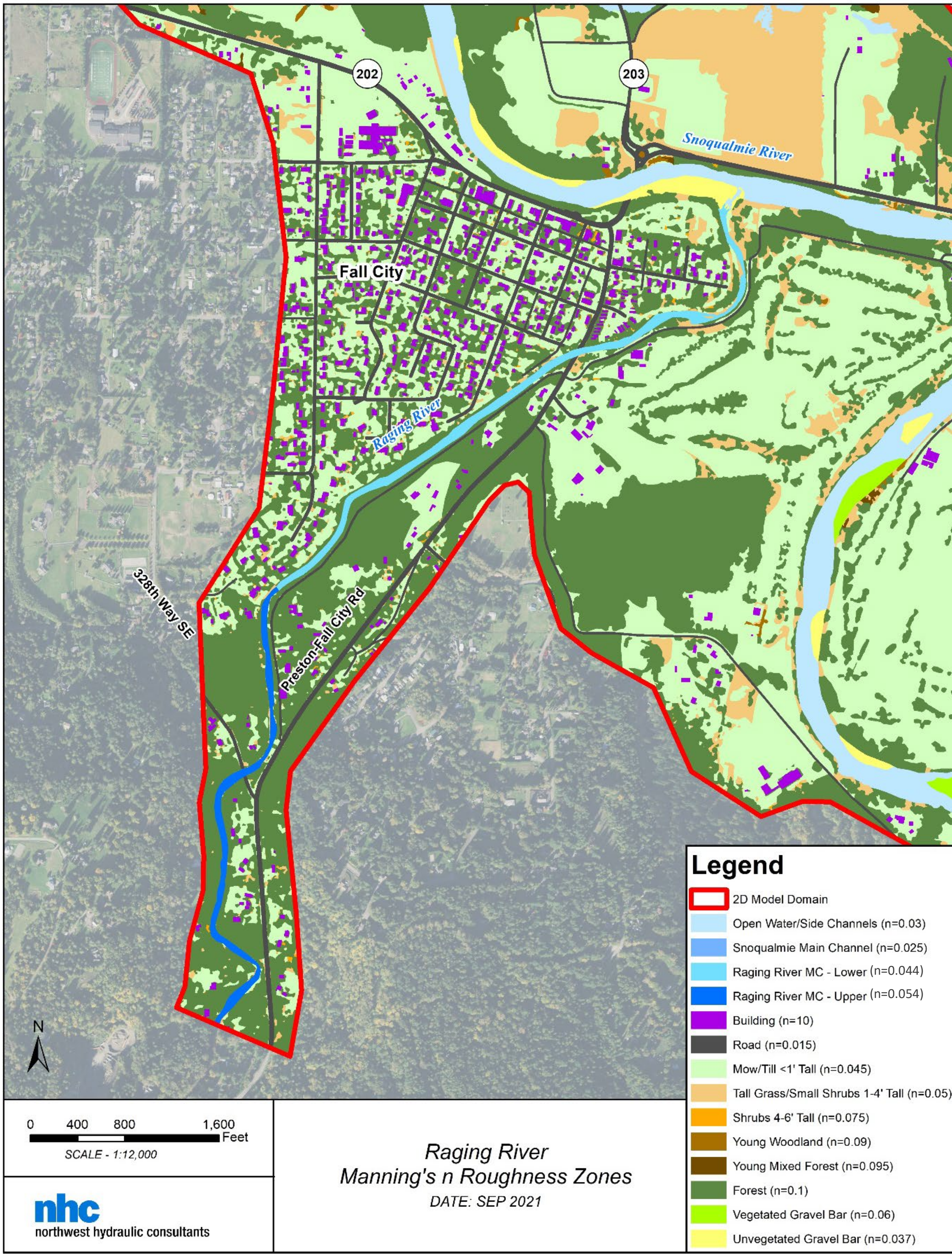


Figure 3.3 Raging River and Fall City final Manning's n roughness zones

3.7 Boundary Conditions

3.7.1 Inflows (Existing Hydrology)

Three inflow locations were defined for the 2D HEC-RAS model to represent contributions from the Raging River, Snoqualmie River, and Patterson Creek:

1. The Raging River boundary condition was assigned from the balanced hydrograph results, detailed in Section 2.4.3. While the balanced hydrograph spans a 72-hour timeframe, the 2D model simulates a 12-hour period, centered on the balanced hydrograph peak.
2. The Snoqualmie River upstream boundary condition consists of a steady-state flow sufficient to produce a stage near the confluence with a return interval that equals the Raging River inflow boundary condition's return interval. To determine the steady-state flow, flow was ramped up in the Snoqualmie River until the stage at the confluence matched the corresponding probability stage.
3. The Patterson Creek inflow was applied as a steady-state condition based on the FEMA effective peak flow (FEMA, 2020c).

Peak inflows for the three inflow boundaries are summarized in Table 3.2.

Table 3.2 Peak inflows for 2D HEC-RAS model (existing hydrology)

Event	Snoqualmie River Peak Flow (cfs) [1]	Raging River Peak Flow (cfs)	Patterson Creek Peak Flow (cfs) [2]
2-year	20,000	2,020	380
5-year	25,000	2,980	490
10-year	34,000	3,650	560
25-year	47,000	4,510	660
50-year	60,000	5,160	740
100-year	75,000	5,830	820
250-year	94,000	6,730	920
500-year	110,000	7,440	990

Notes:
1. Snoqualmie River flows do not correspond to return interval (see results of coincident peak analysis in Section 2.4.2).
2. Patterson Creek flows from FIS (FEMA, 2020c) for 10-, 50-, 100-, and 500-year floods. 2-, 5-, 25-, and 250-year flow data determined using logarithmic interpolation of FIS flows.

3.7.2 Inflows (Future Conditions Hydrology)

Inflows to the model were modified to account for future conditions. Similar to the existing hydrology inflows, the future conditions inflows were defined at the three locations to represent contributions from Raging River, Snoqualmie River, and Patterson Creek:

1. The Raging River boundary condition was assigned a future conditions balanced hydrograph, the development of which was detailed in Section 2.5. While the balanced hydrograph spans a 72-hour time period, the 2D model simulates a 12-hour period centered on the balanced hydrograph peak.
2. The Snoqualmie River inflow consists of a steady-state flow sufficient to produce a stage near the confluence with a return interval that equals the Raging River inflow boundary condition's return interval for future conditions (see Table 2.7 in Section 2.5). To determine the steady-state flow necessary to produce the desired stage, flow was ramped up in the Snoqualmie River until the stage at the confluence matched the corresponding probability stage from the future conditions analysis.
3. The Patterson Creek inflow was evaluated using the projected mean percent change from the Washington Department of Fish and Wildlife (WDFW) Future Projections for Climate-Adapted Culvert Design program (WDFW, 2018). For all events, the 2080 projected change in 100-year flood percentage, equal to 29.8%, was applied. Appendix B contains the information utilized from the WDFW future projections program.

Peak future conditions inflows for the three inflow boundaries are summarized in Table 3.3.

Table 3.3 Peak inflows for 2D HEC-RAS model (future conditions)

Event	Snoqualmie River Peak Flow (cfs) [1]	Raging River Peak Flow (cfs)	Patterson Creek Peak Flow (cfs) [2]
2-year	21,500	2,240	490
10-year	40,000	4,120	730
50-year	86,000	6,300	960
100-year	110,000	7,400	1,060
500-year	168,000	10,480	1,290

Notes:
1. Snoqualmie River flows do not correspond to return interval (see results of future conditions coincident peak analysis in Section 2.5).
2. Patterson Creek flows determined using the WDFW Future Projections for Climate-Adapted Culvert Design program (WDFW, 2018) (refer to Appendix B).

3.7.3 Downstream Boundary

The downstream boundary of the 2D model is aligned with two cross-sections from the FEMA effective model; therefore, two stage-discharge rating curves were applied. The main channel rating curve was extracted from the Snoqualmie River FIS model mainstem reach 7, station 31.15. The left overbank rating curve was extracted from the Snoqualmie River FIS model overflow reach 5, station 2.85. The location of these two sections is depicted in Figure 3.1. For the future conditions analysis, no changes were necessary to the downstream boundary conditions, as the assigned stage-discharge rating curves will automatically produce a higher stage to account for the increased flow.

3.8 Computation Parameters

Model computation parameters were selected from a series of preliminary test runs to accurately simulate hydraulics in the study reach while producing reasonable run times. The model was run using the 'classic' full momentum equation set (Shallow Water Equations with a Eulerian-Lagrangian approach to solving for advection, or SWE-ELM). The simulation time step was set to one second to achieve Courant values of less than 2.0. The main model parameters are summarized in Table 3.4.

A 4-hour ramp up period was applied to the model to establish equilibrium conditions at the beginning of the simulation. Forty percent of the 4-hour period involved ramping up flows from zero to the to the flow rate on the hydrograph corresponding to the start of the model simulation; the remaining time was held at a steady flow, equal to this flow rate. Use of the ramp up period ensured accurate flows at the beginning of the model run and prevented routing errors. Following the ramp up, the model was run with a 12-hour simulation period, centered on the peak of the balanced hydrograph (i.e., hour 30 to hour 42 in Figures 2.6 and 2.7). NHC determined the 12-hour period was sufficient to capture peak water surface elevations throughout the model domain. Simulation of the full 72-hour balanced hydrograph was therefore not necessary and computational run-times were significantly shortened.

Table 3.4 2D HEC-RAS model parameters

Parameter	Value
Equation set	Full momentum, original (SWE-ELM)
Turbulence model	None
Simulation time step	1 second
Simulation period	12 hours

3.9 Calibration, Validation, and Comparison with Other Models

The LRR has very limited data from historical events suitable for calibrating a hydraulic model. In November of 2021, while this report and the LRR model were in development, a small flood occurred that afforded the opportunity to collect additional data. Water surface elevations during the November 2021 event were staked by King County staff and later surveyed by 1 Alliance Geomatics to provide additional data for calibration. NHC calibrated the model to the November 2021 direct measurements and conducted a few qualitative checks using limited data from the November 2006 flood and other hydraulics models.

3.9.1 November 2021 Calibration

A small flood occurred on November 12, 2021, which provided an opportunity to collect additional data for model calibration. USGS provisional flow records for the LRR gage (12145500) reached 1,490 cfs (approximately equal to the mean annual flood), while the Snoqualmie River gage (12144500) peaked at 30,200 cfs. During the flood, King County staff staked 10 locations along the LRR from RM 0.4 to RM 1.5 (328^{th} Way SE bridge), as shown in Figure 3.4. Elevations were staked between 11:50 and 12:50 when

LRR flows were around 1,100 cfs. Stake elevations and locations were surveyed by 1 Alliance Geomatics. Simulated flows were confined to the main channel, had little to no interaction with vegetation on the banks, and were generally 3-4 feet deep.

Figure 3.4 Locations of LRR water surface elevation observations on November 12, 2021

The calibration was conducted by simulating the flood with recorded USGS discharges and adjusting the LRR main channel roughness values until a satisfactory match was achieved. The calibrated roughness coefficients selected were 0.044 and 0.054 for the lower and upper reaches, respectively. Figure 3.5 depicts the calibrated water surface profile with observed points superimposed. One data point (RR-8) was rejected from the November 2021 dataset after profile comparisons showed it was implausible compared to the other measured data points and the model's prevailing water surface slope. Final calibration results comparing the simulated water surface elevations to the observed elevations are shown in Table 3.5. Summary statistics for the errors from Table 3.5 are shown in Table 3.6. The calibration results in a mean error near zero and good measures of overall error with no large outliers (other than data point RR-8, which was discarded), as shown in the mean absolute error and root-mean-square error metrics.

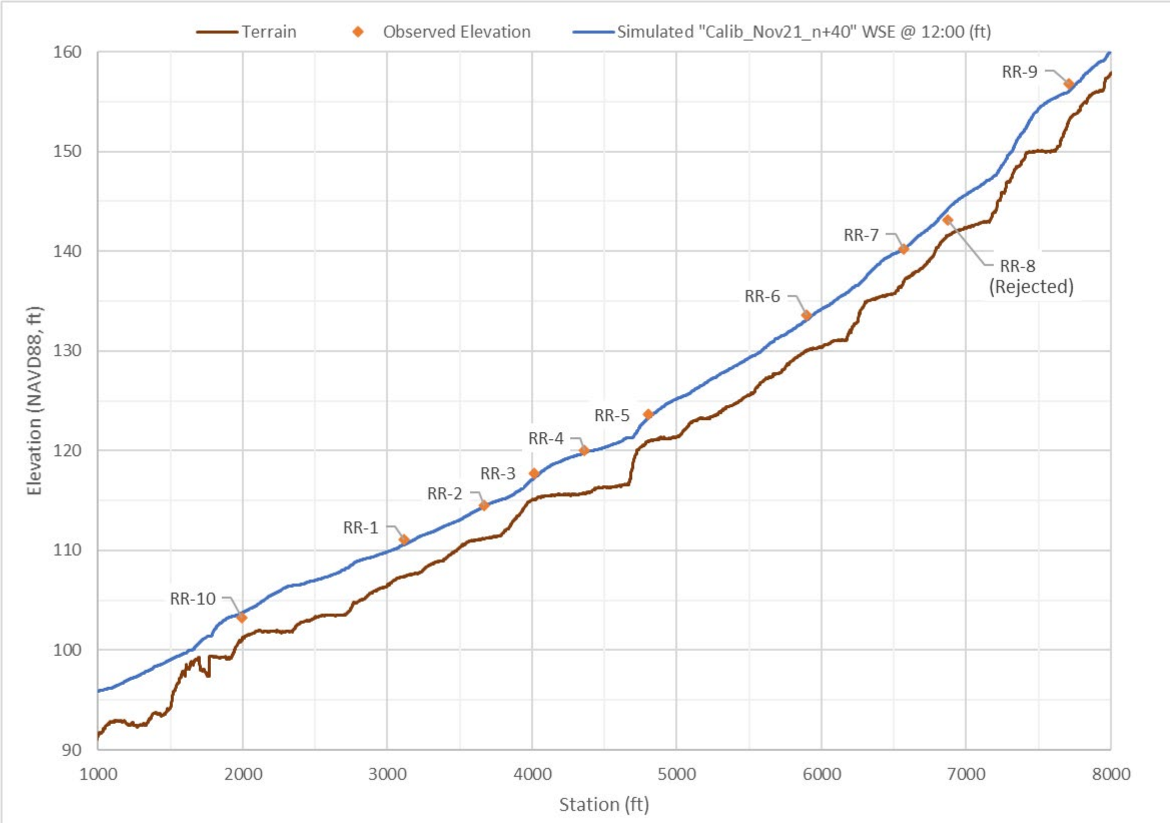


Figure 3.5 Calibrated water surface profile (November 2021 flood)

Table 3.5 Calibration results (November 2021 flood)

Description	Location	Observed Elevation (ft)	Simulated Elevation (ft)	Difference (Sim-Obs) (ft)
RR-10 RB 12:50PM	RM 0.4 (DS)	103.30	104.00	+0.70
RR-1 LB 11:50AM	RM 0.6	111.12	110.72	-0.40
RR-2 RB 11:50AM	RM 0.7	114.51	114.47	-0.04
RR-3 RB 12:00PM	RM 0.76	117.77	117.51	-0.26
RR-4 RB 12:05PM	RM 0.82	119.99	120.01	+0.02
RR-5 RB 12:15PM	RM 0.9	123.64	123.37	-0.27
RR-6 RB 12:20PM	RM 1.1	133.57	133.15	-0.42
RR-7 RB 12:25PM	RM 1.2	140.22	140.42	+0.20
RR-8 RB 12:30PM [1]	RM 1.3	143.18	N/A [1]	N/A [1]
RR-9 RB 12:40PM	RM 1.5 (US)	156.77	156.50	-0.27

Notes:

1. Data point rejected, outlier.

Table 3.6 Summary error statistics in feet for the November 2021 flood simulation

Simulation	Number of Points	Mean Error	Root Mean Square Error	Mean Absolute Error	Max Error	Min Error
November 2021	9	-0.08	0.35	0.29	0.70	-0.42

3.9.2 November 2006 Flood Validation

A search for aerial flood photos on the King County iMap site showed photos of the Raging River during flood events in November 2006, January 2009, and December 2015 (King County, 2021). These flood events are the three largest flood events that have occurred in the last twenty years. Only one photo from these three floods was suitable for estimation of water levels along the Raging River, with inundation captured during the 2006 event at the flat bench inside of the left bank levee (RM 1.0), as shown in Figure 3.6 and Figure 3.7. The edge of water was located on the photo and the WSEL was estimated by extracting the LiDAR terrain model elevations at that location. Applying this technique to gently sloping surfaces minimizes the vertical error given the approximate nature of horizontal positioning required. The estimated water surface elevation from the photo is 129.3 feet. While the photo provided a usable water level, the time stamp on the photo (January 7, 2006, 14:17) was clearly wrong. The flood peaked at the USGS gage at 15:45 on November 6, and sunset occurred at 16:44. Flood photos of the Fall City area, presumably taken within minutes of the photo described here, show no vehicles with headlights on, so it is assumed the photo was taken well before sunset and peak of the flood. Based on the calibrated model rating curve at the photo location, flows were likely around 3,000 cfs when the photo was taken, which corresponds to around 13:00 on November 6. The peak flow for the November 2006 flood event was 4,520 cfs and as previously mentioned, occurred at 15:45. Because the exact time the photo was taken cannot be ascertained, the estimated water level from the photograph can only be used as a minimum threshold for the flood peak water surface elevation. In other words, there is high confidence the peak water level was equal to or greater than the estimated water level from the photograph, but how much higher the peak was than the water level in the photograph cannot be accurately determined. Figure 3.8 shows the simulated peak water levels for the 2006 flood at the photo location reached 131 feet: almost two feet higher than the estimated water level from the photo.

Figure 3.6 Raging River high water level location observed on November 6, 2006

Figure 3.7 Raging River high water level observed on November 6, 2006 (time of photo unknown)

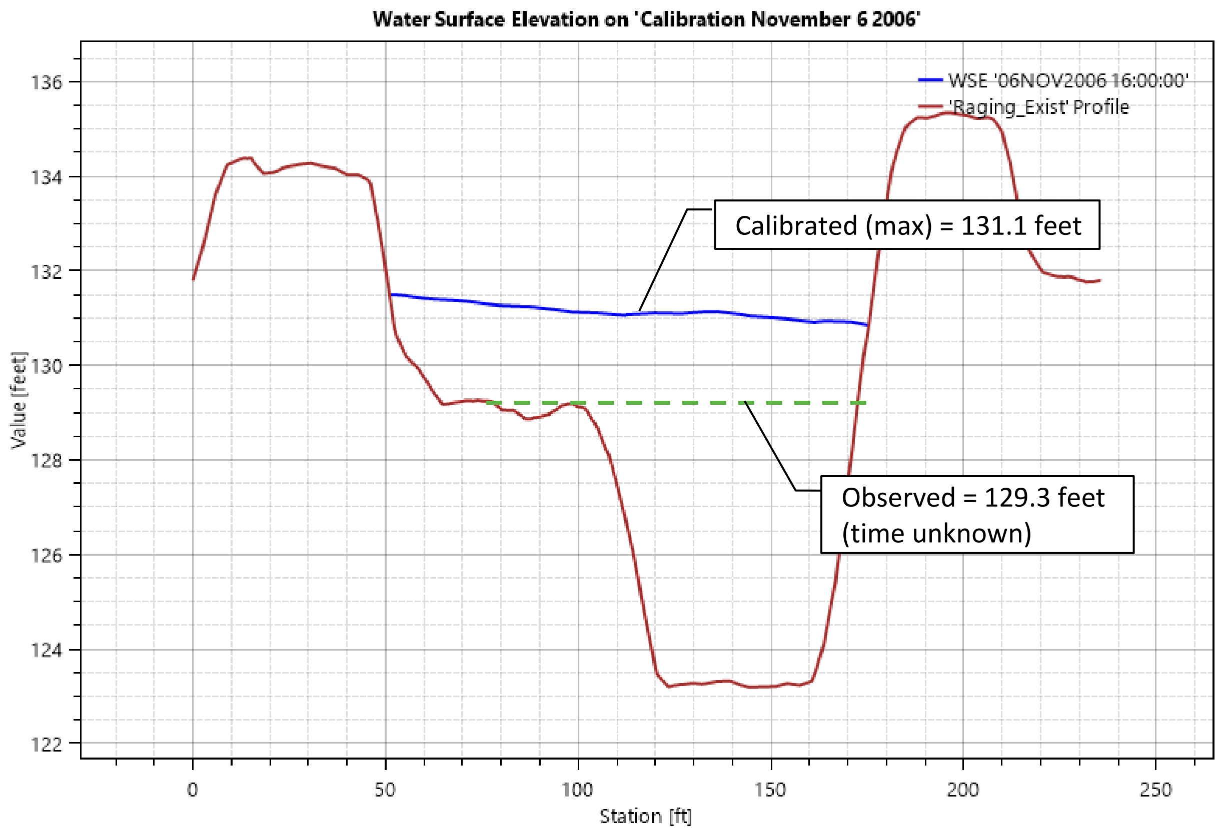


Figure 3.8 2D HEC-RAS model calibration section (2006 flood)

A second photo from the 2006 event (Figure 3.9) provided further qualitative validation of the model near the confluence of the Raging and Snoqualmie rivers. Figure 3.9 depicts the observed inundation at the Raging River's confluence with the Snoqualmie on November 6, 2006. The photo was captured at an unknown time. The Snoqualmie River rose from 8,000 cfs to around 50,000 cfs (as reported at USGS gage 1214450 Snoqualmie River near Snoqualmie) on November 6 and did not crest until the next day. The simulated event exhibited similar inundation limits, as shown in Figure 3.10. In both the simulated and observed events, the Raging River right levee is overtopped approximately 670 feet above the confluence, directly across from SE 43rd Street. A majority of the Twin Rivers Golf Course is inundated in both cases. In addition, both observed and simulated events capture the Snoqualmie River overtopping the right bank upstream of the confluence. Overall, the simulated and observed events provide a good qualitative comparison in the vicinity of the confluence, and indicate the model is simulating Snoqualmie River stages reasonably.

Figure 3.9 Observed inundation November 6, 2006 (time of photo unknown)

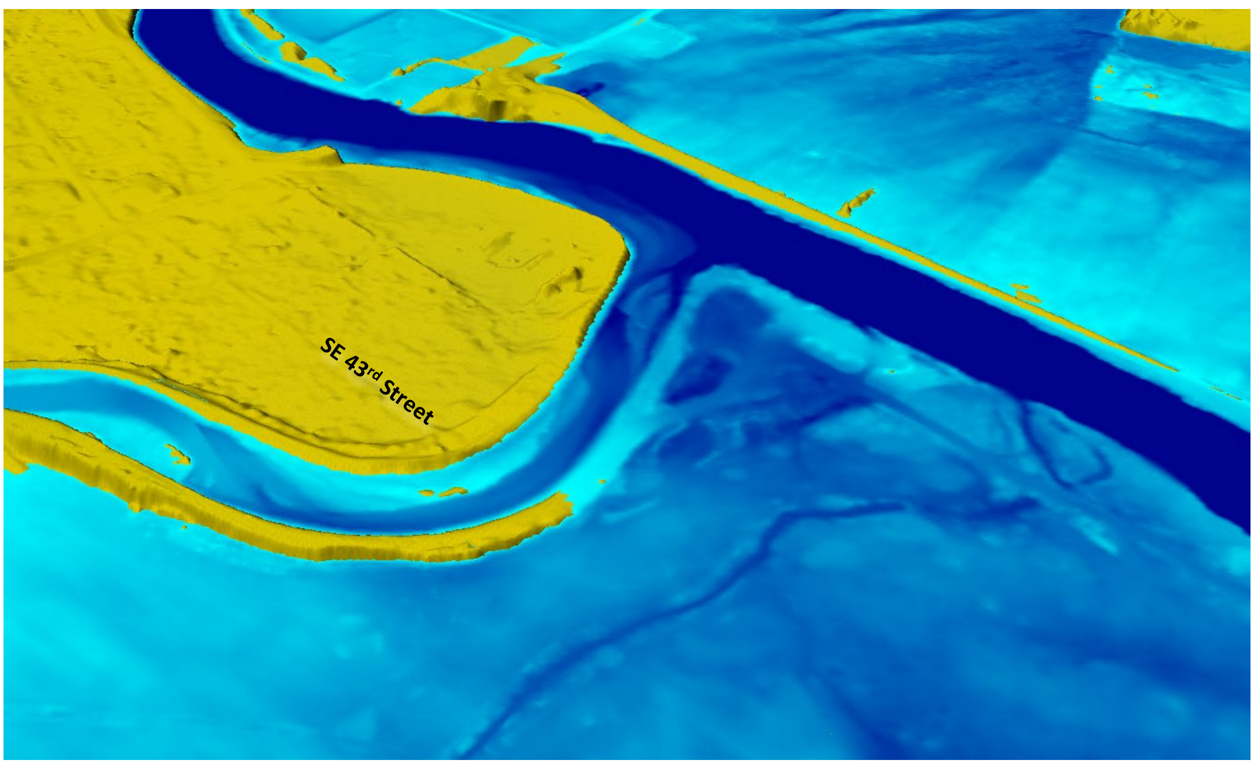


Figure 3.10 Simulated inundation (3D) on November 6, 2006 (peak occurred at 16:00)

3.9.3 Comparison with other hydraulic models

Using the calibrated model, NHC compared the 100-year simulated WSELs along the LRR to the FEMA effective Base Flood Elevations (BFEs). A comparison plot is shown in Figure 3.11. Near the confluence, the 2D HEC-RAS model generally simulates WSELs one to two feet higher than the FEMA mapping. Direct comparisons cannot be made at the confluence location, as the 100-year stage in the 2D HEC-RAS model was forced to match results from the coincidence analysis (Section 2.4.2). Backwater effects from the confluence extend approximately 1,800 feet upstream during the 100-year event (see the profile plot in Appendix C). Upstream of the confluence, the simulated model shows full containment of the 100-year flow within the levee system while the FEMA effective mapping, based on survey data collected in 1993, shows inundation limits without levees, as they are unaccredited structures. Simulated WSELs in the upper reach are generally one to two feet lower than the FEMA BFEs. The simulated WSEL decrease is likely due to the lower inflows used in the LRR model (5,830 cfs) than the effective study (6,970 cfs).

Results from the 2D HEC-RAS model were also compared to the Fall City Restoration Project model (Watershed Science and Engineering, 2020). This model was constructed using RiverFlow2D to simulate the 100-year flood through the Snoqualmie River valley. NHC prepared a difference plot to compare the 100-year WSELs, as shown in Figure 3.12. Near the confluence and throughout the Snoqualmie River influenced area, the 2D HEC-RAS model simulates WSELs approximately one to two feet higher than the Fall City Restoration model, even though the Fall City Restoration model used a 100-year peak flow of 83,065 cfs while the 2D HEC-RAS targeted inflow was 75,000 cfs. Two factors are likely responsible for the 2D HEC-RAS model’s higher WSELs under lower flow conditions. First, the largest differences occur from the SR202 bridge upstream some distance and are probably due to differences in how the two models treat bridge hydraulics. Second, no attempt was made to calibrate the Snoqualmie River portion of the 2D HEC-RAS model; the Fall City Restoration Project RiverFlow2D roughness values were used. 2D HEC-RAS typically requires lower roughness values than other 2D hydraulic models due to its solution methods, so it would be expected that the WSELs for a 2D HEC-RAS model would be higher than for an equivalent RiverFlow2D model with the same roughness values. Regardless, direct comparisons are not applicable in the Snoqualmie River floodplain area as the 2D HEC-RAS model’s 100-year stage was forced at the confluence to match results from the coincidence analysis (Section 2.4.2), not from a Snoqualmie River 100-year flow event.

In the Raging River, the 2D HEC-RAS model generally simulates lower water surface elevations. The flows used for the Raging River were very similar between both models, but this portion of the Fall City Restoration Project model was not calibrated.

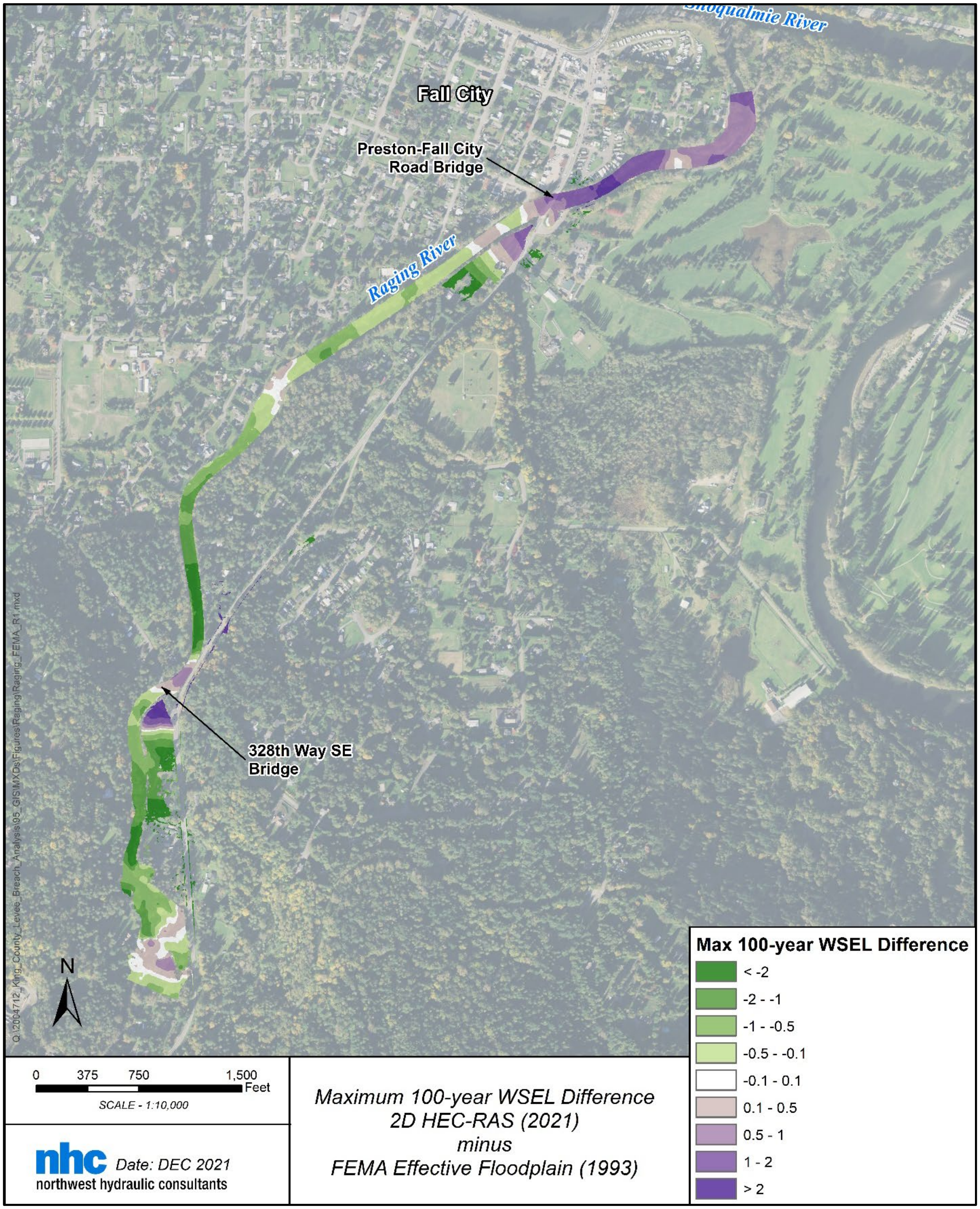


Figure 3.11 Maximum 100-year WSEL difference – 2D HEC-RAS (2021-5,830 cfs) minus the FEMA FIS effective floodplain (1993-6,970 cfs)

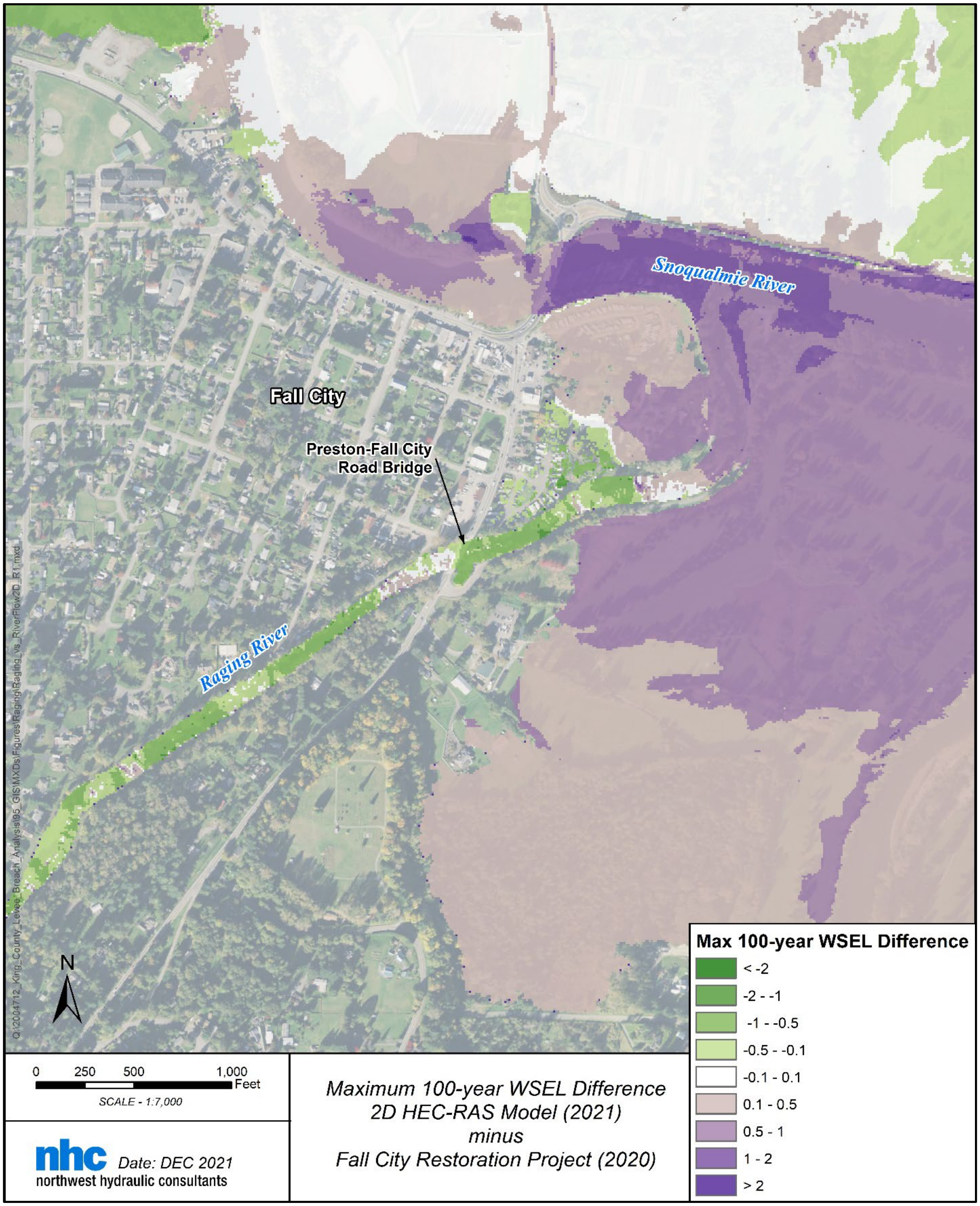


Figure 3.12 Maximum 100-year WSEL difference – 2D HEC-RAS model (2021-5,830 cfs) minus the Fall City Restoration Project (2020-5,827 cfs)

3.10 Sensitivity Testing

3.10.1 Flow Sensitivity

NHC assessed the model sensitivity to flow rate by comparing results from all the existing condition simulated floods, ranging from the 2- to 500-year future hydrologic condition scenarios (LRR inflow of 2,020 to 10,480 cfs). Changes in flood stage between the 2- and 100-year events is generally 3.5 feet along a majority of the reach and increases to 7.5 feet near the confluence. Stage increases between the 2- and 500-year events are generally 4.5 feet along a majority of the river alignment and 9.5 feet near the confluence. Stage changes approximately 0.8 to 1 foot per thousand cfs increase in flow in the LRR reach not affected by Snoqualmie River stage. Referring to Table 2.2 and using this relationship to convert flow to stage; at a 10-year flow the 5% and 95% confidence limit band is about one foot, increasing to four feet for the 100-year flood. Uncertainty in flow estimates is part of the Flood Damage Reduction Analysis (HEC-FDA) process for determining flood risk and levee reliability.

3.10.2 Parameter Sensitivity Testing

3.10.2.1 Time Step Sensitivity

The model's computational time step was selected from a series of preliminary runs to accurately simulate hydraulics in the study reach within reasonable run times. The simulation time step was set to 1.0 second to achieve Courant values of less than 2.0. As a sensitivity test, NHC reduced the time step to 0.5 seconds for the 100-year flood. Computed water surface elevations along the Raging River were generally within 0.05 feet of the 1.0 second timestep results; however, the computation time doubled to 8.5 hours. As the change in results was minimal and computer processing was doubled, NHC determined that a 1.0 second timestep was sufficient for the Project needs.

3.10.2.2 Roughness Sensitivity

NHC tested the hydraulic model for sensitivity to roughness values by increasing and decreasing the calibrated roughness values by 50 percent during the 100-year event. Difference results mapping is provided in Appendix C of this report. On average, increasing roughness increased flood stage by +1.5 feet along a majority of the reach and +0.8 feet near the confluence. No levee overtopping occurred through the segment from the 328th Way SE bridge and the Preston-Fall City Road bridge. Near the confluence, additional flow overtopped the left levee, causing stage increases of +1.0 feet in the developed area bound by the Raging River and Preston-Fall City Road. Conversely, decreasing roughness values resulted in stage decreases of -1.5 feet along a majority of the reach and -1.0 feet near the confluence. Near the confluence, less flow overtopped the left levee, causing stage decreases of -1.0 feet in the developed area bound by the Raging River and Preston-Fall City Road. Roughness sensitivity results will be used as a component of developing stage-frequency uncertainty curves for the risk analysis.

3.10.2.3 Downstream Boundary Condition Sensitivity

The initial downstream boundary condition for the model consists of a discharge-stage rating curve. NHC performed a sensitivity analysis on the downstream boundary condition to ensure the selected rating curve did not impact hydraulics in the Project area. To test impacts, the downstream boundary condition was shifted vertically by +/- 1 foot. Results from the sensitivity analysis revealed the hydraulics in the Project area were unaffected by the selected downstream boundary condition; all changes were isolated to one mile downstream of the confluence with the Snoqualmie River.

3.10.2.4 Weir Coefficient Sensitivity

The hydraulic model was also tested for sensitivity to selected weir coefficients for the LRR levee system. Sensitivity to weir coefficients was tested with the 500-year event with future conditions impacts, as most levees were overtopping during this event. The 1D weir equation option was selected for 10 of the 14 LRR levee segments; changes in weir coefficients impacted these segments directly. The initial weir coefficient applied to all structures was 2.6. NHC increased and decreased coefficients by 20 percent (an increase of 20 percent corresponds to around the maximum coefficients in the literature for smooth broad crested weirs). When weir coefficients were increased by 20 percent, WSELs along the LRR decreased by 0.02 feet and flow over the levees increased by 12 percent on average. The increase in flow across the left levee caused WSELs to increase by 0.2 feet in the developed area bounded by the LRR and Preston-Fall City Road. Developed areas west of Preston-Fall City Road were impacted minimally, with a 0.01-foot increase. When weir coefficients were decreased by 20 percent, WSELs along the LRR increased by 0.02 feet and flow over the levees decreased by 13 percent on average. The decrease in levee flow caused WSELs to lower by 0.2 feet on average in the developed area bounded by the LRR and Preston-Fall City Road. Developed areas west of Preston-Fall City Road were impacted minimally, with a 0.02-foot decrease, generally. The testing indicates flood inundation and depth results are not very sensitive to weir flow coefficients for overtopping levees.

3.11 Hydraulic Model Results

Results mapping of flow depths, velocities, and water surface elevations for all flood events are provided in Appendix C. During the 2-year event, flow begins to overtop the LRR right bank levee near the confluence. The overtopping is partially related to WSELs in the LRR and partially influenced by the upstream Snoqualmie River floodplain. Flow between the 5- and 10-year events significantly overtops the LRR right bank levee near the confluence. During these events, a majority of the Twin Rivers Golf Course (east of the LRR) is inundated by the Snoqualmie River floodplain. The LRR first overtops the left bank levee near the confluence during the 25-year event, flooding a few residences west of the confluence. During this event, inundation is mainly confined to Bernard Memorial Park, adjacent to the Snoqualmie River. The 25-year flood also causes significant ponding of the right bank floodplain directly upstream of the Preston-Fall City Bridge, resulting from river flow backwatering through the 30-inch corrugated metal pipe that penetrates through the right bank levee at RM 0.6. The 50- and 100-year inundation extents show extensive flooding of the residential area west of the LRR-Snoqualmie River confluence. Computed in-channel depths for the 100-year event are approximately 6 to 8 feet along a majority of the levee system and 10 to 13 feet near the confluence. Maximum in-channel velocities are approximately 10 fps along the levee system, and closer to 6 fps near the confluence. Backwater effects from the confluence extend approximately 1,800 feet upstream during the 100-year event (see the profile plot in Appendix C). Above the 328th Way SE bridge, there are two locations where flow overtops the main channel banks and enters the adjacent floodplain. The floodplain widths in this area range from 300 to 400 feet. At the existing conditions 500-year event, significant flooding between the Raging and Snoqualmie rivers occurs up to Preston-Fall City Road SE. During all simulated existing conditions flood events, the levees are observed to fully contain flow between the 328th Way SE bridge and the Preston-Fall City bridge with exception of ponding caused by the 30-inch CMP at RM 0.6. However, right bank flow during floods greater than the 100-year event crosses the intersection of Preston-Fall City Road and 328th Way SE and flows down the floodplain on the landward side of the right bank levee until it drains out of the 30-inch culvert at RM 0.6. The magnitude of overflow along this route is small, on the order of 100 cfs, with generally shallow flood depths.

The future conditions simulated results were also evaluated by NHC. The 100-year future conditions mapping resulted in similar results as the 500-year existing conditions mapping, due to similar inflow for the LRR. During the 500-year future conditions event, flows overtop the left- and right-bank levees between the 328th Way SE bridge and the Preston-Fall City bridge. This causes significant flooding within the Fall City area, west of Preston-Fall City Road, with depths up to 2.5 feet. Along the right overbank, flows overtop the levee and enter the right floodplain, where flows are routed to the northeast. See Appendix C for inundation mapping and water surface profiles of the future conditions events.

3.12 Quality Control

A Quality Control (QC) check was performed by an experienced HEC-RAS modeler at NHC not directly involved in the model development. This QC check occurred at the end of the model development phase and is documented in Appendix D of this report.

4 REFERENCES

CIG (2020). Hourly stream flow projections for the Raging River at Fall City. [online] Available from: https://data.cig.uw.edu/picea/mauger/2020_12_SnohoCounty_Flooding/DATA/pub/snoho_wrf_results/RagingRNrFallCity/ (Accessed 5 September 2021).

England Jr, J. F., Cohn, T. A., Faber, B. A., Stedinger, J. R., Thomas Jr, W. O., Veilleux, A. G., Kiang, J. E., and Mason Jr, R. R. (2019). *Guidelines for determining flood flow frequency—Bulletin 17C*. US Geological Survey.

FEMA (2020a). *Flood Insurance Rate Map, King County, Washington, Unincorporated Areas. Community Panel Number 53033C0709H*. [online] Available from: https://map1.msc.fema.gov/firm?id=53033C0709H (Accessed 5 October 2021).

FEMA (2020b). *Flood Insurance Rate Map, King County, Washington, Unincorporated Areas. Community Panel Number 53033C0717H*. [online] Available from: https://map1.msc.fema.gov/firm?id=53033C0717H (Accessed 5 October 2021).

FEMA (2020c). *Flood Insurance Study King County, Washington and Incorporated Areas* (53033CV001). King County, Washington.

Harper Righellis, Inc. (1993). *Raging River Flood Plain Hydrology and Hydraulics Report*. Unpublished Report to King County Surface Water Management Division.

King County (2019). *Raging River Channel Migration Study 2019*. Seattle, WA. 119 pp.

King County (2021). King County iMap. [online] Available from: https://gismaps.kingcounty.gov/iMap/ (Accessed 27 August 2021).

Microsoft Maps (2020). US Building Footprints. [online] Available from: https://services.arcgis.com/P3ePLMYs2RVChkJx/arcgis/rest/services/MSBFP2/FeatureServer (Accessed 6 February 2022).

National Agricultural Imagery Program (2019). *2019 Washington NAID Digital Ortho Photo Imagery*.

NHC (2006). *Flood Insurance Mapping Study for the Snoqualmie River (Skykomish River Confluence to Snoqualmie Falls) and Skykomish River (Snoqualmie River Confluence to RM 8.95)* (Project No. 20261). King and Snohomish Counties, WA. 43 pp. [online] Available from: https://snohomishcountywa.gov/DocumentCenter/View/6664/Lower-Skykomish-River-and-Snoqualmie-River-Floodplain-Mapping-Project-TSDN.

PRISM Climate Group (2019). 30-Year Normals: Northwest Alliance for Computational Science and Engineering. Oregon State University Database. [online] Available from: https://prism.oregonstate.edu/ (Accessed 5 September 2020).

Quantum Spatial (2020). *LiDAR Remote Sensing Snoqualmie River on behalf of King County*.

Watershed Science and Engineering (2016). *Snoqualmie River Hydraulic Study: Evaluation of Effects of the Snoqualmie Falls Projects on Downstream Flooding*.

Watershed Science and Engineering (2019). *Levee Breach Analysis for King County Rivers*.

Watershed Science and Engineering (2020). *Technical Memorandum for Haffner-Barfuse Floodplain Restoration Project – Current Conditions Hydraulic Modeling*.

WDFW (2018). Culverts and Climate Change Web App. [online] Available from: https://wdfw.wa.gov/species-habitats/habitat-recovery/fish-passage/climate-change (Accessed 5 December 2021).

APPENDIX A

FUTURE CLIMATE PROJECTIONS MEMORANDUM

1 APPENDIX A: CALCULATION OF MULTIPLICATION FACTORS FOR FUTURE CONDITIONS HYDROGRAPHS

Future conditions hydrographs were developed by scaling the existing conditions balanced hydrographs using multiplication factors derived from hydrologic projections for the Raging River by the Climate Impacts Group (CIG) at the University of Washington (CIG, 2020)[1]. This Appendix describes how these multiplication factors were calculated.

1.1 Future Hydrologic Conditions

Future conditions hydrographs were developed by scaling the existing conditions balanced hydrographs using multiplication factors derived from hourly hydrologic projections for the Raging River at Fall City stream gage created by the CIG at the University of Washington. Multiplication factors for 1hr, 24hr and 72hr durations were used, obtained from the same hydrologic projections.

The projections used are from the CMIP5 database and for the future pathway of global greenhouse gas emissions and atmospheric concentrations known as RCP8.5[2]. RCP 8.5 is the highest of the future pathways for which a large number of global climate models were run to obtain future climate projections. The peak flow projections are summarized in Section 1.1.1 and are interpreted in relation to atmospheric river projections (given the association of large historical peak flow events with atmospheric rivers) in Section 1.1.2. The main sources of uncertainty associated with these projections are summarized in Section 1.1.3.

1.1.1 Peak Flow Projections

Projections of future flood flows were created as follows:

a. Flood quantiles were created by CIG, for return periods up to 500 years, for each GCM for the historical period and the future time horizon of interest, defined by the water years 1981-2010 (the "1990s") and 2040-2069 (the "2050s"). Quantiles were obtained by fitting a generalized extreme value (GEV) distribution to the series of annual maximum hourly flows, 24-hour flows, and 72-hour flows.

b. NHC calculated the ratio of the flood quantiles in (a) for the future time horizon of the 2050s against the historical time horizon of the 1990s, to obtain a percent change in peak flows for each return period. The percent change varied between GCMs, and the median, minimum and maximum value were recorded for each return period (Table 1.1).

c. Flood peak quantiles were calculated by NHC for the observed series of annual maximum instantaneous peak flow, using the data reported for the Fall City stream gage for 1945-2021[3]. This was done by fitting a Log Pearson III distribution to the observed series following Bulletin

[1] CIG's hourly stream flow projections for the Raging River at Fall City were downloaded from: https://data.cig.uw.edu/picea/mauger/2020_12_SnohoCounty_Flooding/DATA/pub/snoho_wrf_results/RagingRNrFallCity/

[2] RCP8.5 refers to a climate state with an average rate of 8.5 Watts/m^2 net radiative flux surplus imbalance by year 2100.

[3] For the current water year of 2021 the peak flow was not available and was replaced by the 15-minute peak flow.

17C methods, using HEC-SSP software. To reflect the uncertainty in estimating the distribution's parameters, 95% confidence interval limits were calculated. For 24hr and 72hr peaks, 15-minute observed stream flow data from 1988-2021 was aggregated to these durations using running means, and the time series annual maxima was fitted to a generalized extreme value distribution. All results are given in Table 1.2 and plotted in Figure 1.1.

d. Future median flood quantiles were calculated by multiplying the observed quantiles in (c) by the percent changes calculated in (b). Results are given in Table 1.2 and plotted in Figure 1.1. Median, minimum and maximum quantiles were calculated based on the percentage changes in the three right-most columns of Table 1.1

Note that the process described above modifies the observed instantaneous peak flows with scaling factors derived from CIG's hourly data. NHC verified this was not a concern by comparing the observed peak hourly flows to the observed instantaneous peaks for the overlapping period where both datasets are available (1989-2020). The result, shown in Figure 1.2, exhibits a very high degree of correlation.

The increase in flood quantiles projected by most GCMs studied is consistent with a projected increase in the number of landfalling atmospheric rivers arriving in the Pacific Northwest in the cool season and/or in the intensity of their moisture transport identified in studies of larger ensembles of GCMs for RCP 8.5 (e.g., Hagos et al, 2016[4]; Warner and Mass, 2017[5]). As shown in Figure 1.3, the largest floods experienced in the Raging River have been associated with the landfall of an atmospheric river.

[4] Hagos, S. M., L. R. Leung, J.-H. Yoon, J. Lu, and Y. Gao, 2016: A projection of changes in landfalling atmospheric river frequency and extreme precipitation over western North America from the large ensemble CESM simulations. Geophys. Res. Lett., 43, 1357–1363, doi:10.1002/2016GL067392.

[5] Warner, M. D., and C. F. Mass (2017) Changes in the Climatology, Structure, and Seasonality of Northeast Pacific Atmospheric Rivers in CMIP5 Climate Simulations. *J. Hydrometeorology*, 18, 2121-2141, doi: 10.1175/JHM-D-16-0200.1.

Table 1.1 Percent changes in flows projected for the 2050s (compared to the 1990s) by the different GCMs for different return periods. Median, minimum and maximum changes are on the columns on the right.

Return Period (Years)	access1.0	access1.3	bcc-csm1.1	canesm2	ccsm4	csiro-mk3.6.0	fgoals-g2	gfdl-cm3	giss-e2-h	miroc5	mri-cgcm3	Noresm1-m	MEDIAN	MINIMUM	MAXIMUM
							Hourly Flows								
1.0101	47%	16%	-17%	5%	12%	3%	56%	64%	55%	13%	16%	-16%	13%	-17%	64%
2	-1%	23%	8%	14%	14%	-7%	17%	32%	8%	20%	2%	2%	11%	-7%	32%
5	2%	19%	19%	9%	14%	3%	19%	33%	0%	28%	9%	9%	11%	-13%	33%
10	7%	15%	26%	5%	13%	12%	22%	35%	-4%	34%	13%	13%	13%	-21%	35%
20	13%	10%	34%	1%	13%	23%	27%	39%	-8%	39%	18%	18%	15%	-28%	39%
50	21%	4%	44%	-4%	13%	40%	34%	44%	-11%	46%	2%	-35%	22%	-35%	46%
100	28%	-2%	51%	-7%	12%	55%	39%	48%	-13%	51%	9%	-40%	27%	-40%	55%
500	47%	-13%	69%	-13%	11%	98%	53%	58%	-17%	64%	13%	-50%	41%	-50%	98%
							24-hr Flows								
1.0101	103%	-5%	81%	55%	-41%	252%	126%	12%	43%	-35%	-3%	-50%	27%	-50%	252%
2	2%	9%	8%	10%	6%	-17%	-3%	27%	-4%	9%	3%	-4%	5%	-17%	27%
5	-1%	8%	9%	11%	12%	-13%	3%	33%	-5%	17%	-6%	4%	6%	-13%	33%
10	1%	5%	15%	13%	11%	-6%	14%	37%	-4%	22%	-14%	8%	10%	-14%	37%
20	4%	2%	26%	16%	10%	2%	31%	41%	0%	26%	-22%	11%	10%	-22%	41%
50	12%	-1%	49%	21%	5%	14%	63%	45%	5%	30%	-34%	14%	14%	-34%	63%
100	18%	-4%	70%	25%	2%	25%	95%	47%	11%	33%	-41%	16%	21%	-41%	95%
500	34%	-10%	139%	36%	-8%	54%	210%	55%	25%	39%	-57%	21%	35%	-57%	210%
							72-hr Flows								
1.0101	-1%	-15%	89%	294%	85%	-54%	-41%	310%	-94%	-25%	21%	-71%	-8%	-94%	310%
2	20%	31%	-2%	8%	29%	22%	15%	37%	11%	-11%	-12%	21%	17%	-12%	37%
5	8%	25%	7%	4%	21%	17%	20%	22%	10%	-13%	-16%	38%	13%	-16%	38%
10	-2%	17%	20%	4%	19%	8%	23%	19%	7%	-10%	-17%	47%	13%	-17%	47%
20	-10%	7%	37%	5%	18%	-2%	23%	20%	7%	-6%	-18%	55%	7%	-18%	55%
50	-21%	-5%	70%	8%	19%	-15%	24%	23%	-2%	1%	-18%	67%	5%	-21%	70%
100	-30%	-14%	102%	11%	20%	-24%	23%	27%	-7%	8%	-18%	76%	9%	-30%	102%
500	-46%	-33%	209%	18%	23%	-42%	24%	37%	-16%	26%	-17%	97%	21%	-46%	209%

Table 1.2 **Observed and projected flows for different return periods for the Raging River at Fall City. The confidence intervals pertain to the observed flows and reflect uncertainty in fitting of the distribution.**

Return Period (Years)	Observed Flows (cfs)	Observed Flow 95% Conf. Int. (Lower) (cfs)	Observed Flow 95% Conf.Int. (Upper) (cfs)	2050s Projected Median (cfs)	2050s Projected Minimum (cfs)	2050s Projected Maximum (cfs)
Hourly Flows						
1.0101	(*) 657	461	850	743	545	1,078
2	2,016	1,813	2,239	2,238	1,875	2,661
5	2,981	2,688	3,329	3,308	2,593	3,964
10	3,645	3,268	4,217	4,119	2,880	4,921
20	4,297	3,731	5,215	4,942	3,094	5,973
50	5,163	4,213	6,782	6,299	3,356	7,538
100	5,830	4,516	8,210	7,404	3,498	9,036
500	7,436	5,096	12,593	10,484	3,718	14,722
24-hr Flows						
1.0101	(**) 470	119	779	598	237	1,653
2	1,554	1,381	1,763	1,624	1,282	1,977
5	2,188	1,917	2,483	2,313	1,909	2,912
10	2,603	2,188	3,054	2,853	2,244	3,567
20	2,996	2,359	3,788	3,298	2,323	4,220
50	3,500	2,609	4,887	3,992	2,318	5,716
100	3,872	2,681	5,990	4,693	2,271	7,568
500	4,720	2,800	9,220	6,383	2,038	14,625
72-hr Flows						
1.0101	(**) 321	68	549	294	19	1,318
2	1,047	929	1,187	1,226	916	1,437
5	1,423	1,237	1,580	1,614	1,202	1,961
10	1,651	1,400	1,867	1,858	1,378	2,428
20	1,854	1,504	2,172	1,987	1,529	2,878
50	2,097	1,619	2,671	2,194	1,647	3,572
100	2,265	1,650	3,076	2,473	1,596	4,580
500	2,614	1,759	4,337	3,152	1,424	8,076

* The observed flows given under “Hourly Flows” are instantaneous peak flows for the full period for which such data is available (1945-2021).

** The observed flows given under “24-hr Peak Flows” and “72-hr Peak Flows” are obtained from the series of annual maxima of flows aggregated to those durations

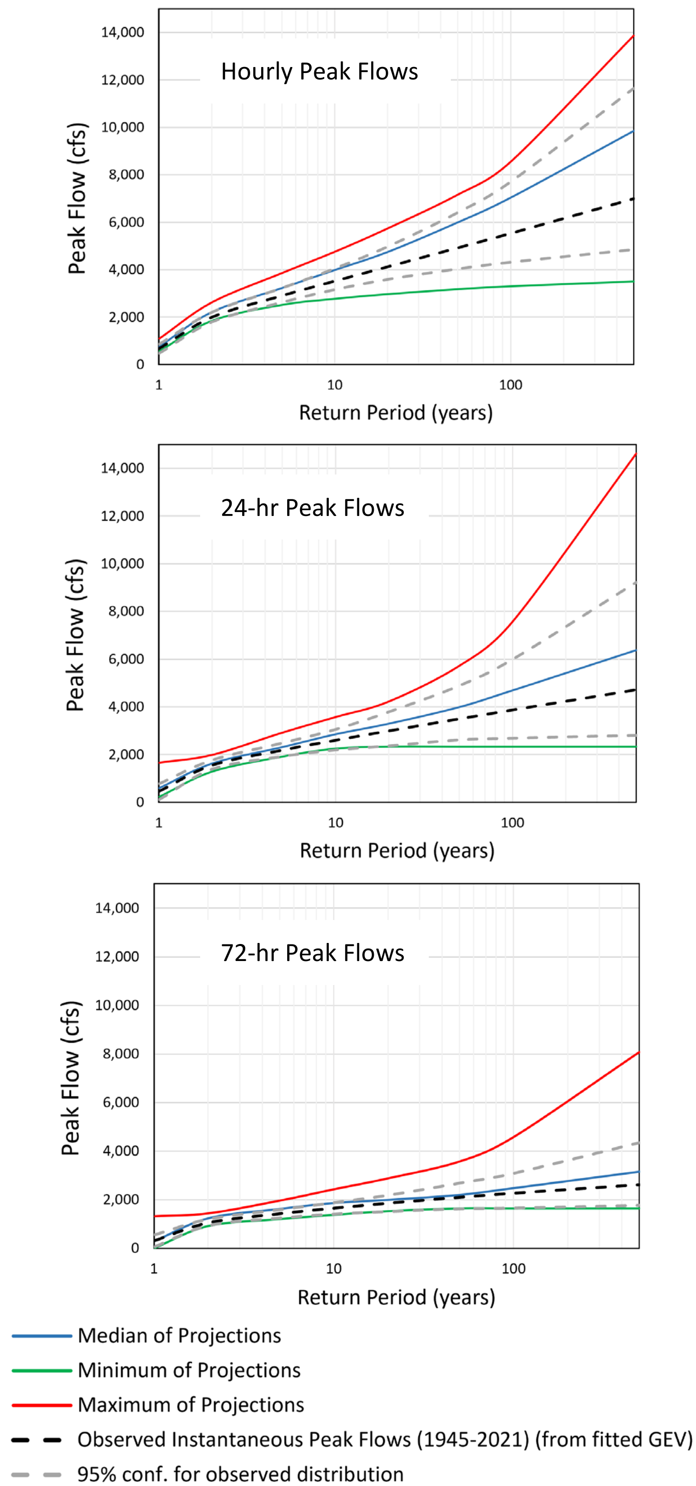


Figure 1.1 Projected Peak Flows for the 2050s time horizon. The data plotted are listed in Table 1.2. The green lines (minimum of projections) were constrained to not decline for higher return periods.

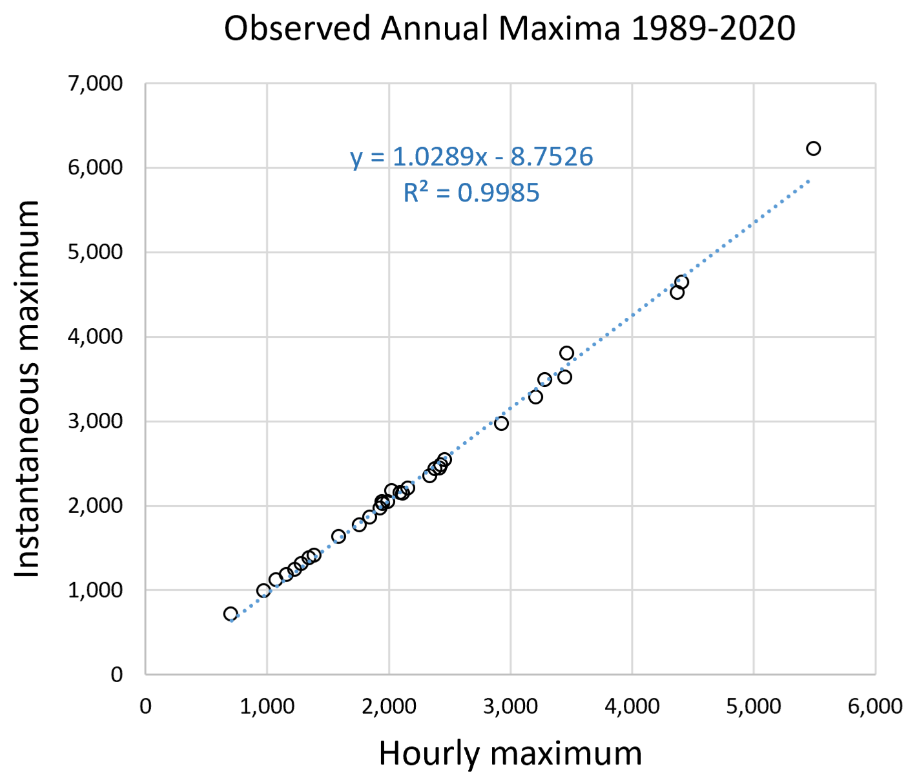


Figure 1.2 Relationship between hourly and instantaneous maximum (peak) flow each water year in the overlapping period of the two records, 1989 to 2020.

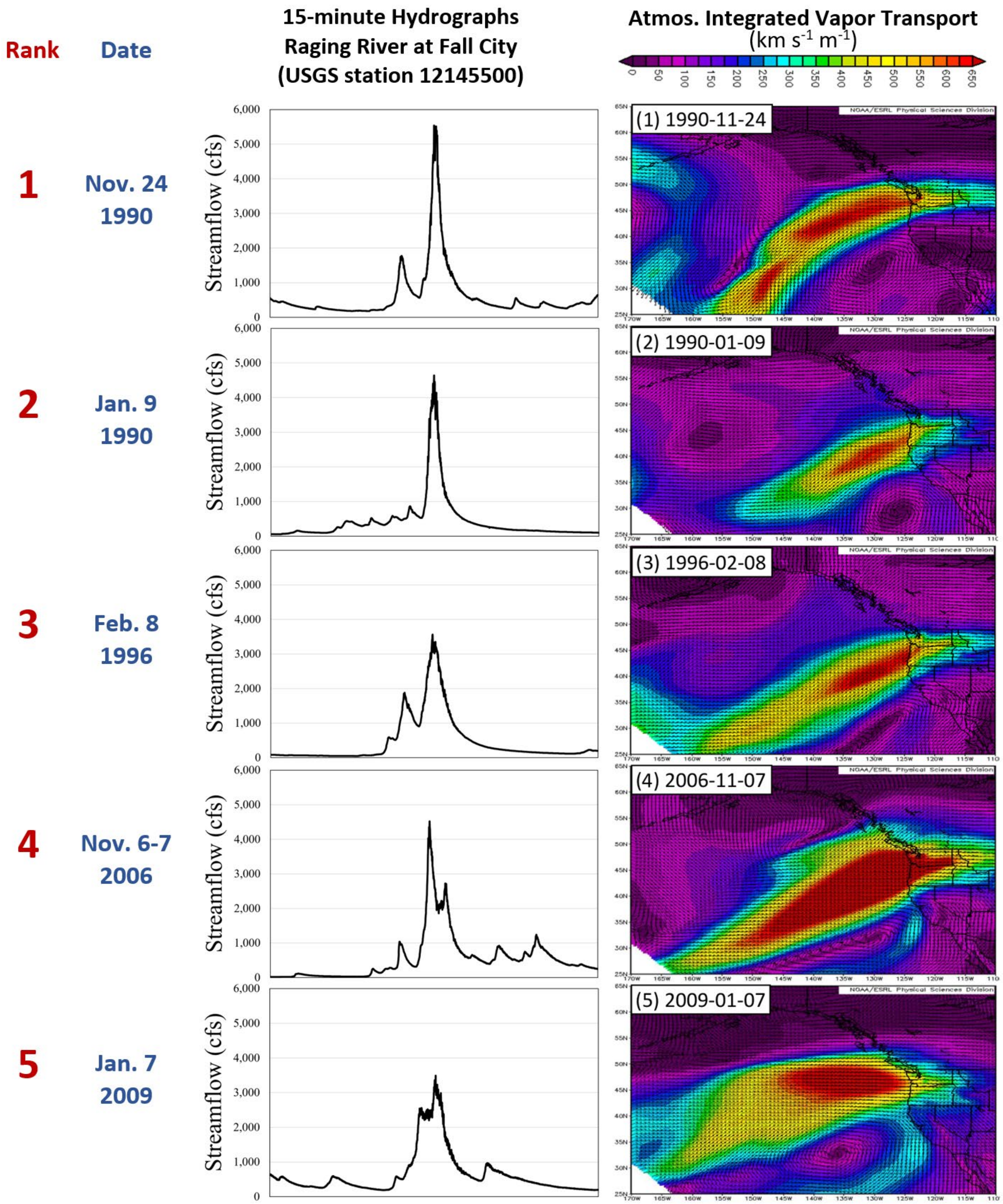


Figure 1.3 **Hydrographs and maps of moisture transported by the atmosphere, for the 5 largest flow events. Events were ranked based on maximum 24-hour flow. The hydrographs have 15-minute resolution and the x axis spans 20 days centered on the peak. The maps are derived from satellite imagery (Neiman et al., 2011, J. Hydrometeorology, doi:10.1175/2011JHM1358.1) and show the eastern Pacific and Western North America. The red areas have the most intense flux of moisture, forming an atmospheric river. Each major flooding event in Washington's rain-dominated watersheds is associated with an atmospheric river.**

1.1.2 Uncertainty of these Flood Projections

While there is a need to provide quantitative information for adaptation to future changes in hydrologic regime and extreme event frequency and intensity, the underlying projections of climate change are subject to large and unquantifiable uncertainty. The main sources of uncertainty are unknown future global emissions of greenhouse gases, uncertain response of the global climate system to increases in greenhouse gas concentrations, and incomplete understanding of regional manifestations that will result from global changes.

The downscaling in space of GCM-projected climate variables, the application of the hydrologic model, and any extrapolation of frequency analyses to extreme return periods, all represent additional sources of uncertainty. The hydrologic projections developed in this work should therefore be considered to be plausible representations of the future, given the best current scientific information, and do not represent specific predictions. The actual future realizations of streamflow and other hydrologic variables in the Raging River watershed may differ from any of these scenarios, and their difference compared to historical values may be greater or smaller than the differences projected in this work.

APPENDIX B

WDFW FUTURE PROJECTIONS FOR CLIMATE CHANGE (PATTERSON CREEK)

Future Projections for Climate-Adapted Culvert Design

Project Name:	Patterson Creek Future Projection
Stream Name:	Patterson Creek
Drainage Area:	13,277 ac

Projected mean percent change in bankfull flow:

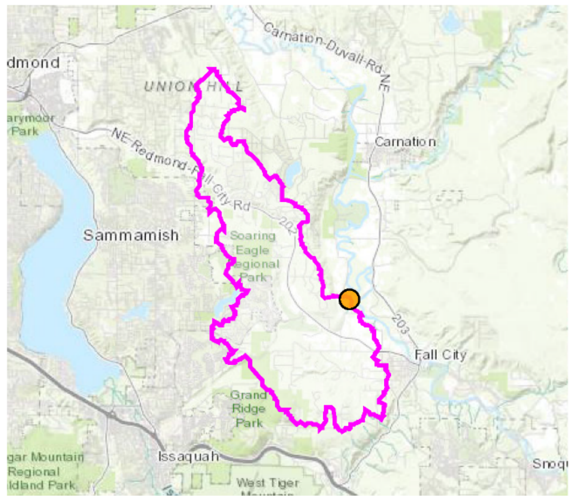
2040s:	15.2%
2080s:	20.3%

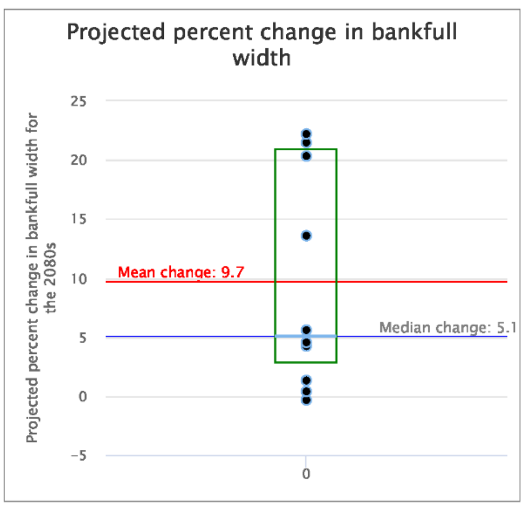
Projected mean percent change in bankfull width:

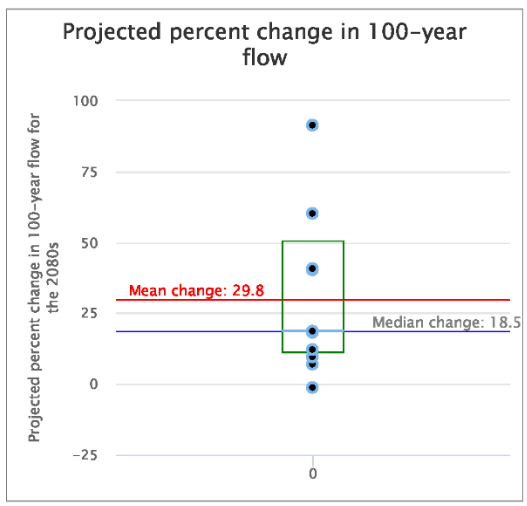
2040s:	7.4%
2080s:	9.7%

Projected mean percent change in 100-year flood:

2040s:	20.6%
2080s:	29.8%



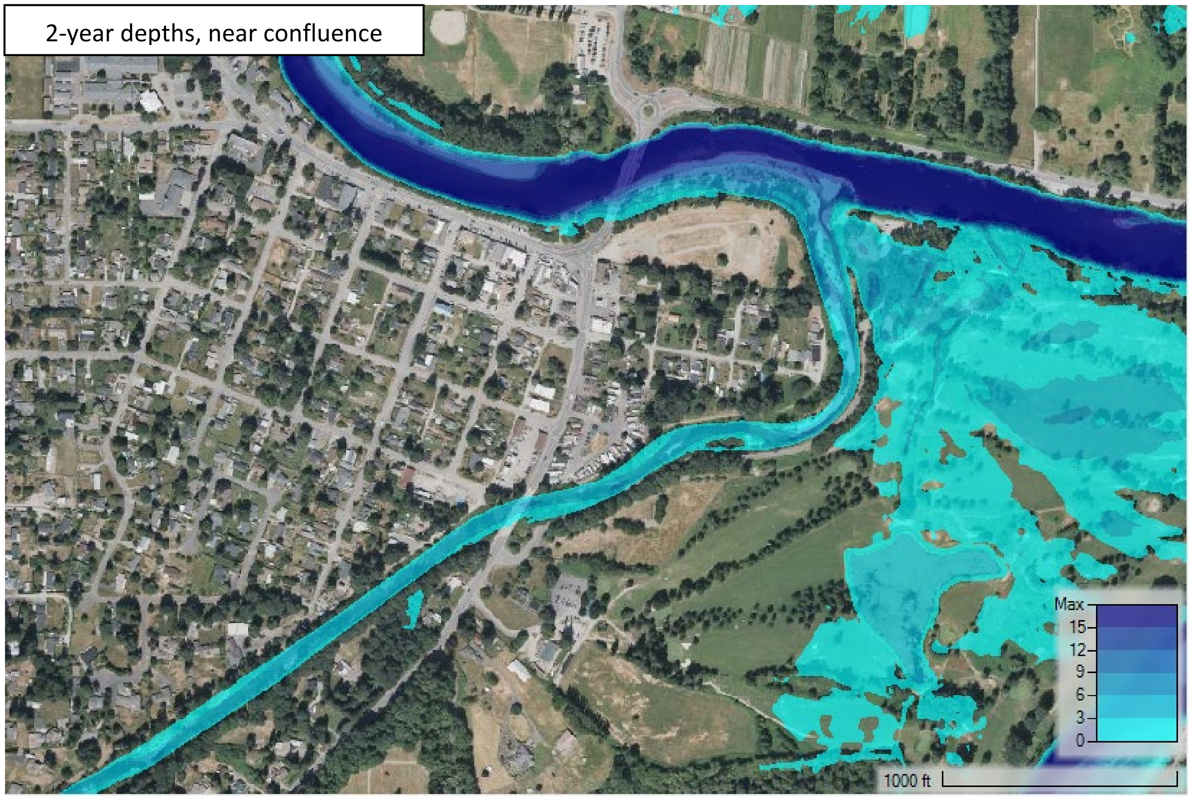


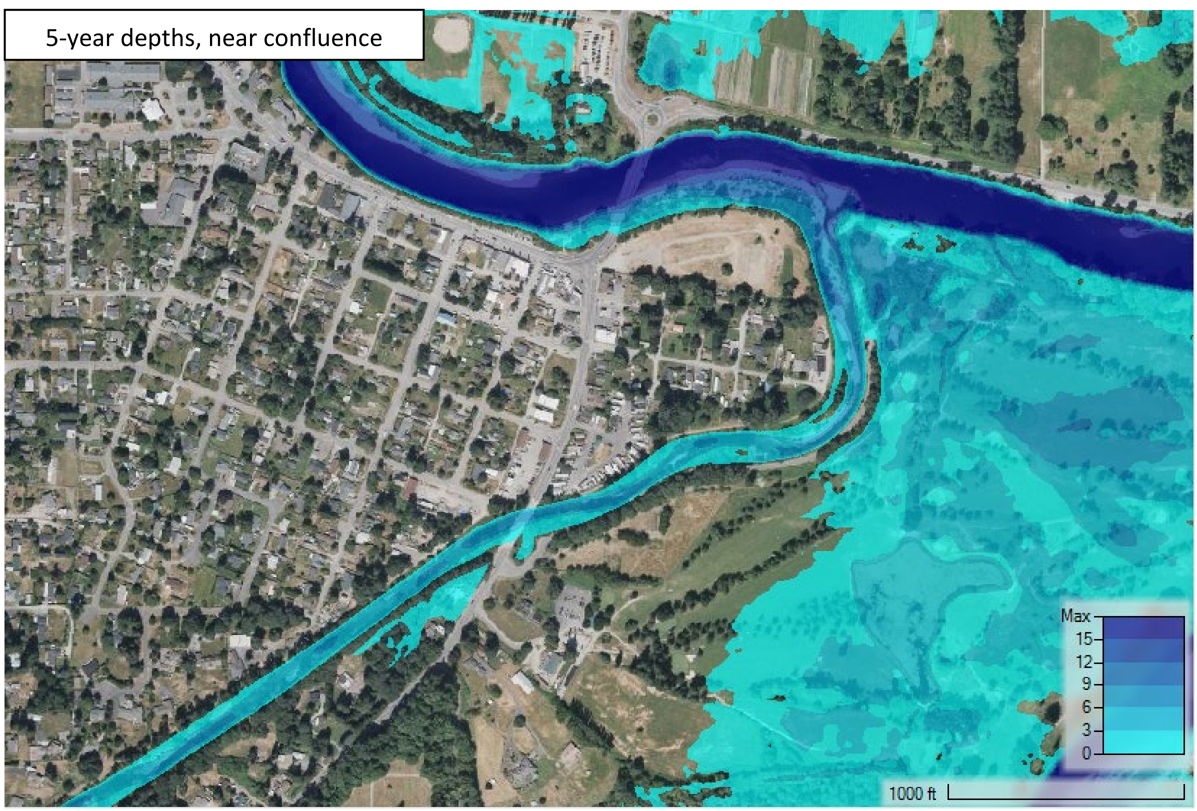


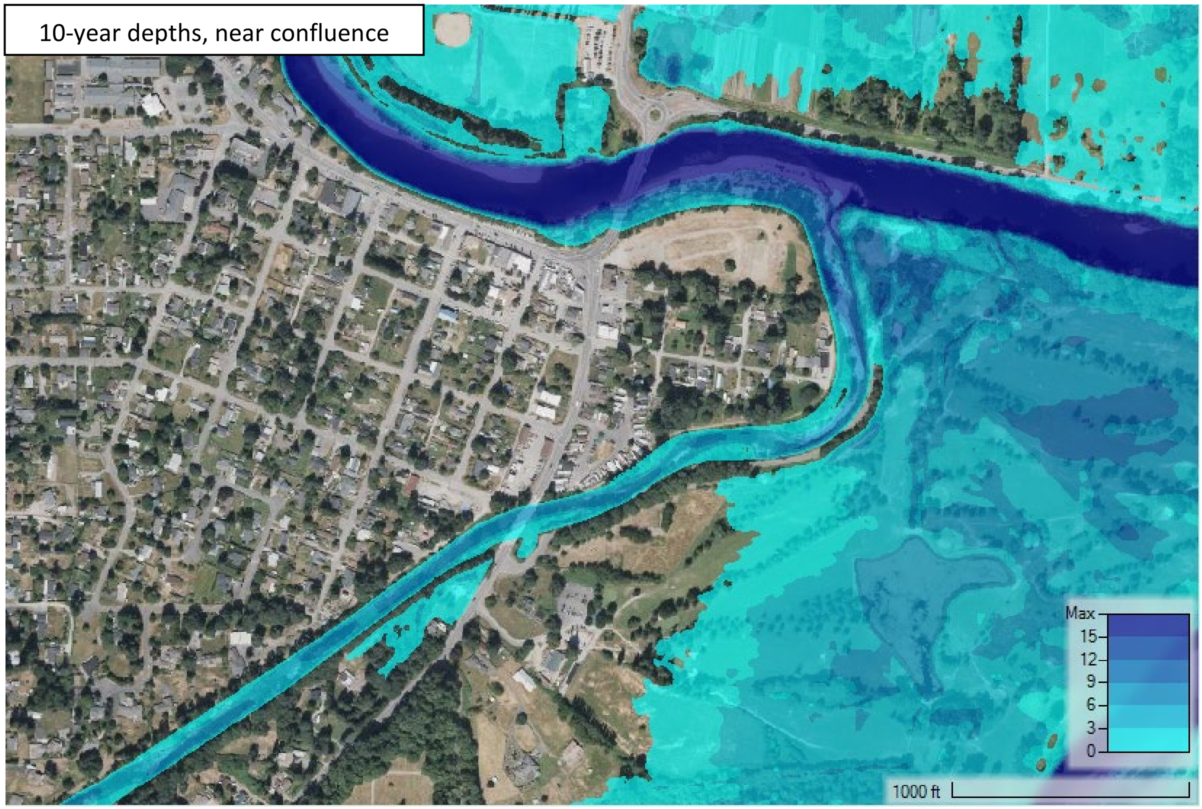
Black dots are projections from 10 separate models

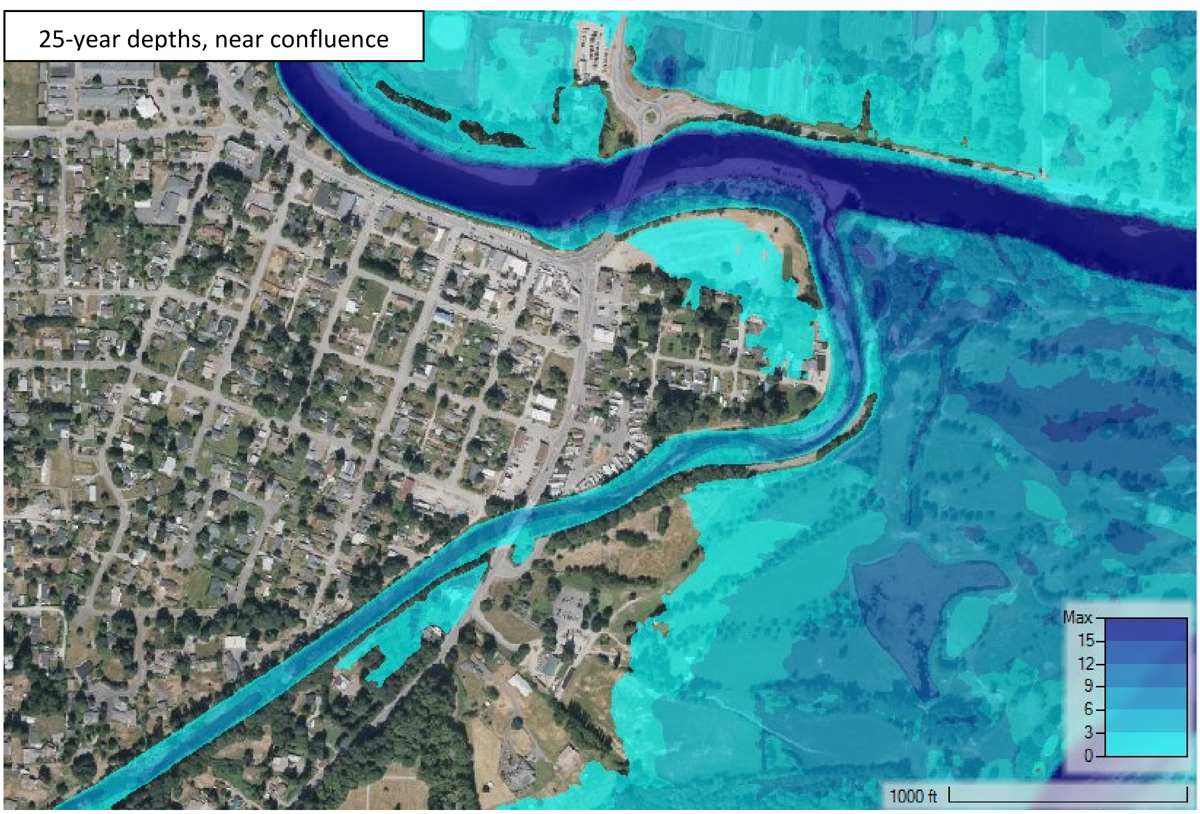
APPENDIX C

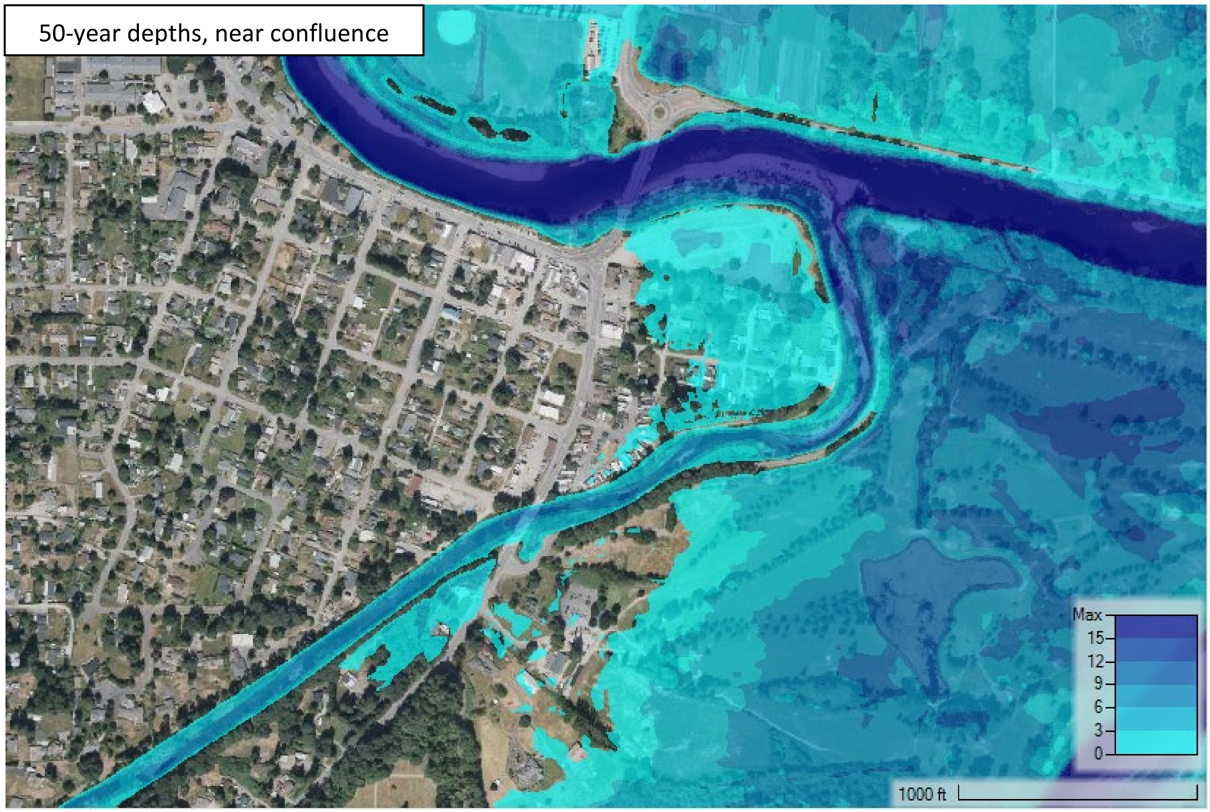
MODEL OUTPUT FIGURES

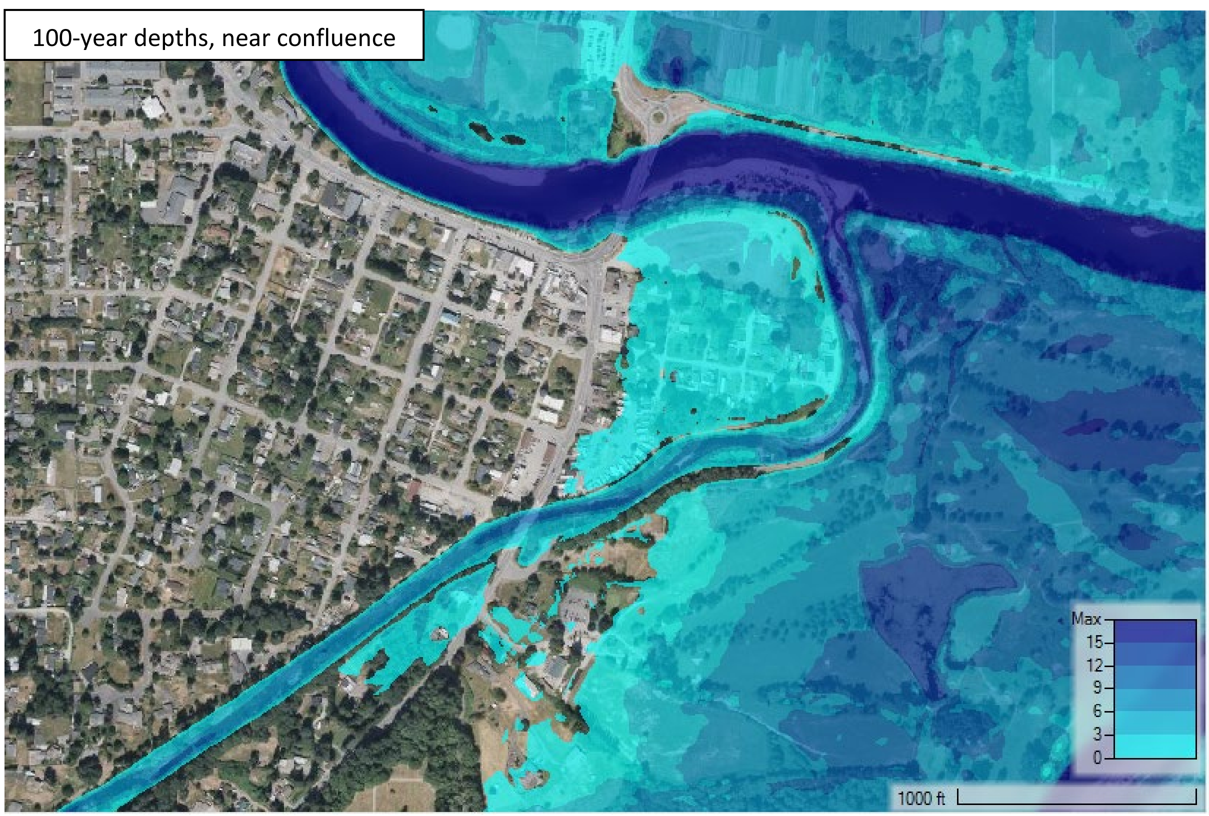

2-year depths, near confluence
Max
15
12
9
6
3
0
1000 ft

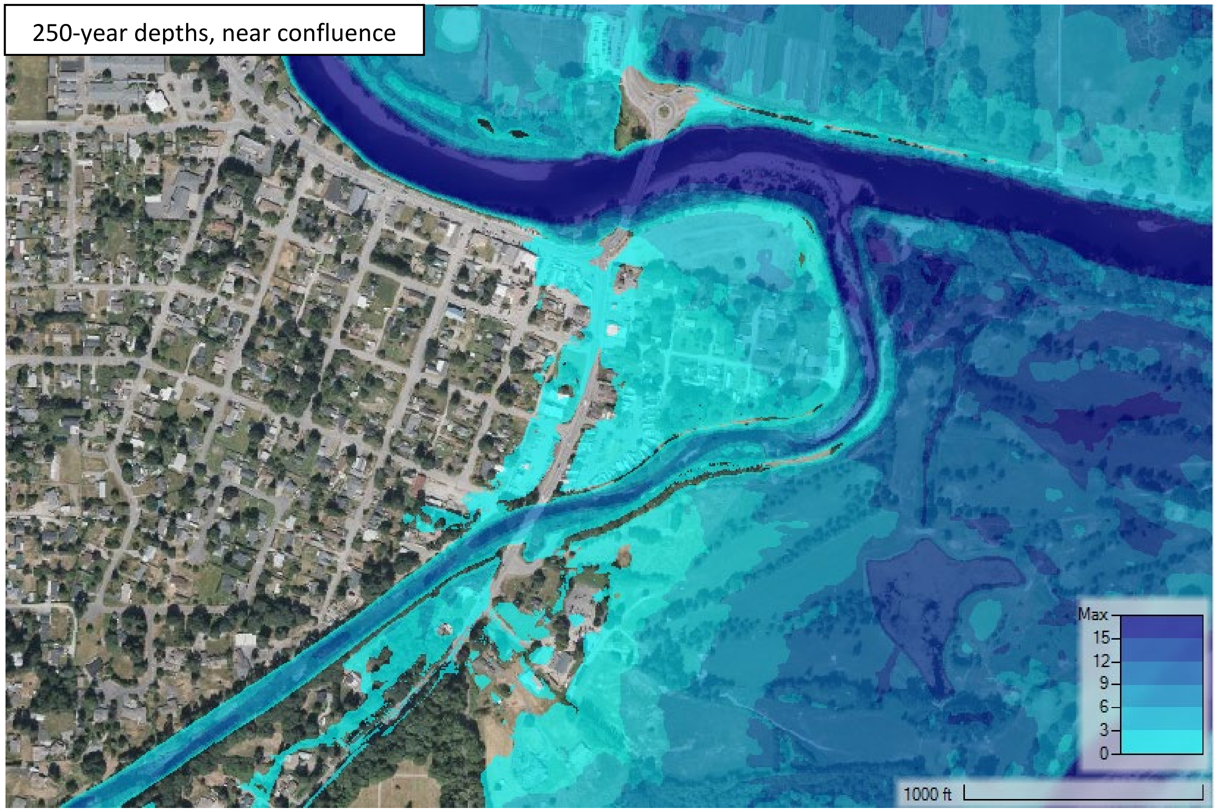

5-year depths, near confluence
Max
15
12
9
6
3
0
1000 ft

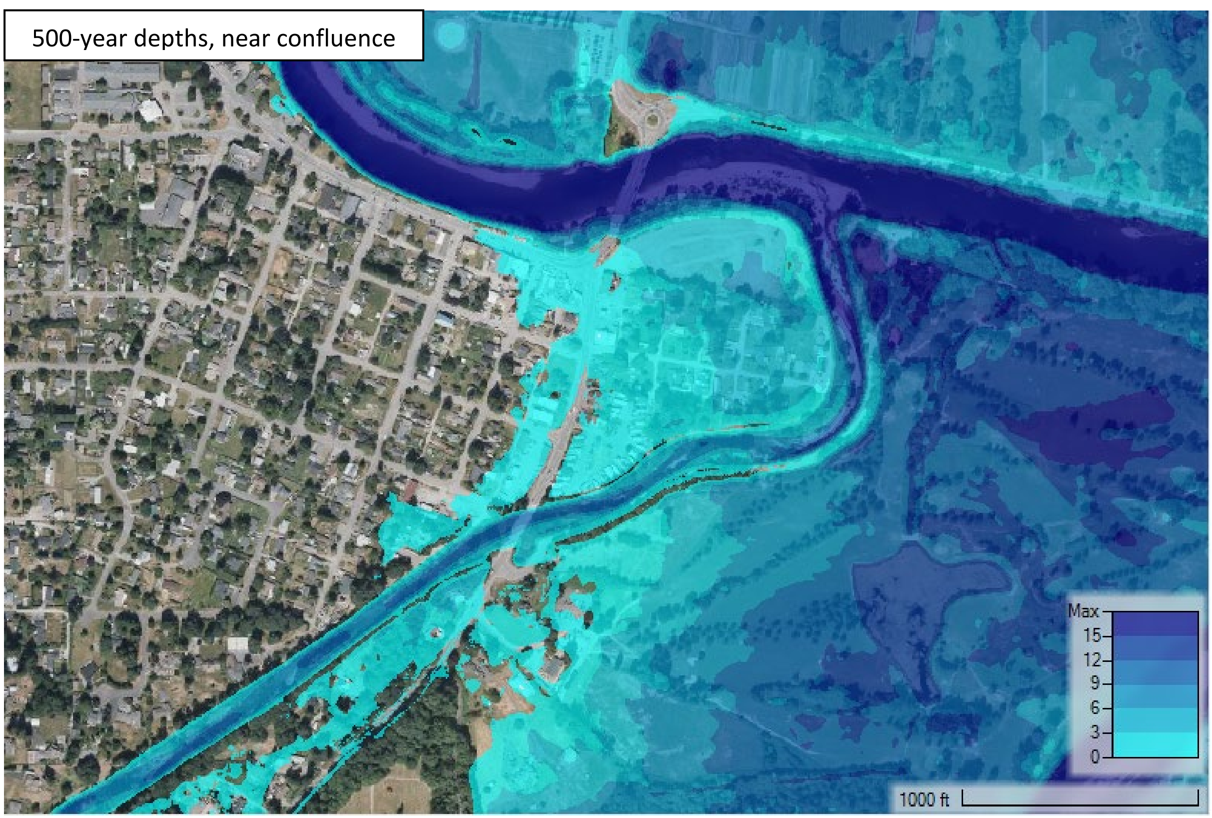

10-year depths, near confluence
Max
15
12
9
6
3
0
1000 ft

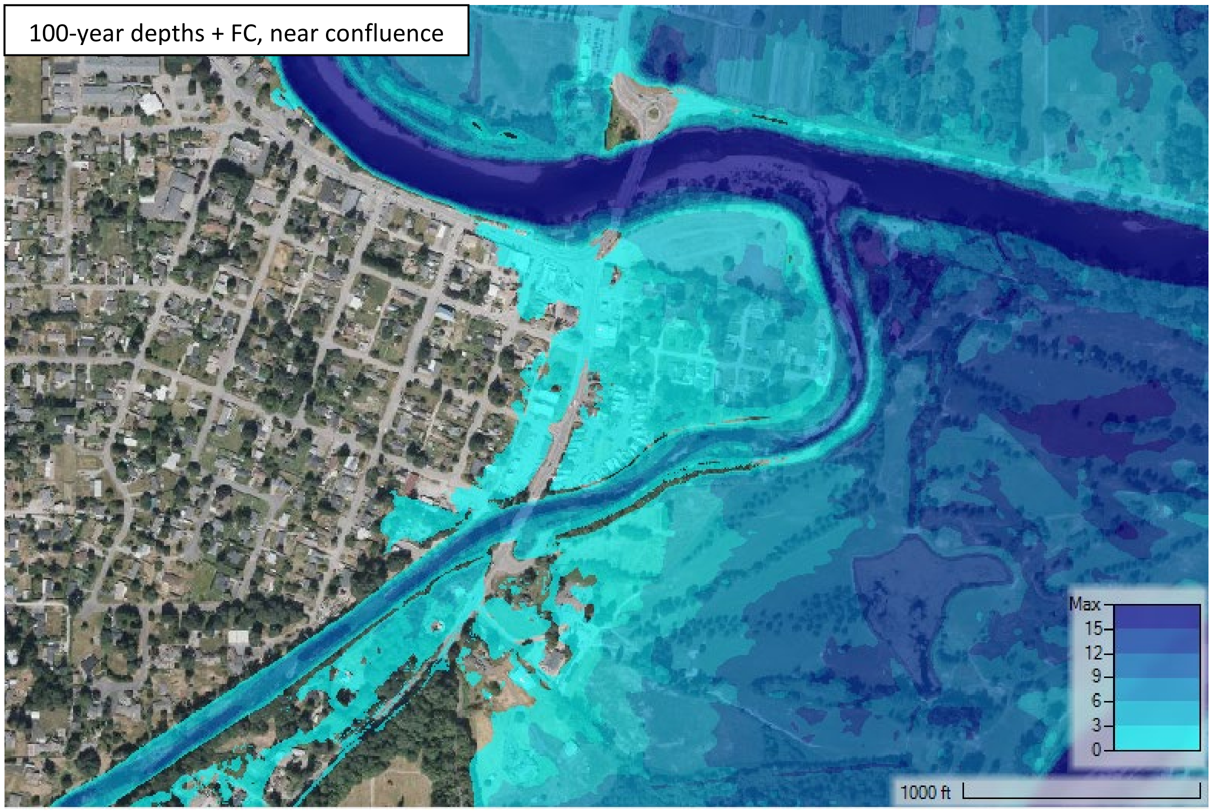

25-year depths, near confluence
Max
15
12
9
6
3
0
1000 ft

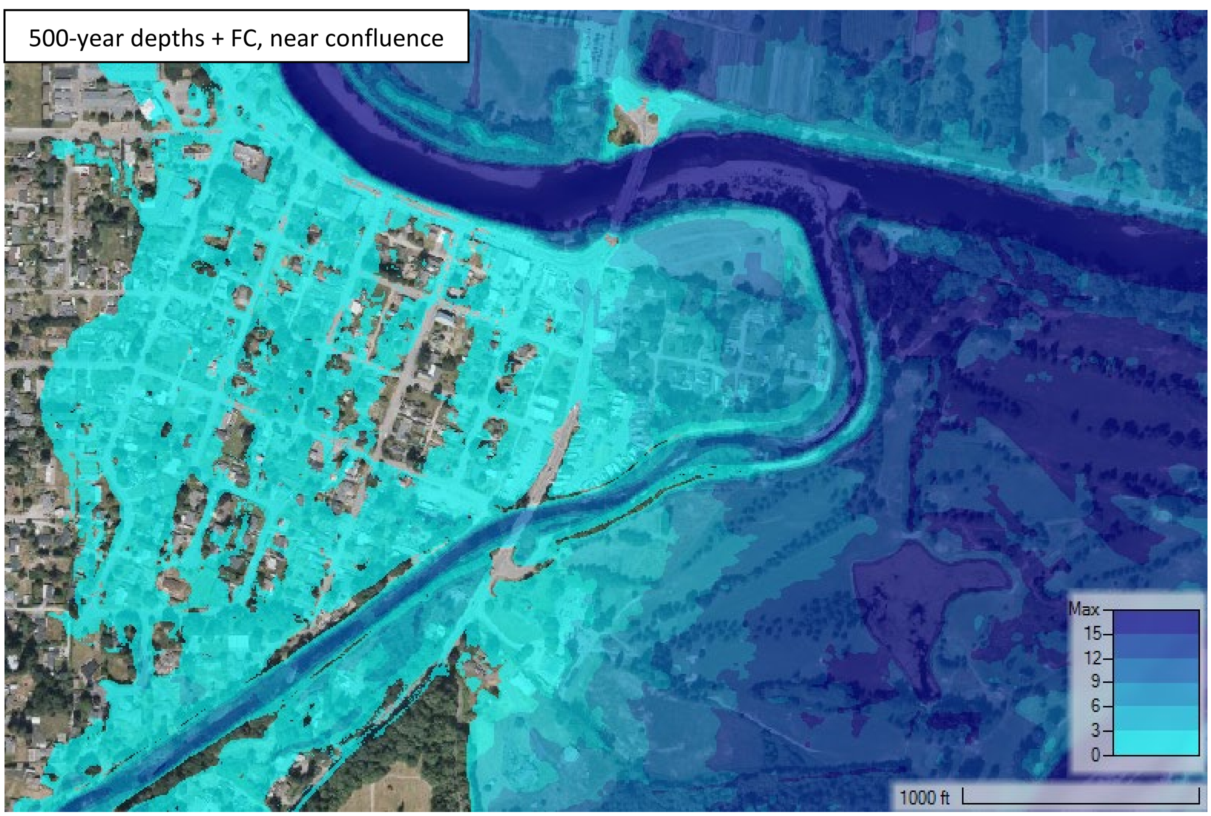

50-year depths, near confluence
Max
15
12
9
6
3
0
1000 ft

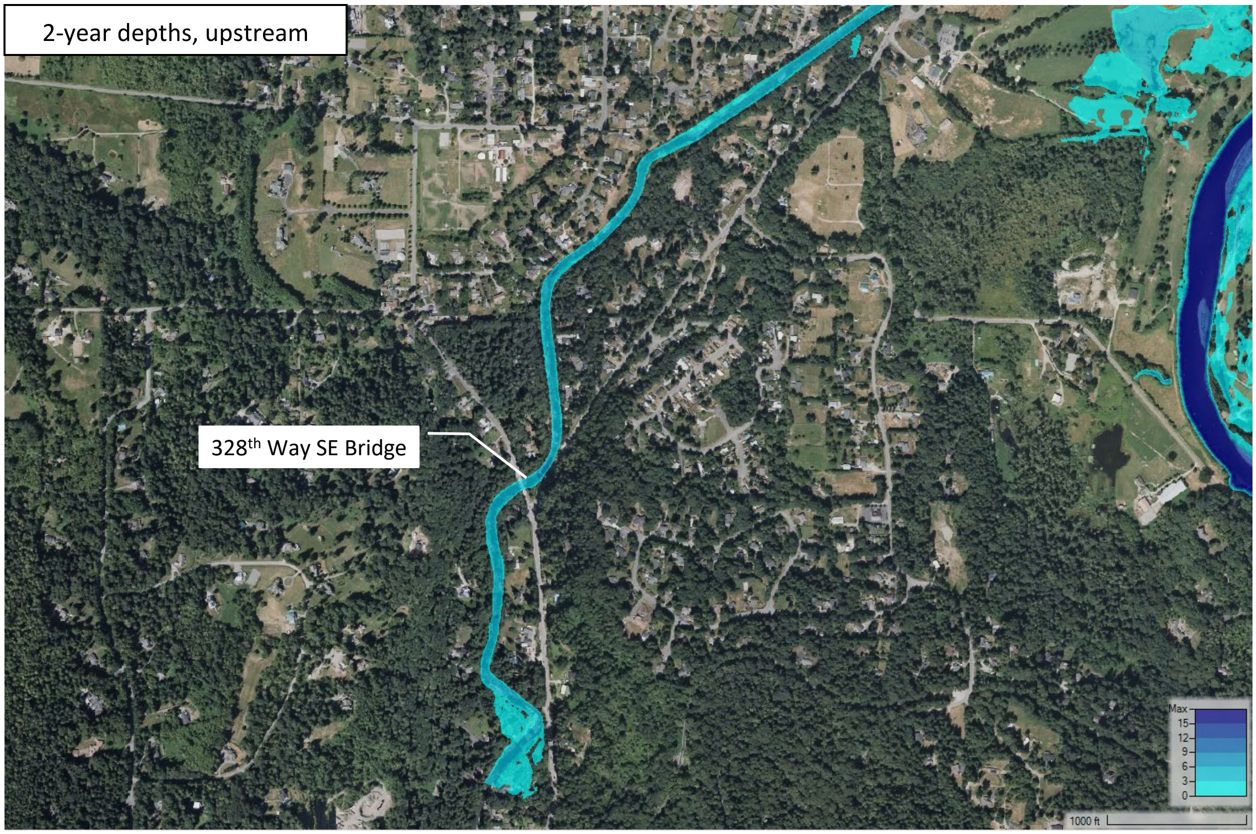

100-year depths, near confluence
Max
15
12
9
6
3
0
1000 ft

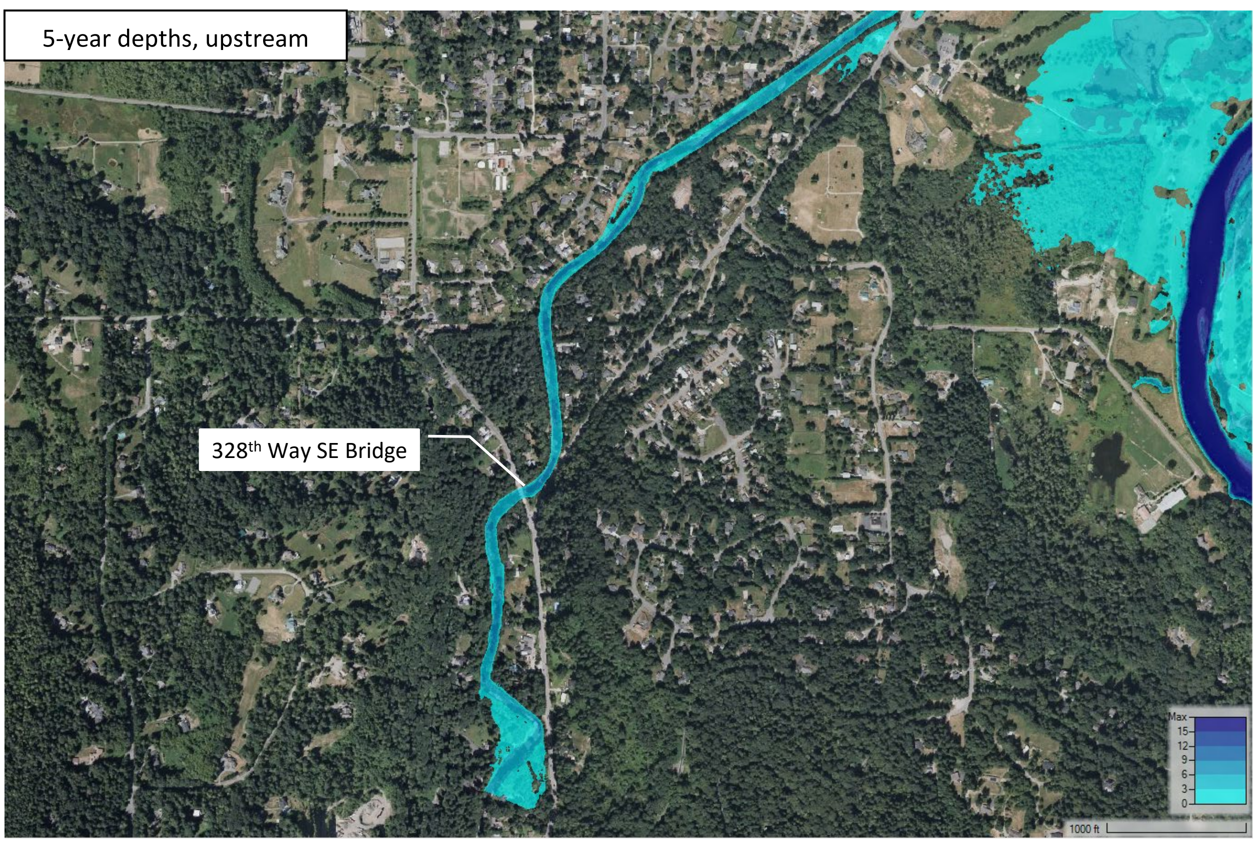

250-year depths, near confluence
Max
15
12
9
6
3
0
1000 ft

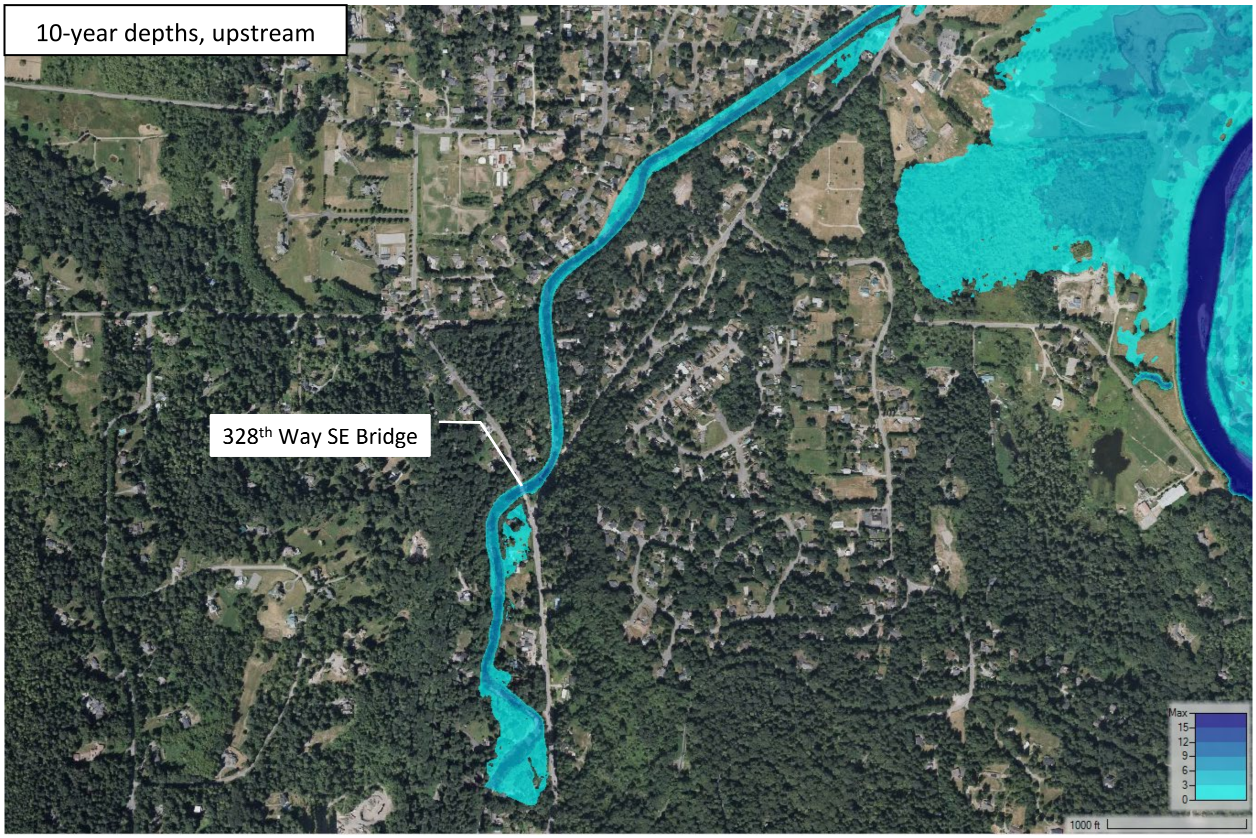

500-year depths, near confluence
Max
15
12
9
6
3
0
1000 ft

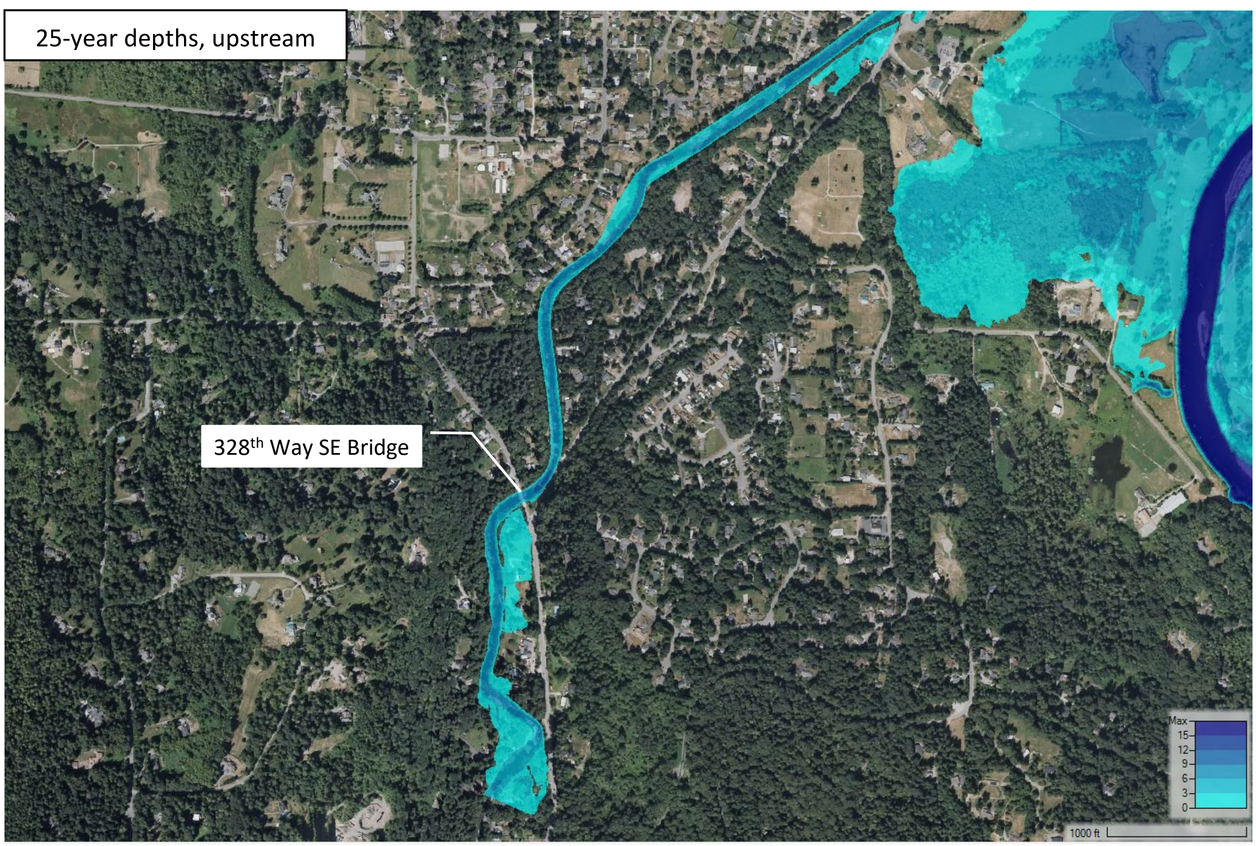

100-year depths + FC, near confluence
Max
15
12
9
6
3
0
1000 ft

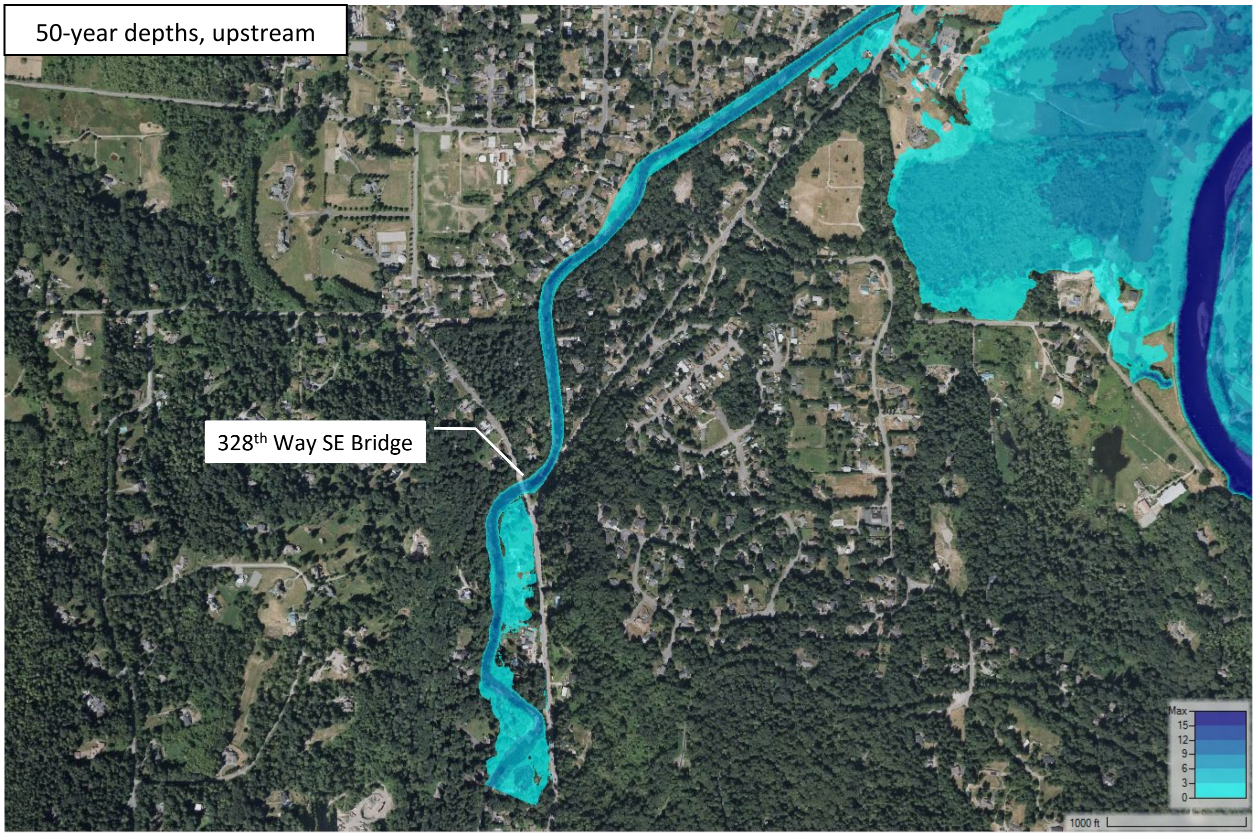

500-year depths + FC, near confluence
Max
15
12
9
6
3
0
1000 ft


2-year depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

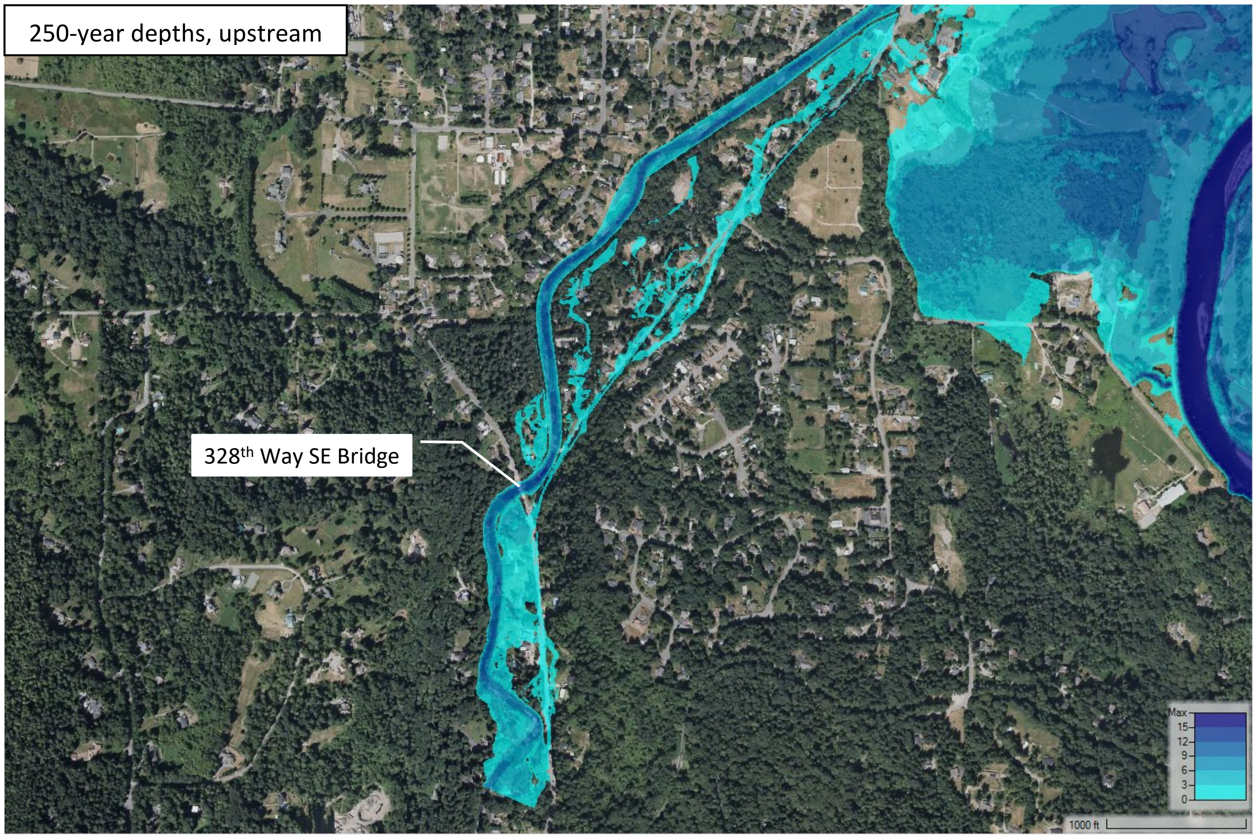

5-year depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

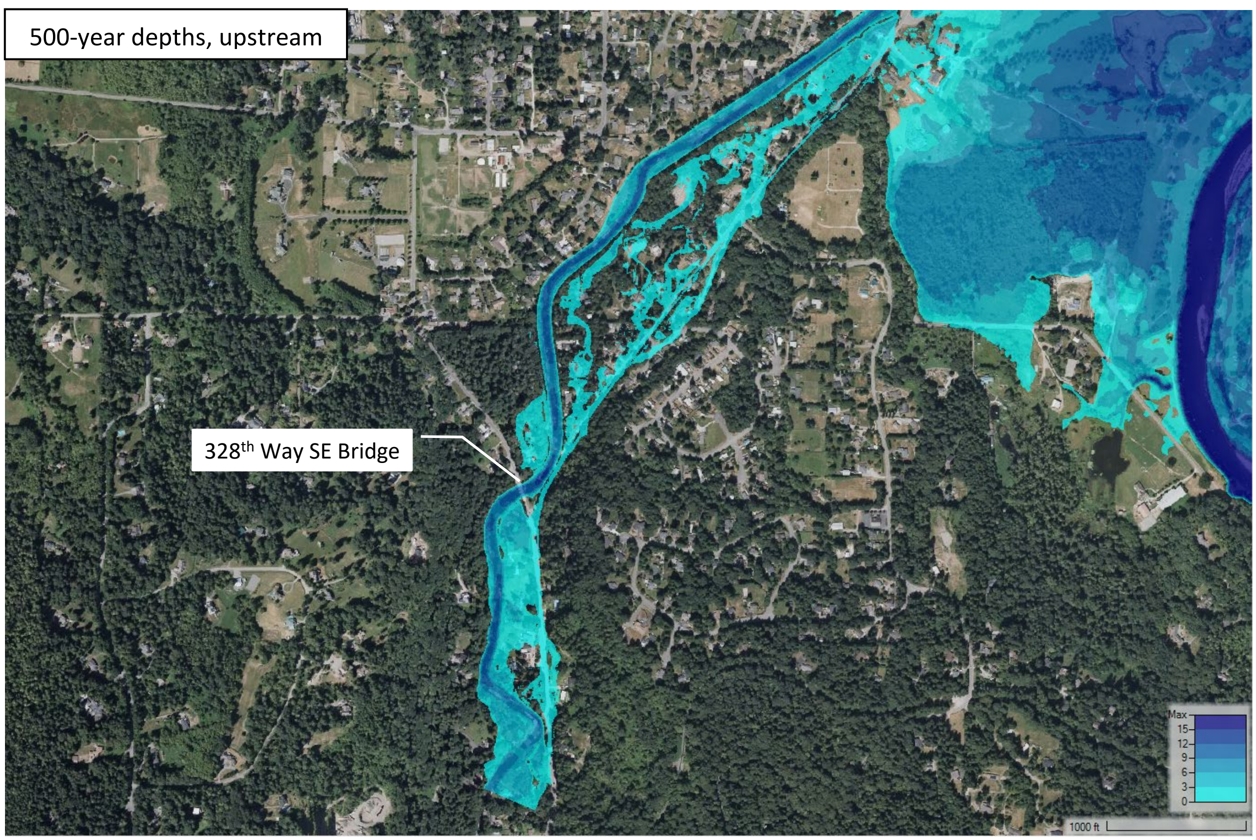

10-year depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

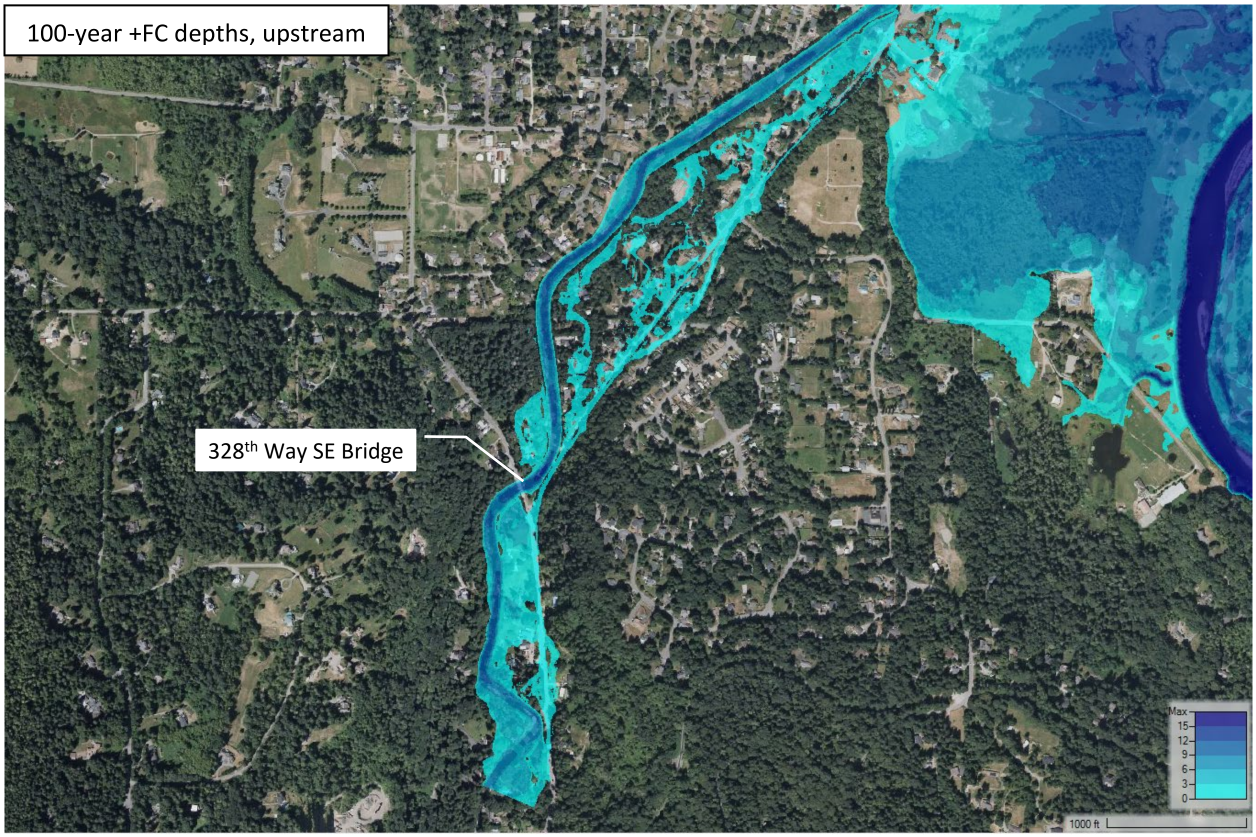

25-year depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

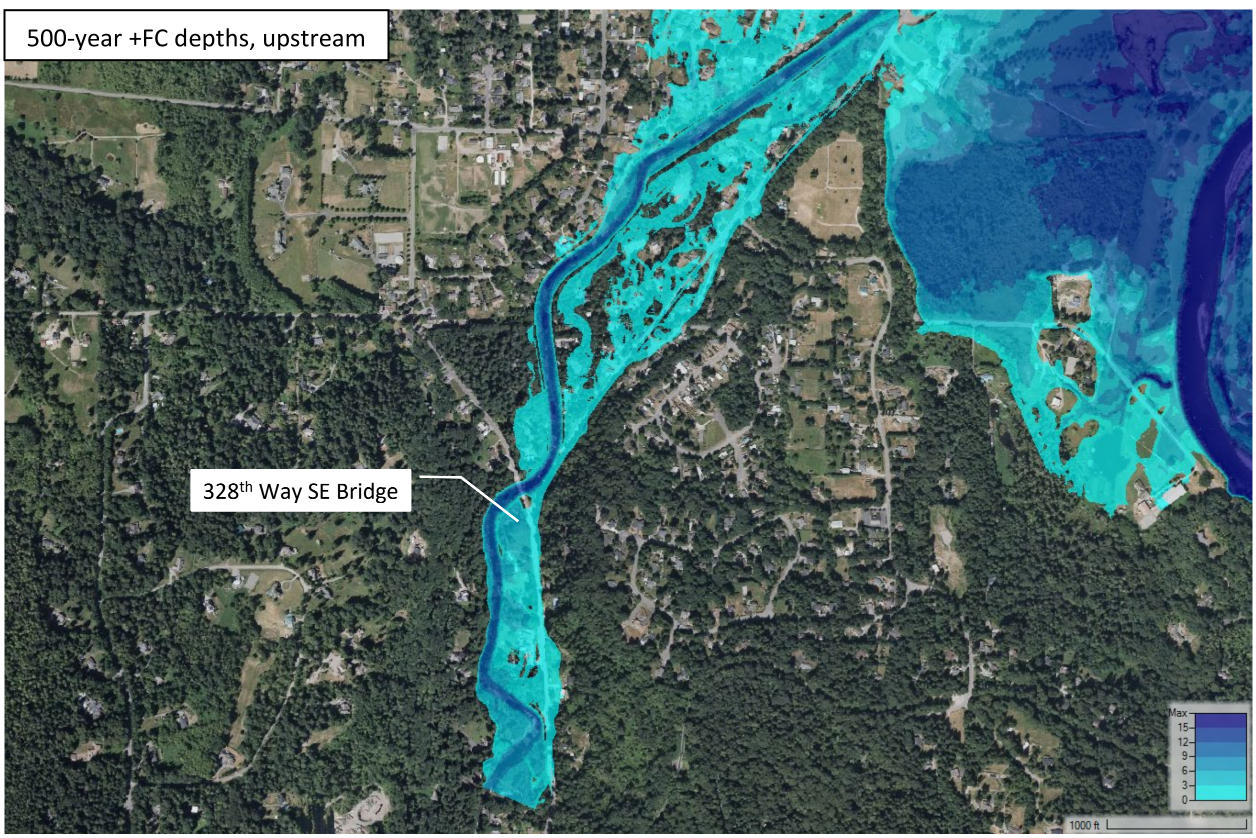

50-year depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

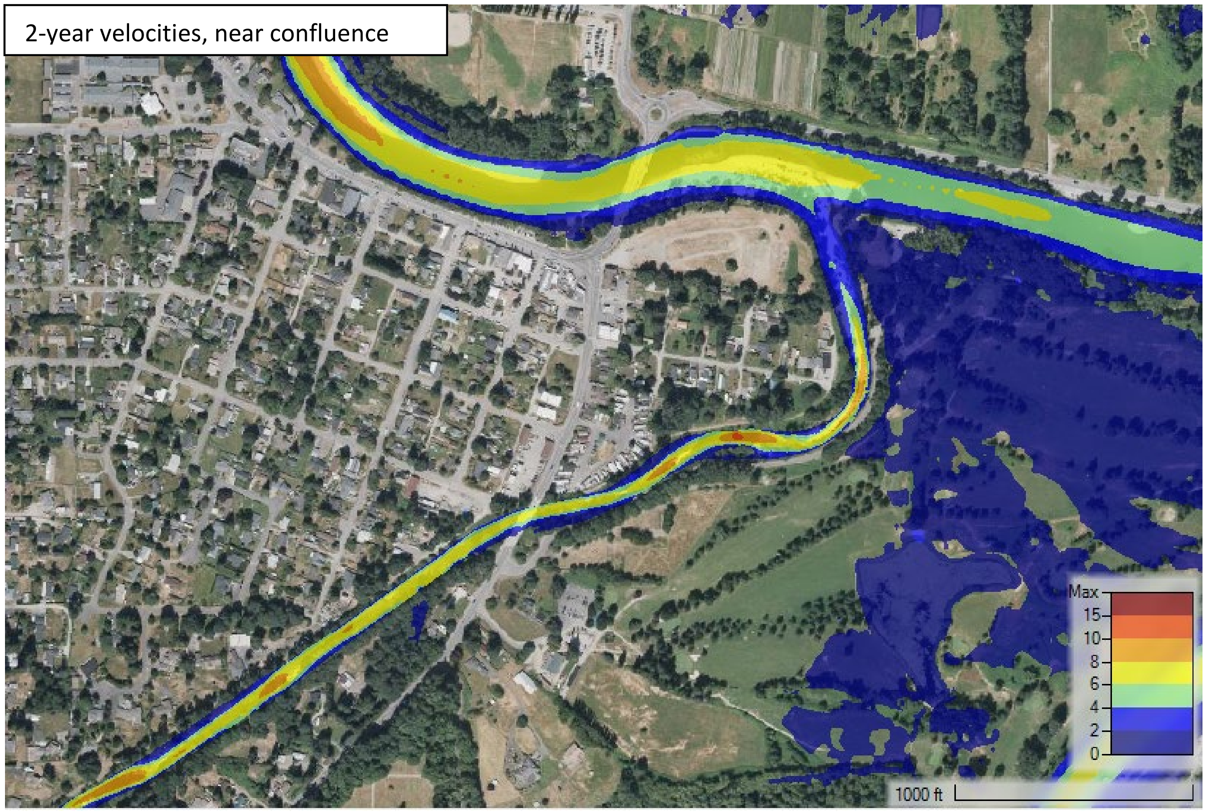

100-year depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

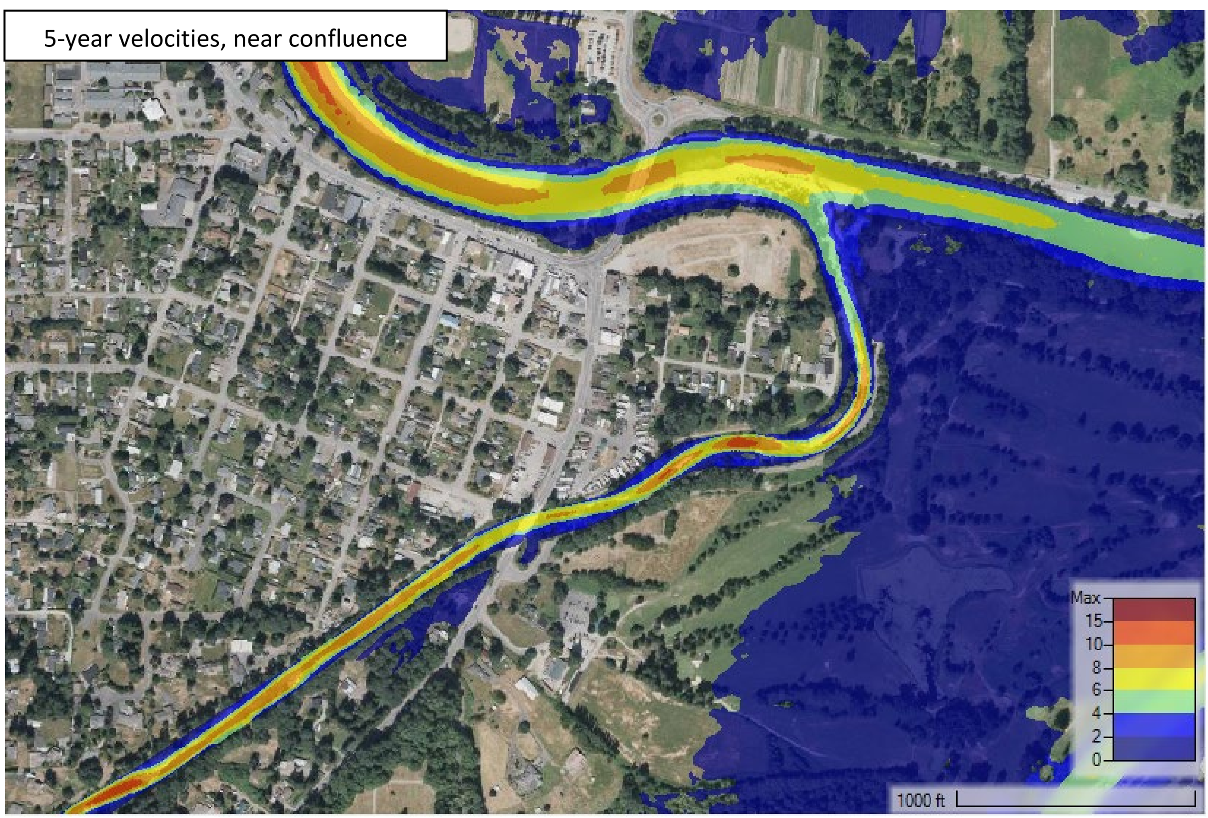

250-year depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

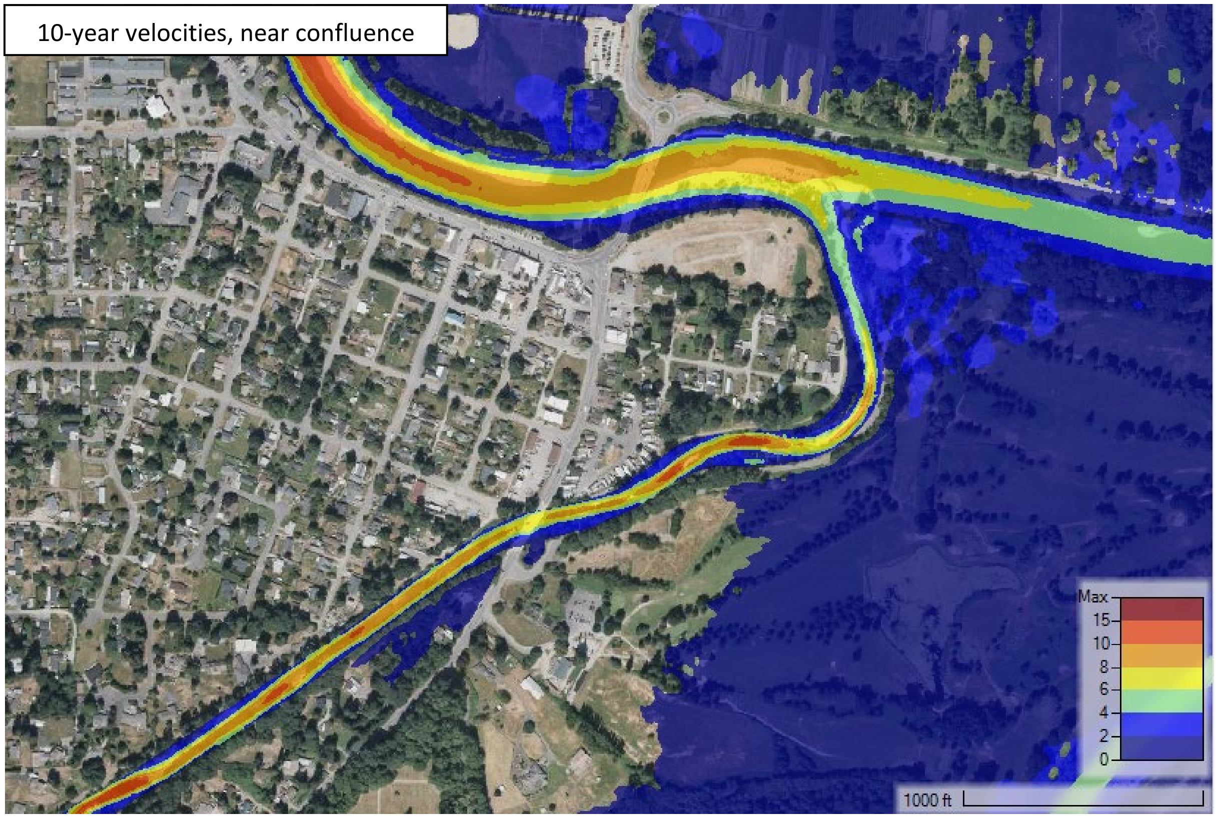

500-year depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

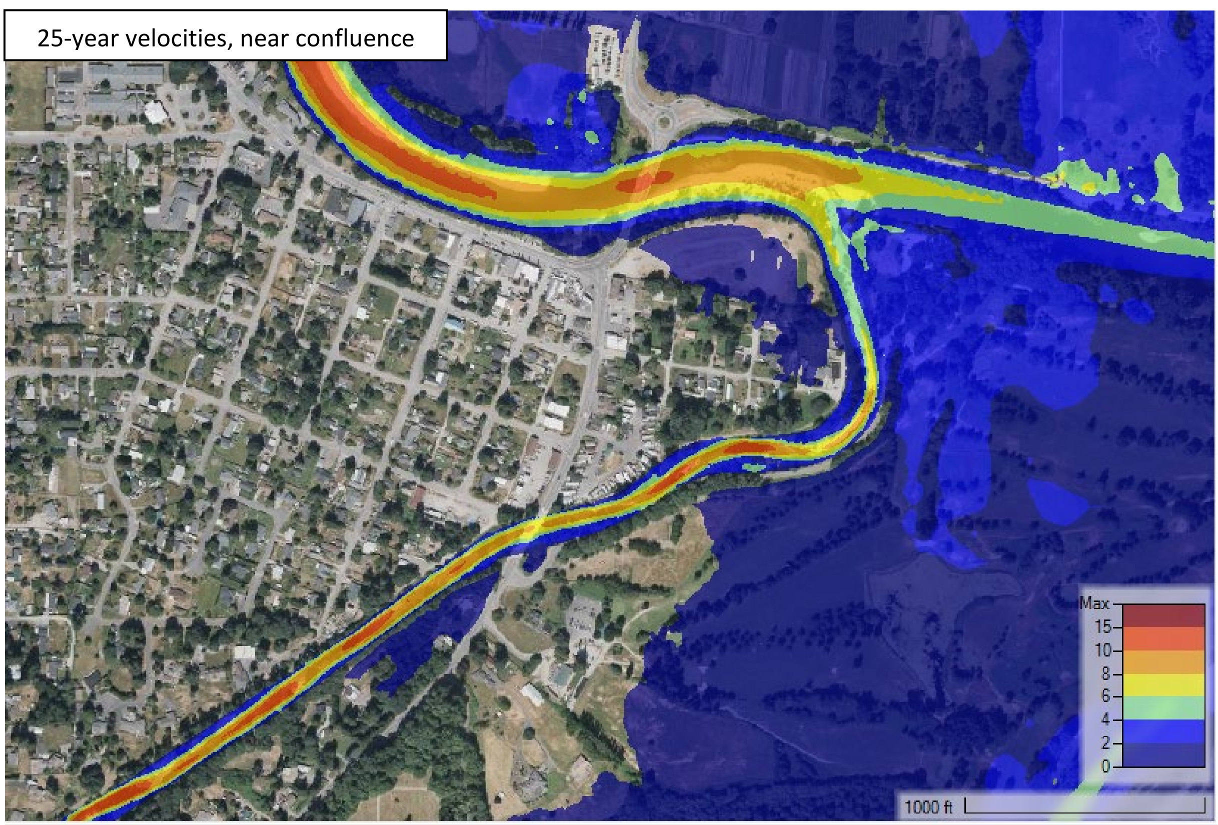

100-year +FC depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

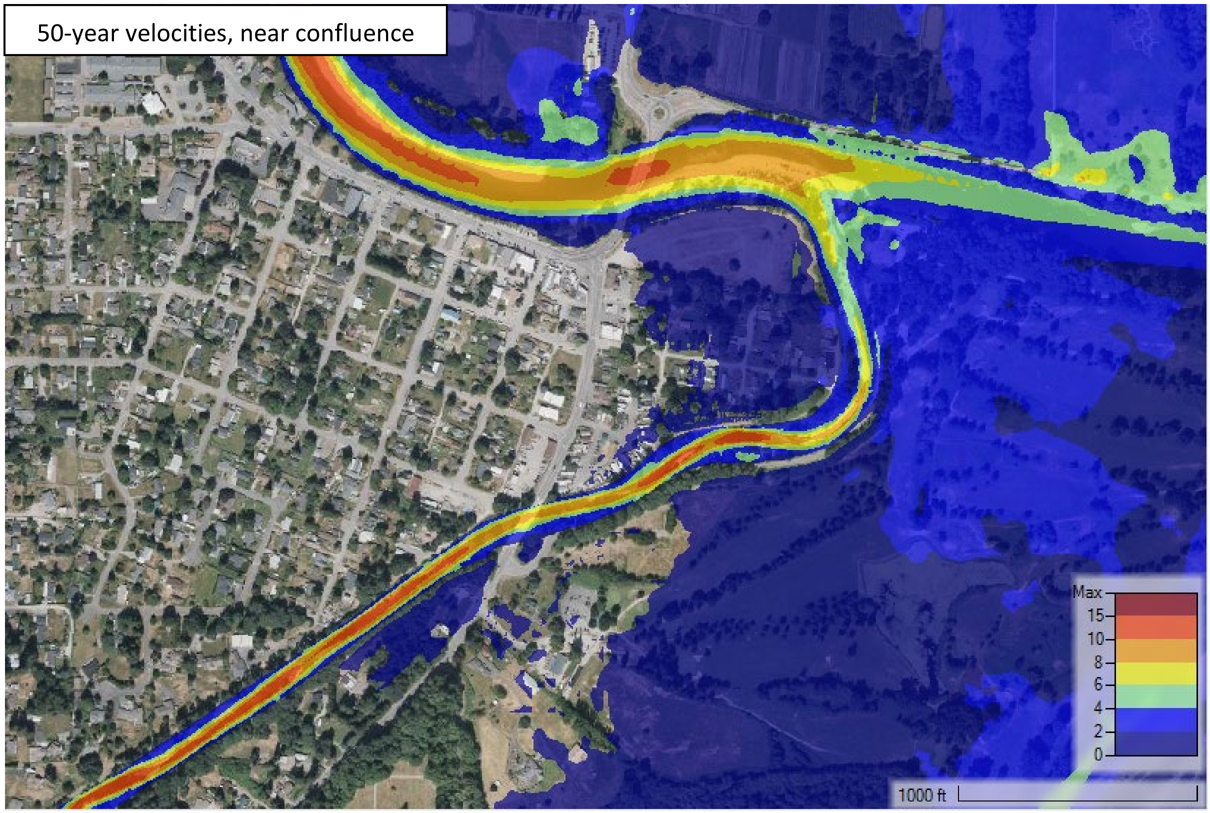

500-year +FC depths, upstream
328th Way SE Bridge
Max
15
12
9
6
3
0
1000 ft

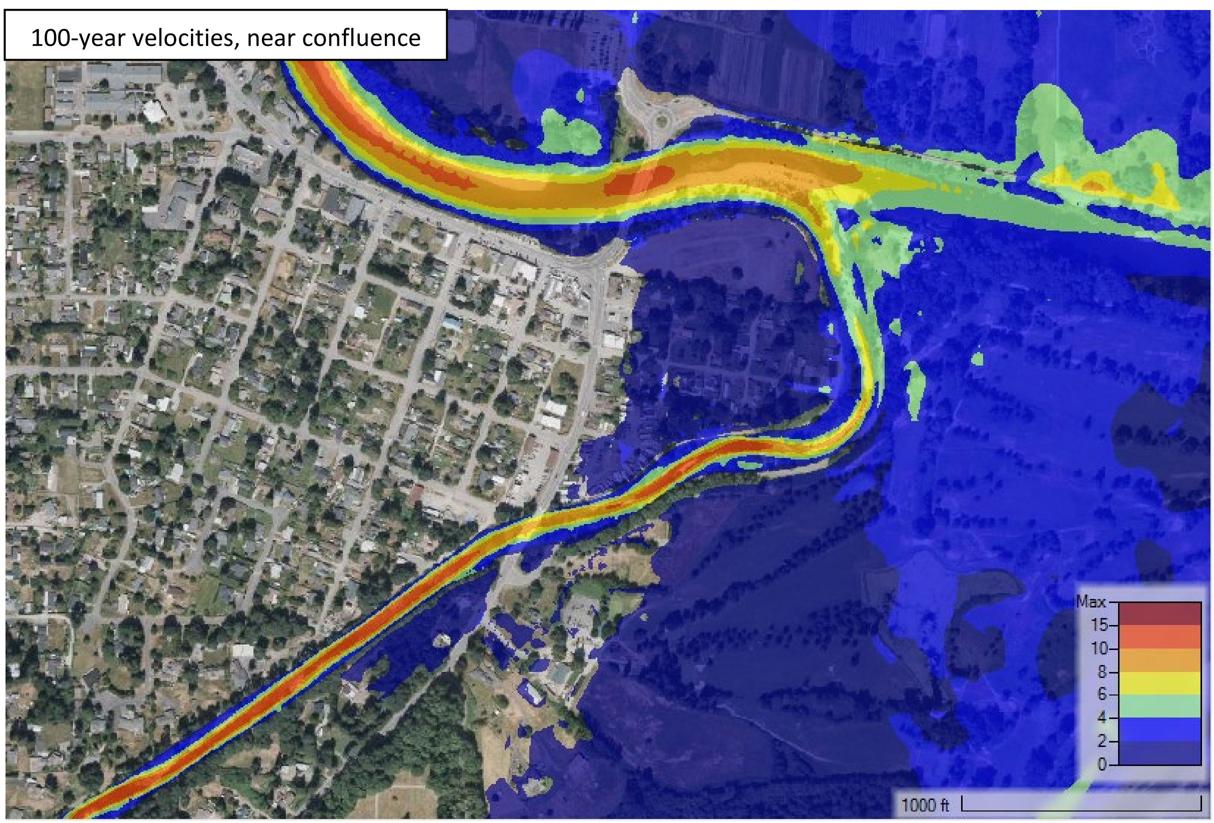

2-year velocities, near confluence
Max
15
10
8
6
4
2
0
1000 ft

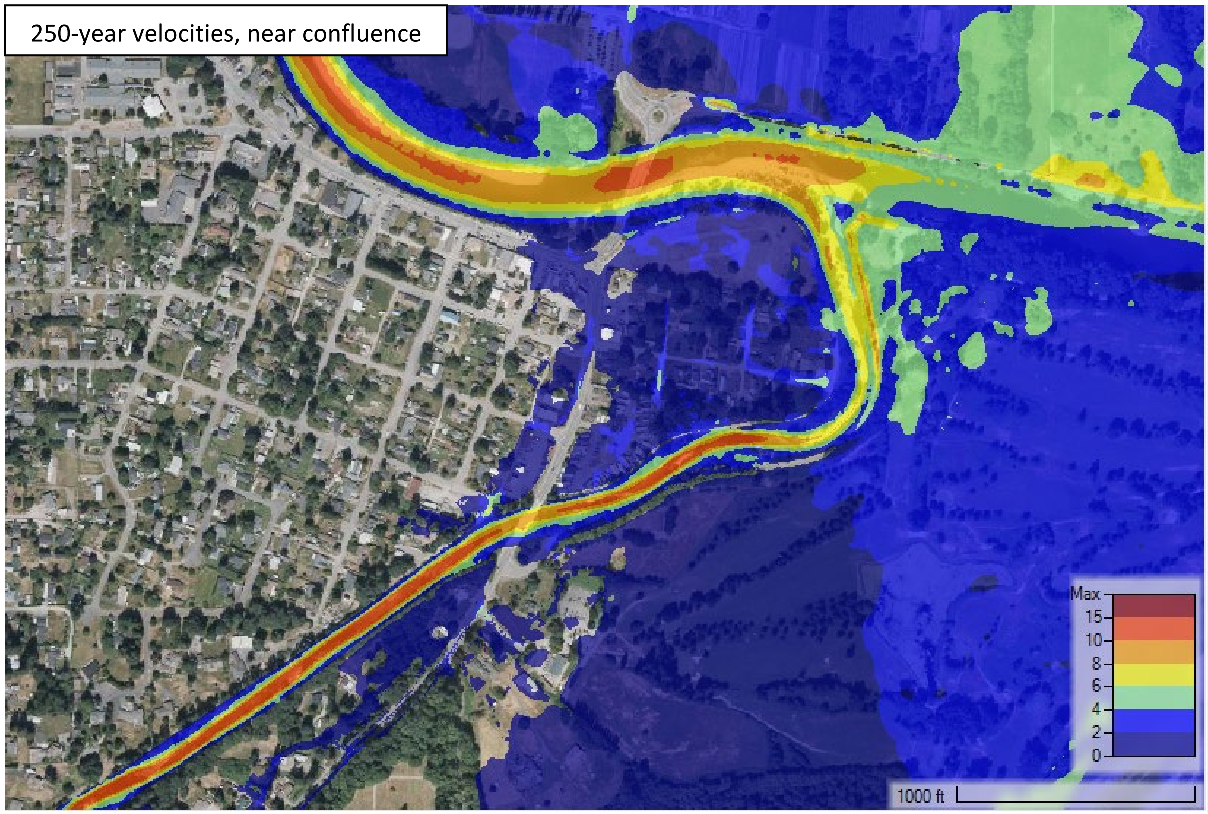

5-year velocities, near confluence
Max
15
10
8
6
4
2
0
1000 ft

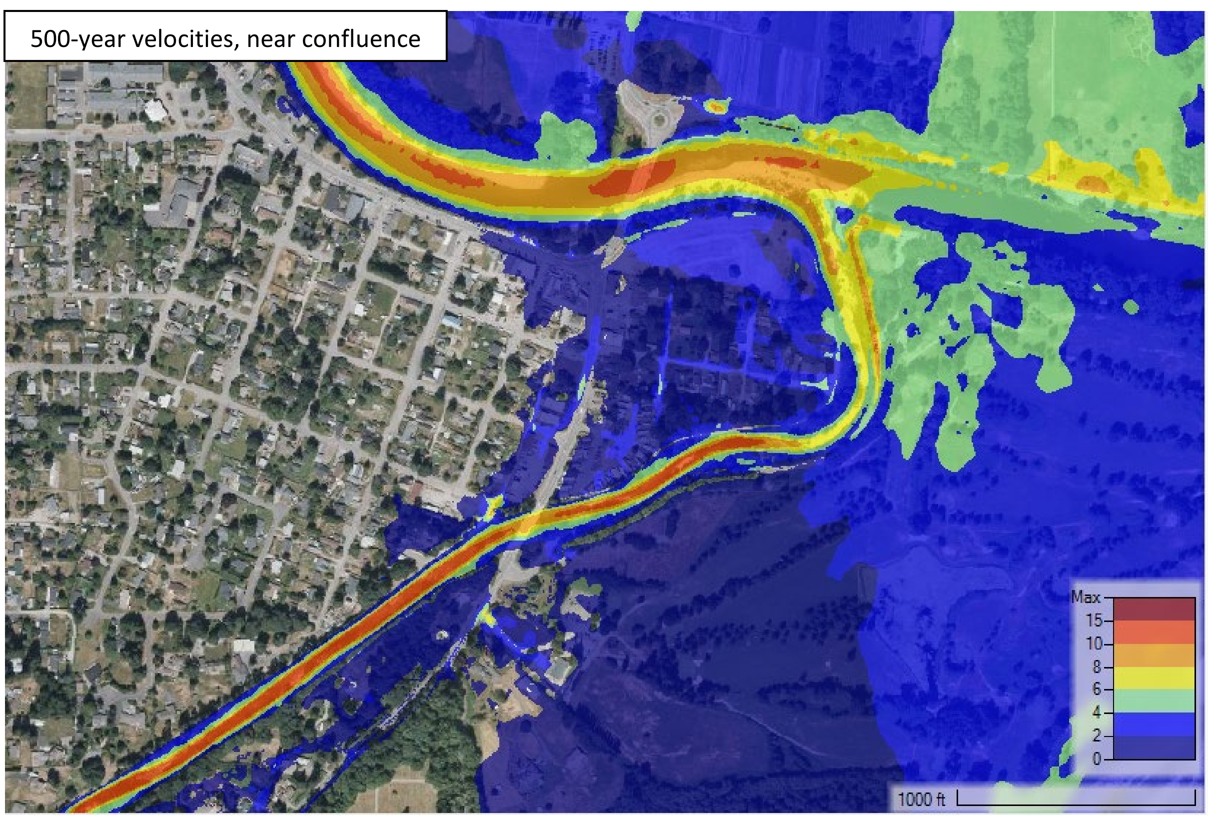

10-year velocities, near confluence
Max
15
10
8
6
4
2
0
1000 ft


25-year velocities, near confluence
Max
15
10
8
6
4
2
0
1000 ft

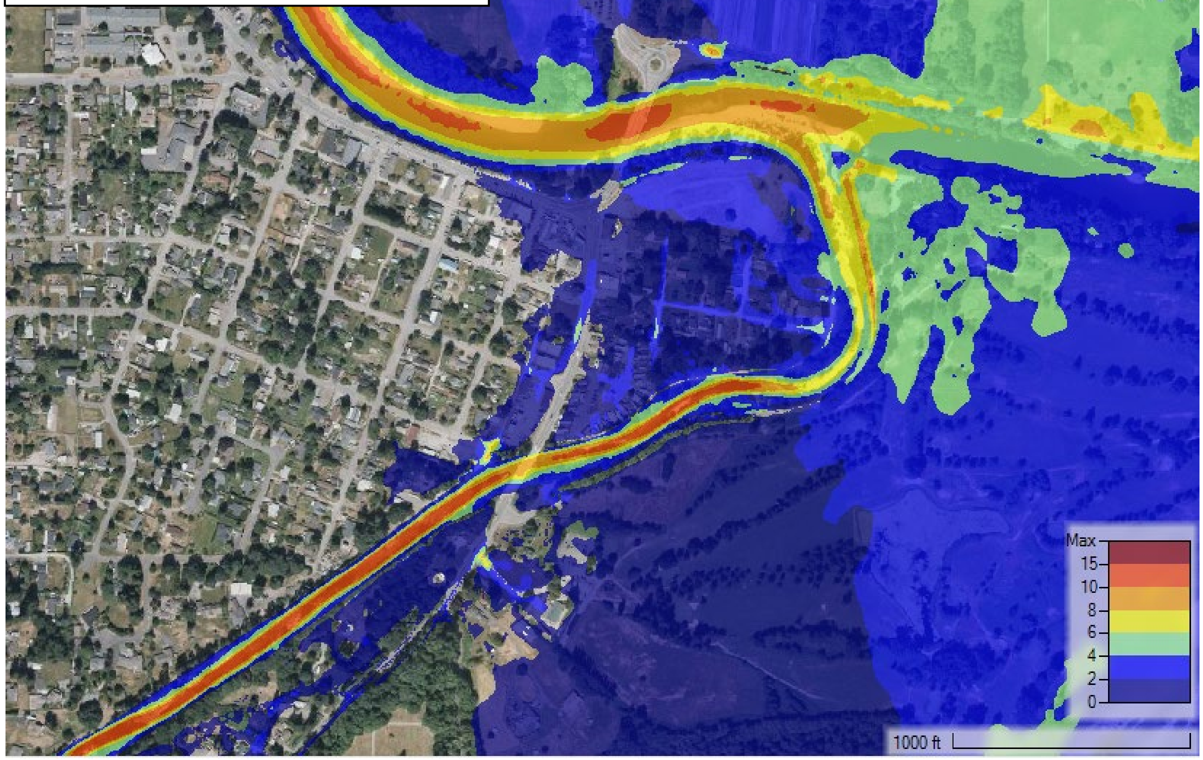

50-year velocities, near confluence
Max
15
10
8
6
4
2
0
1000 ft


100-year velocities, near confluence
Max
15
10
8
6
4
2
0
1000 ft

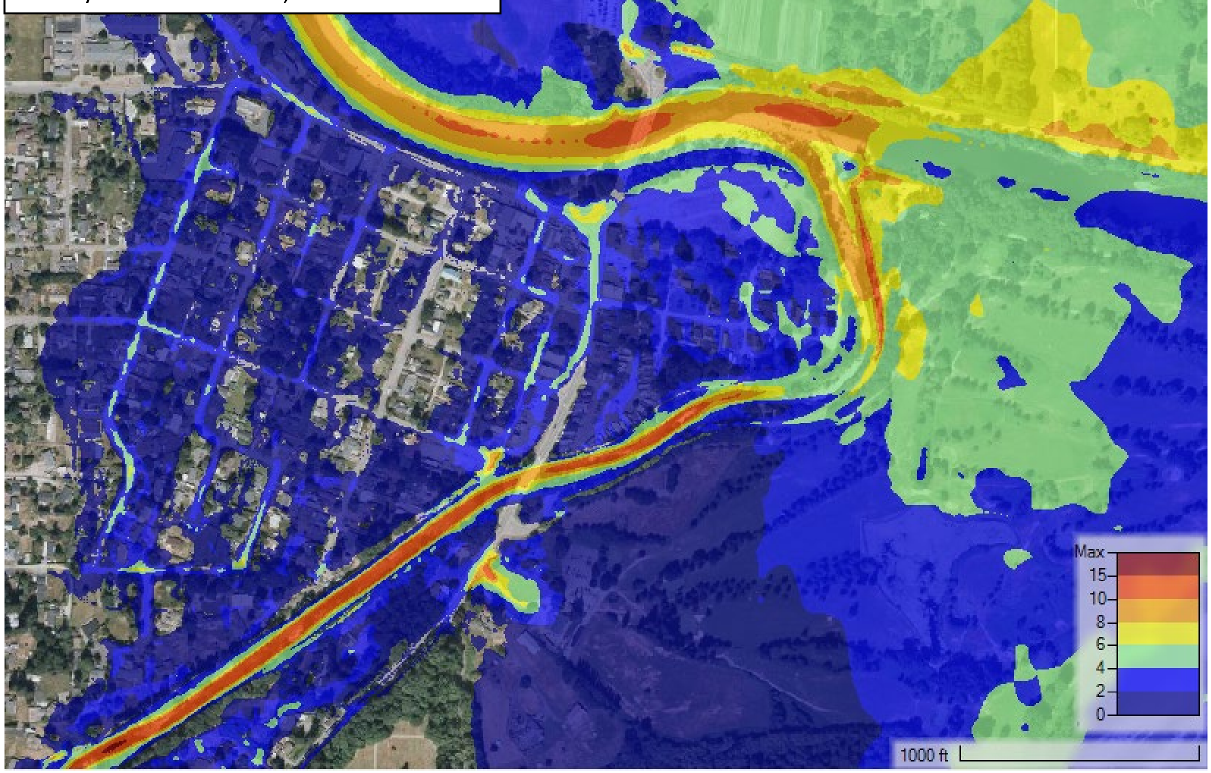

250-year velocities, near confluence
Max
15
10
8
6
4
2
0
1000 ft

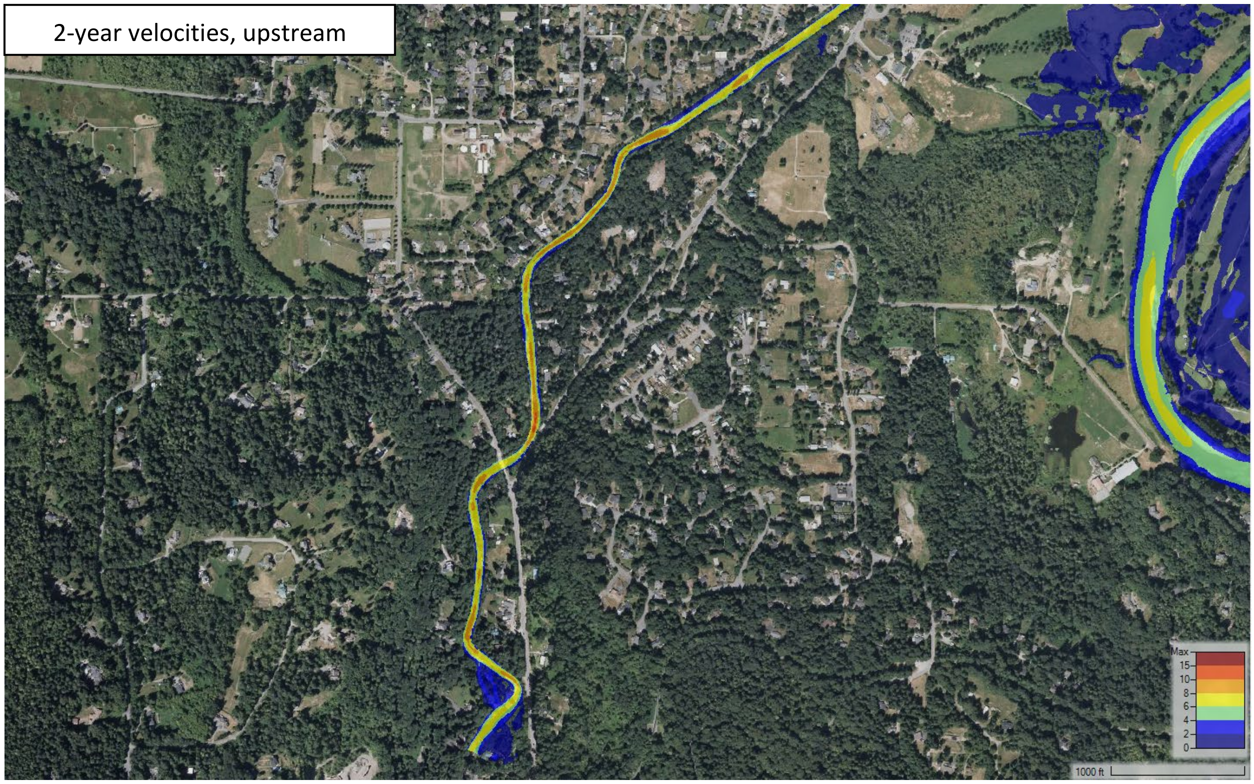

500-year velocities, near confluence
Max
15
10
8
6
4
2
0
1000 ft

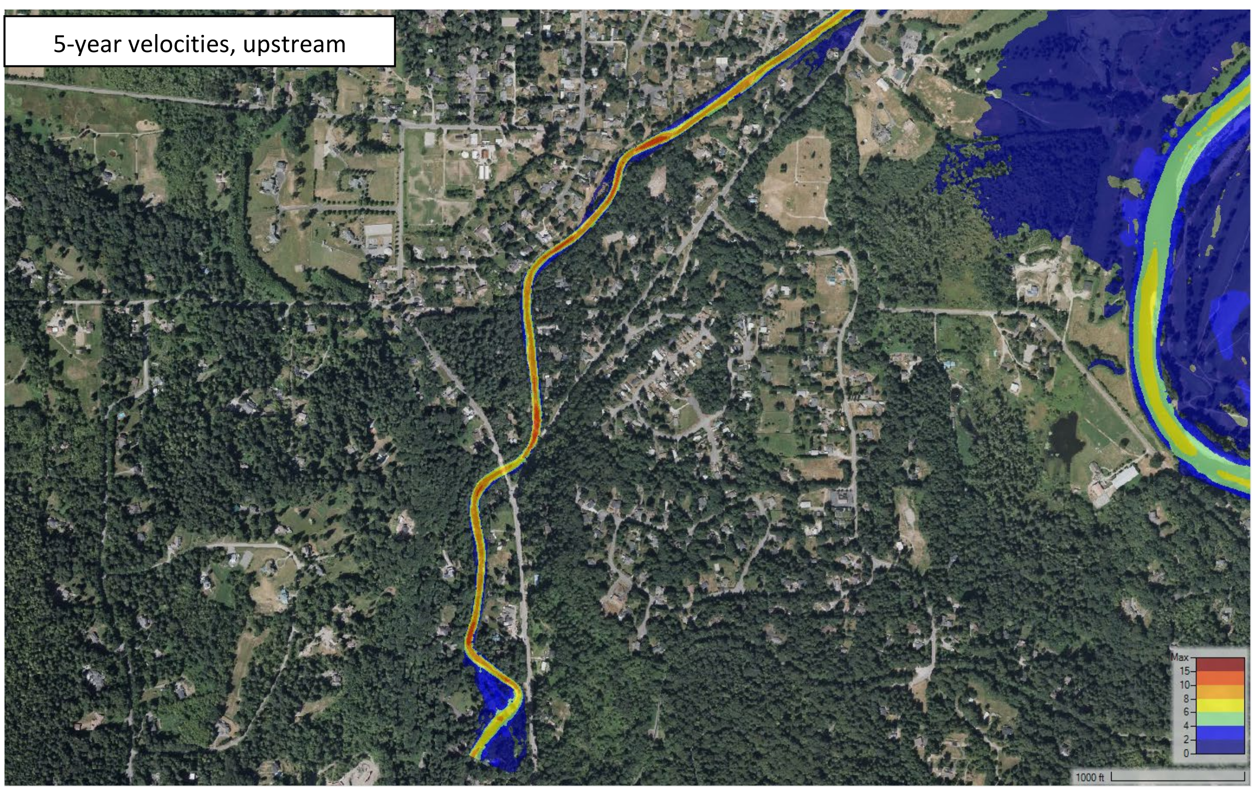
100-year +FC velocities, near confluence

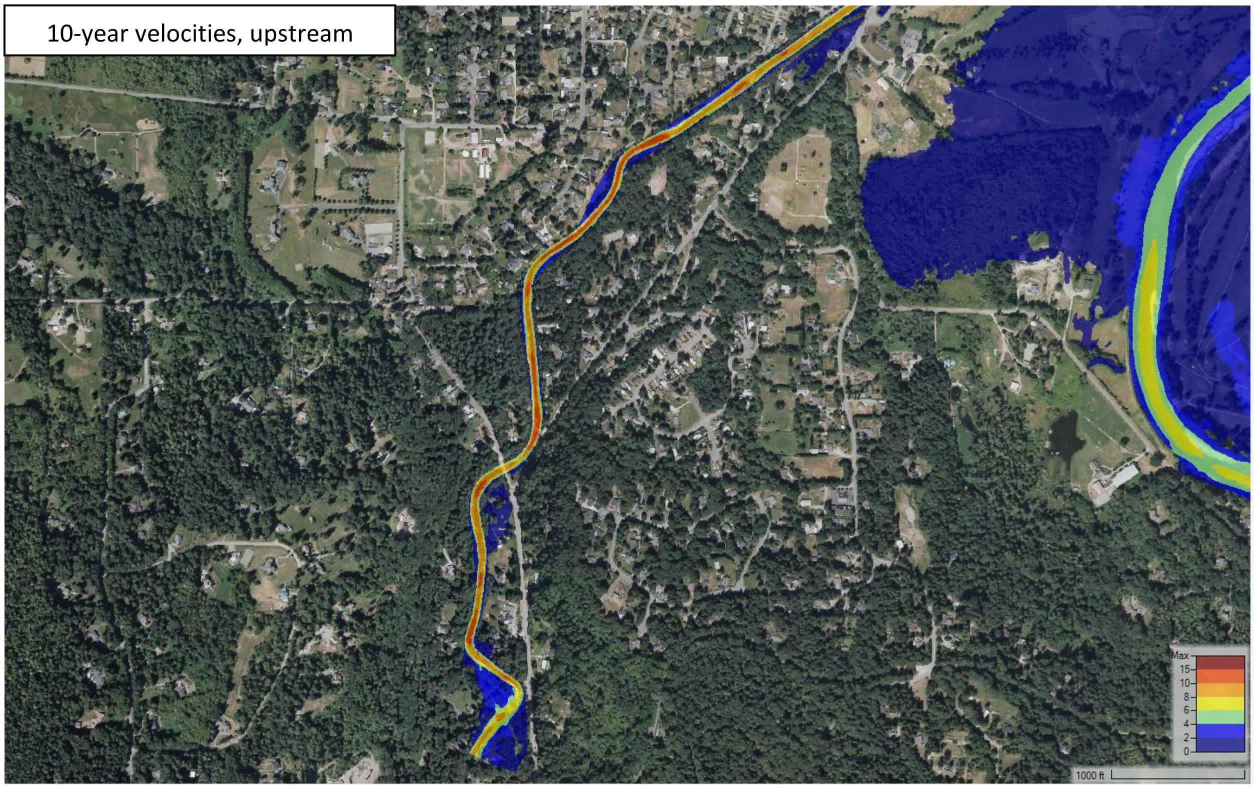


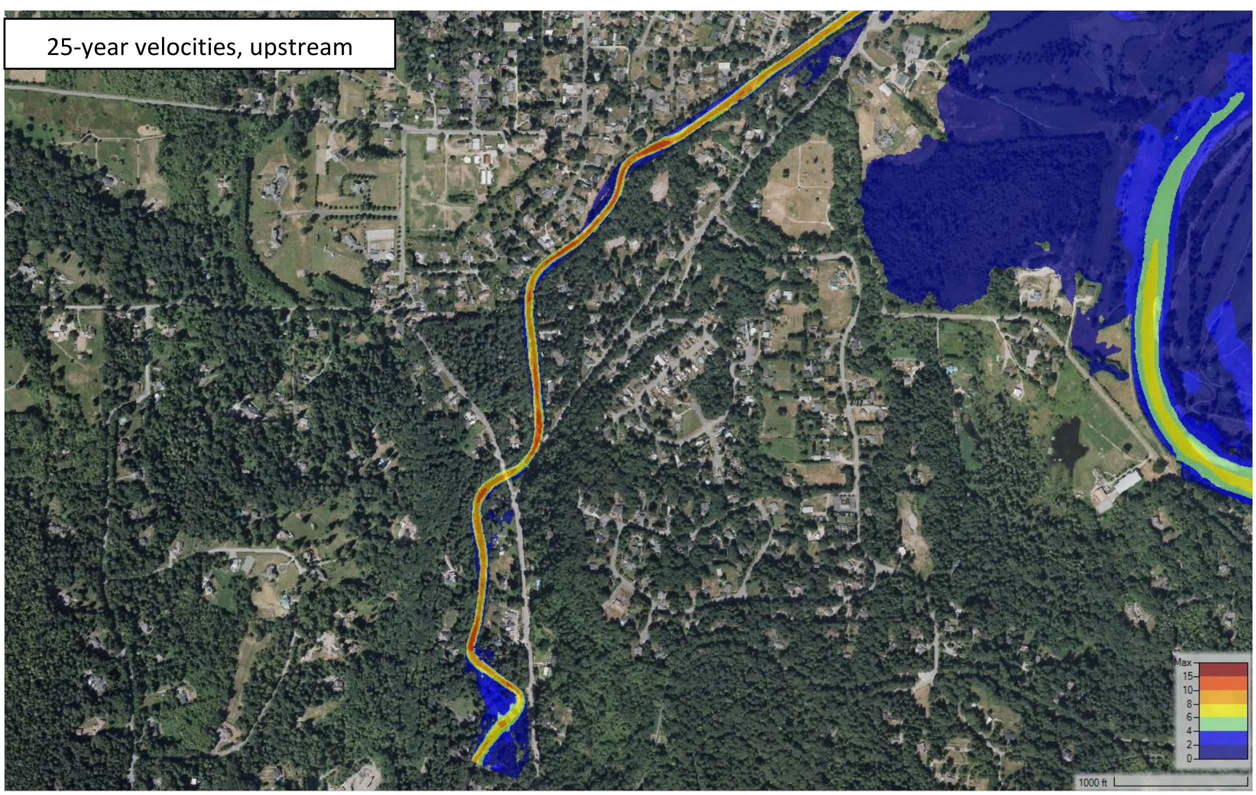
500-year +FC velocities, near confluence

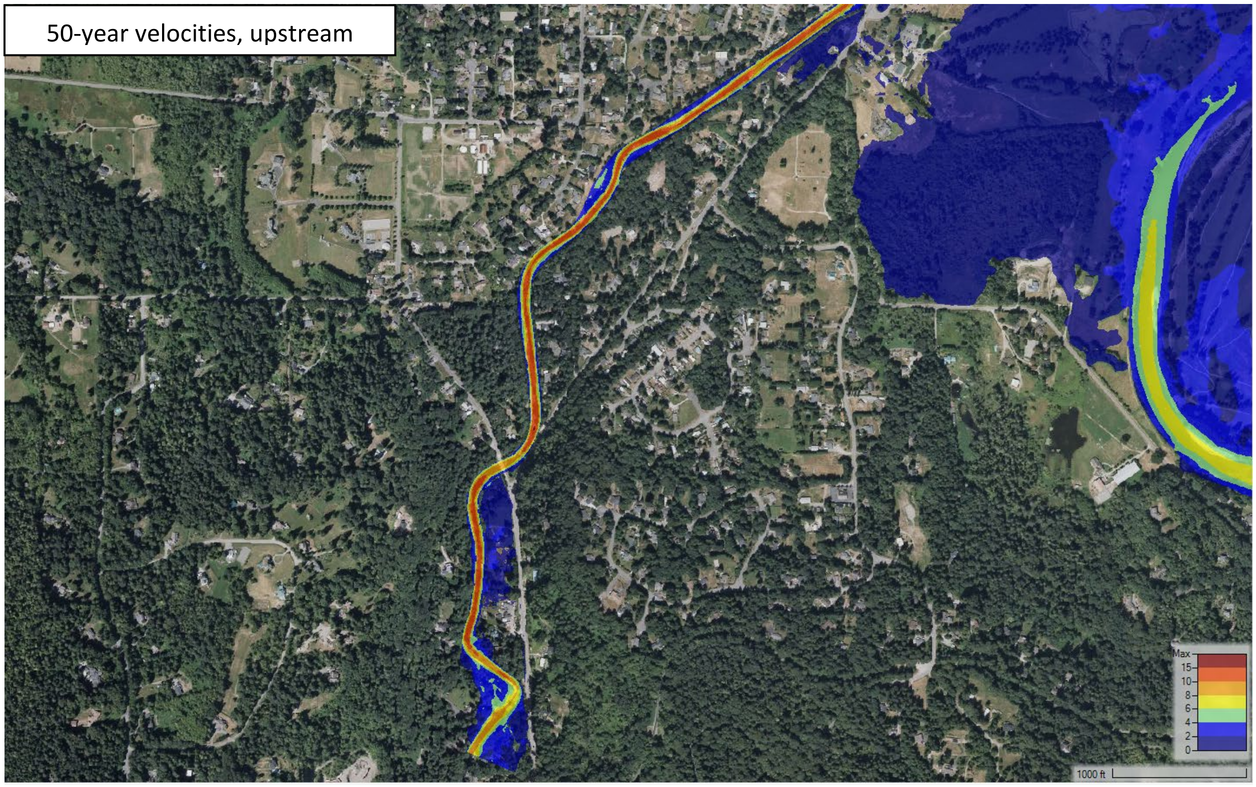


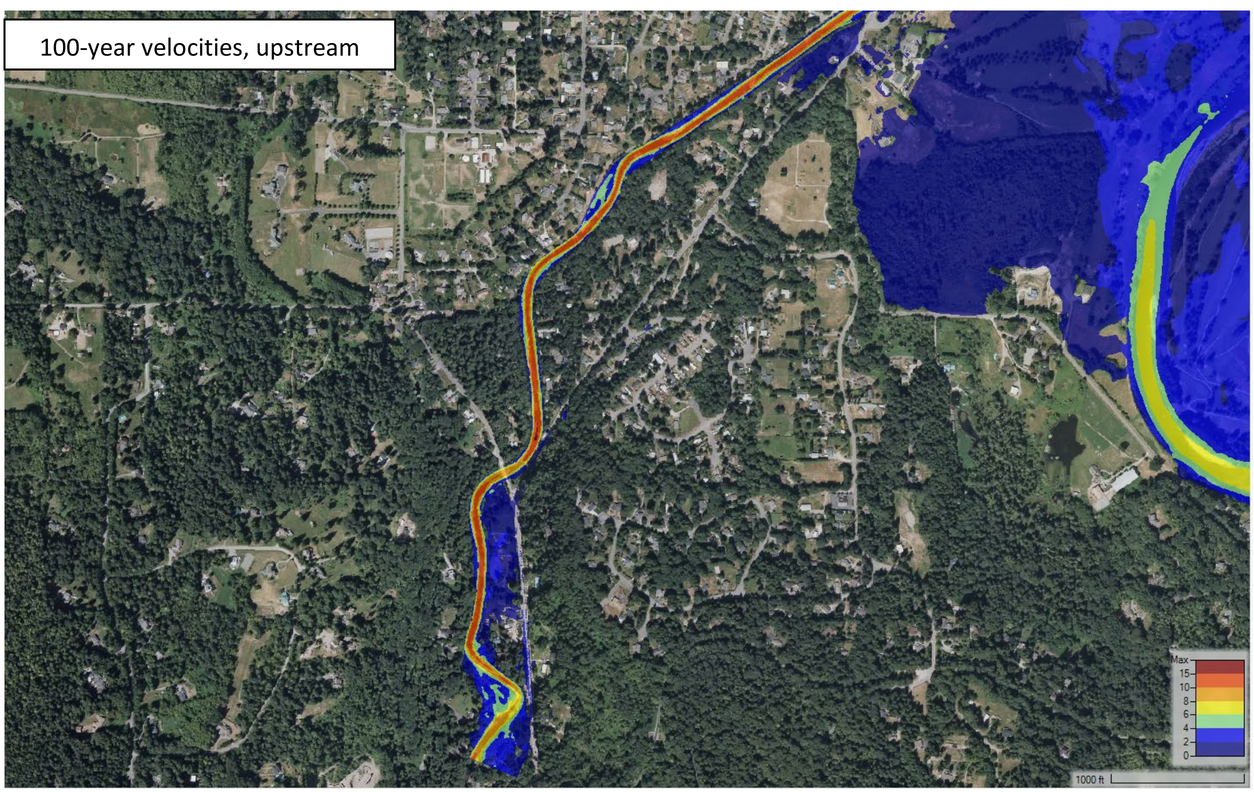

2-year velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft

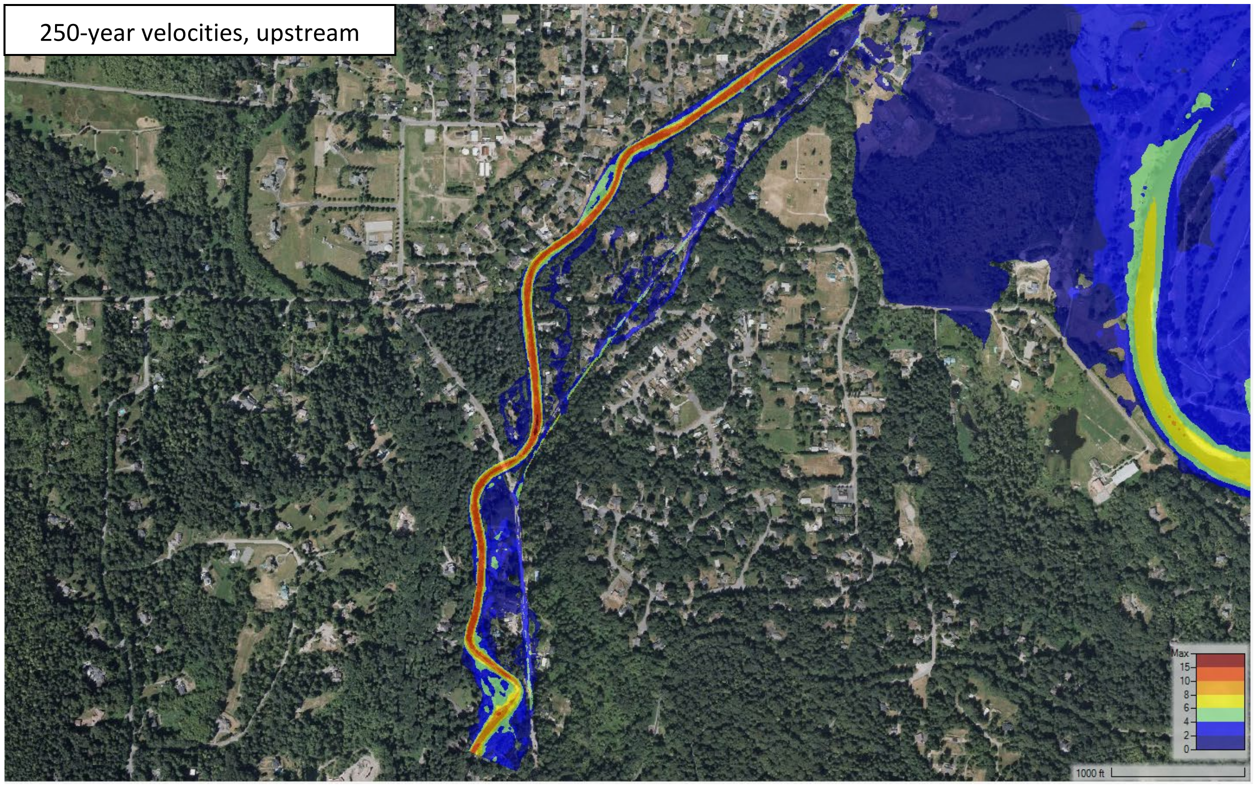

5-year velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft

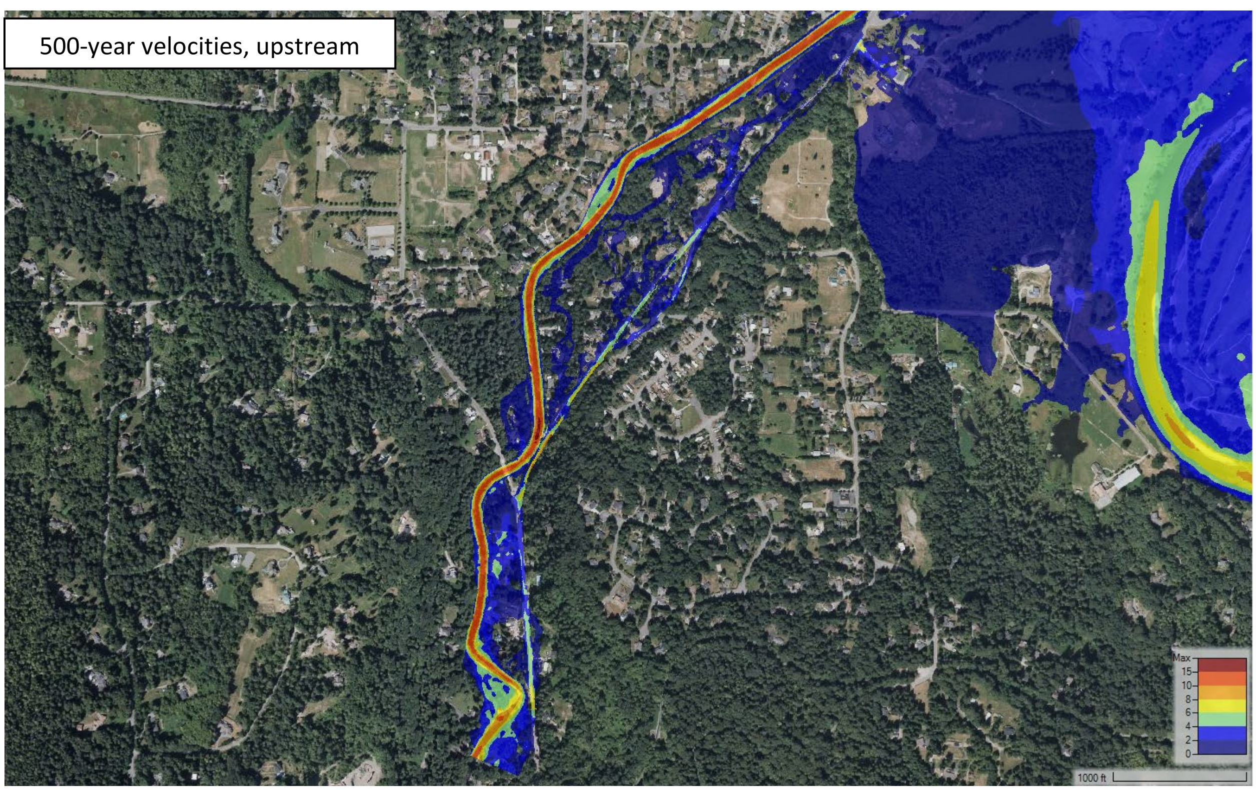

10-year velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft

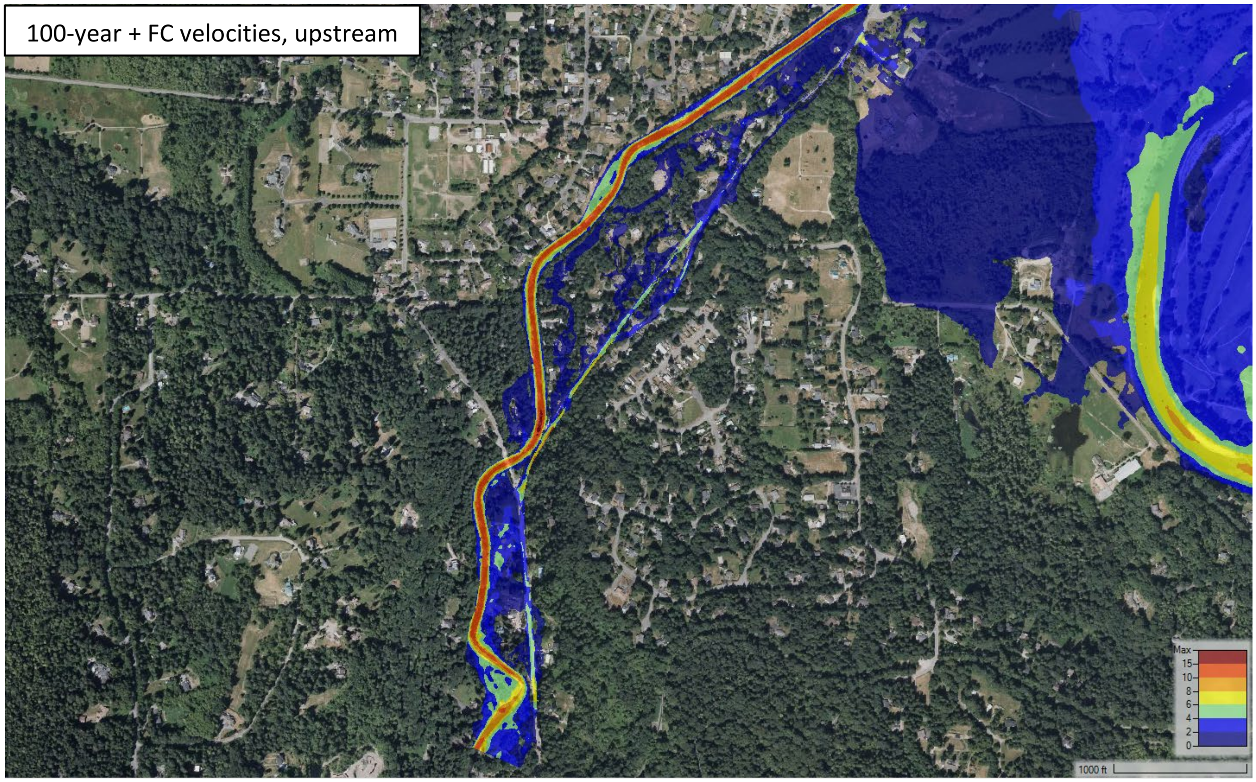

25-year velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft

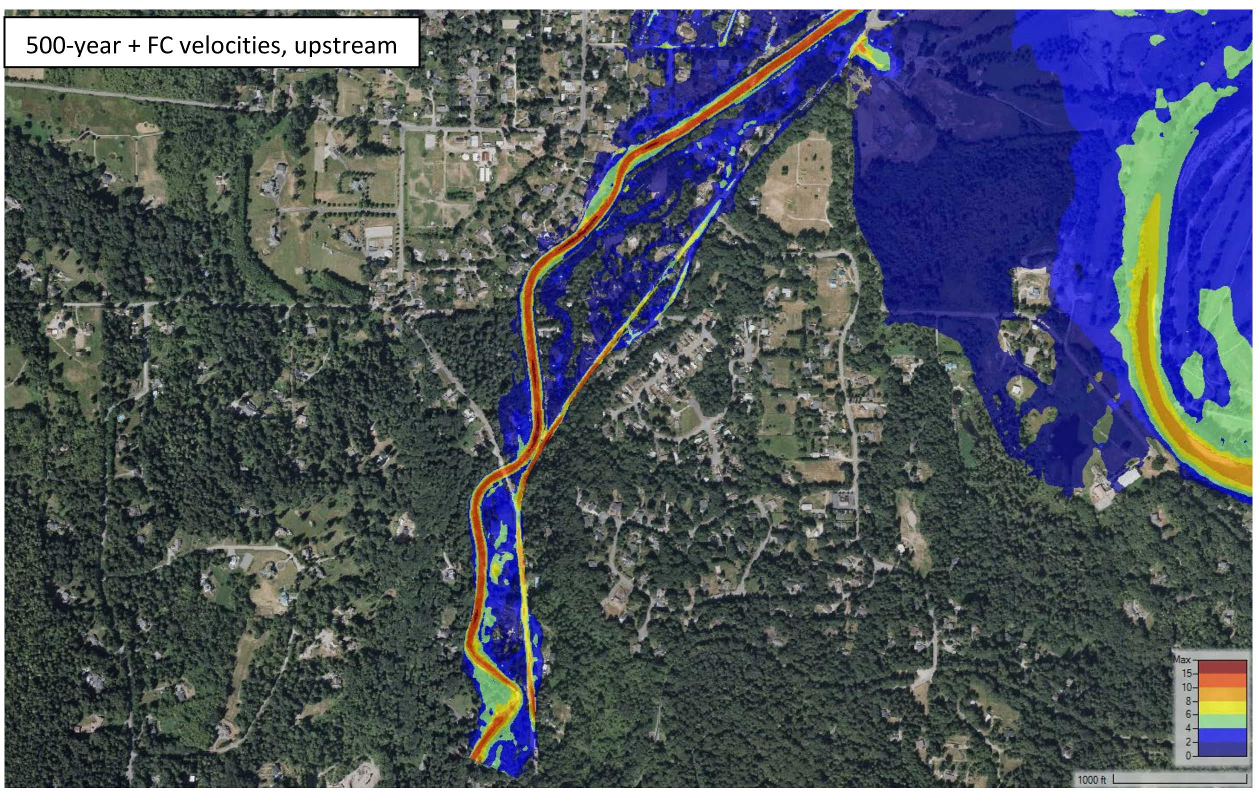

50-year velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft

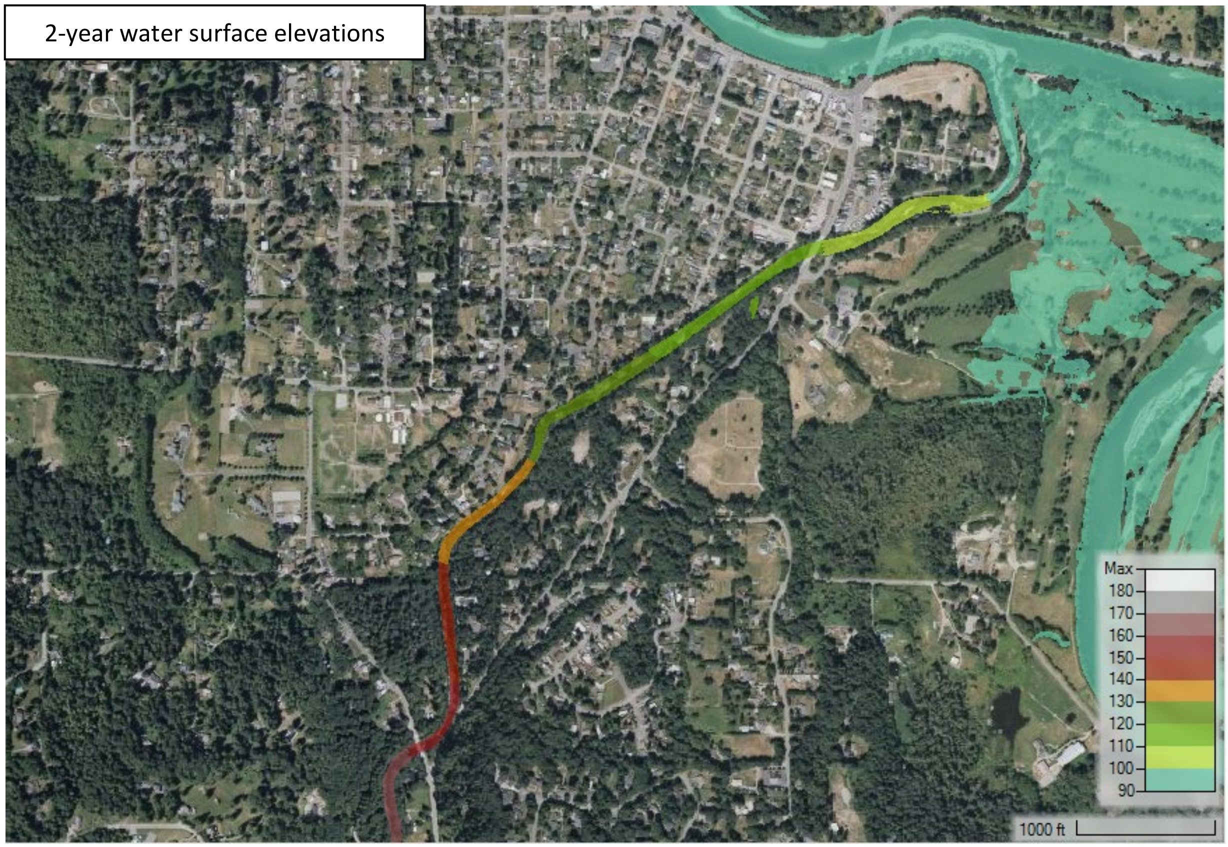

100-year velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft


250-year velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft

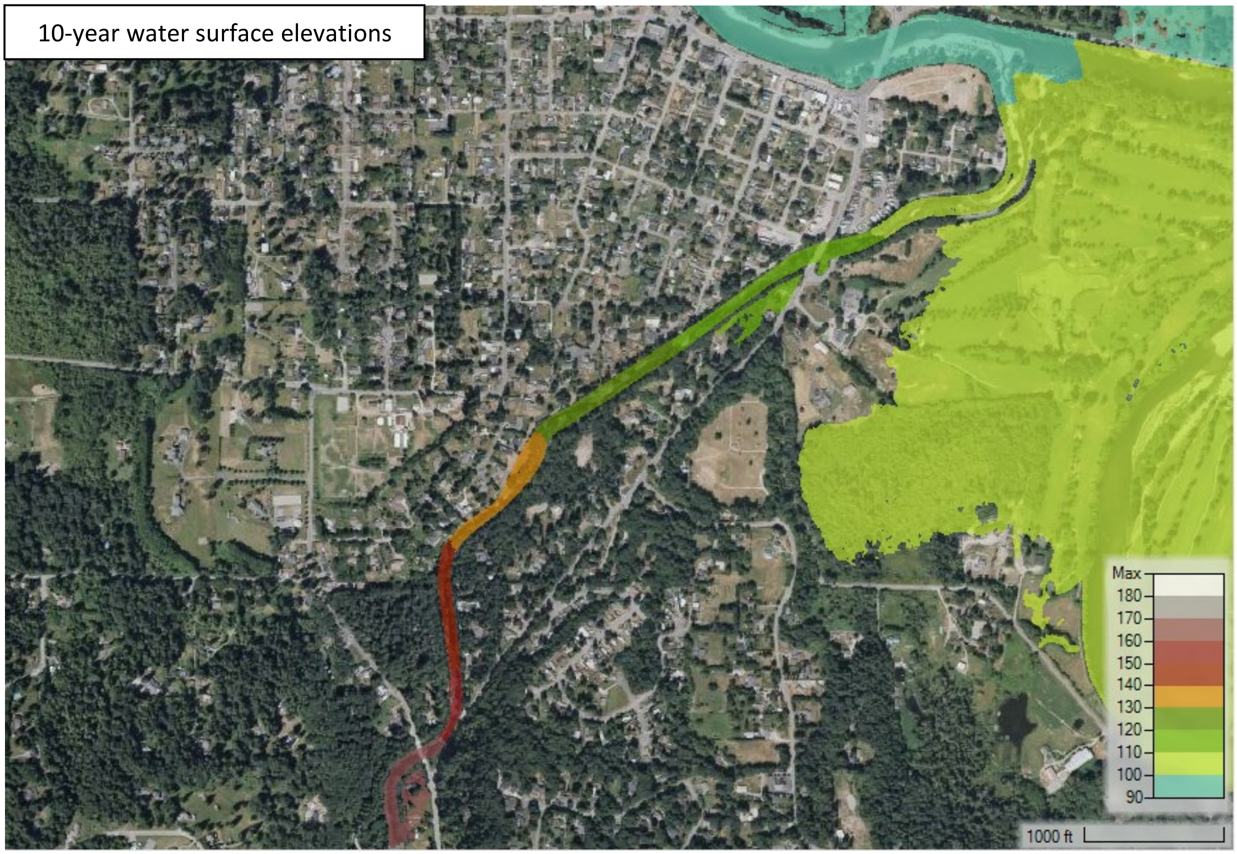

500-year velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft

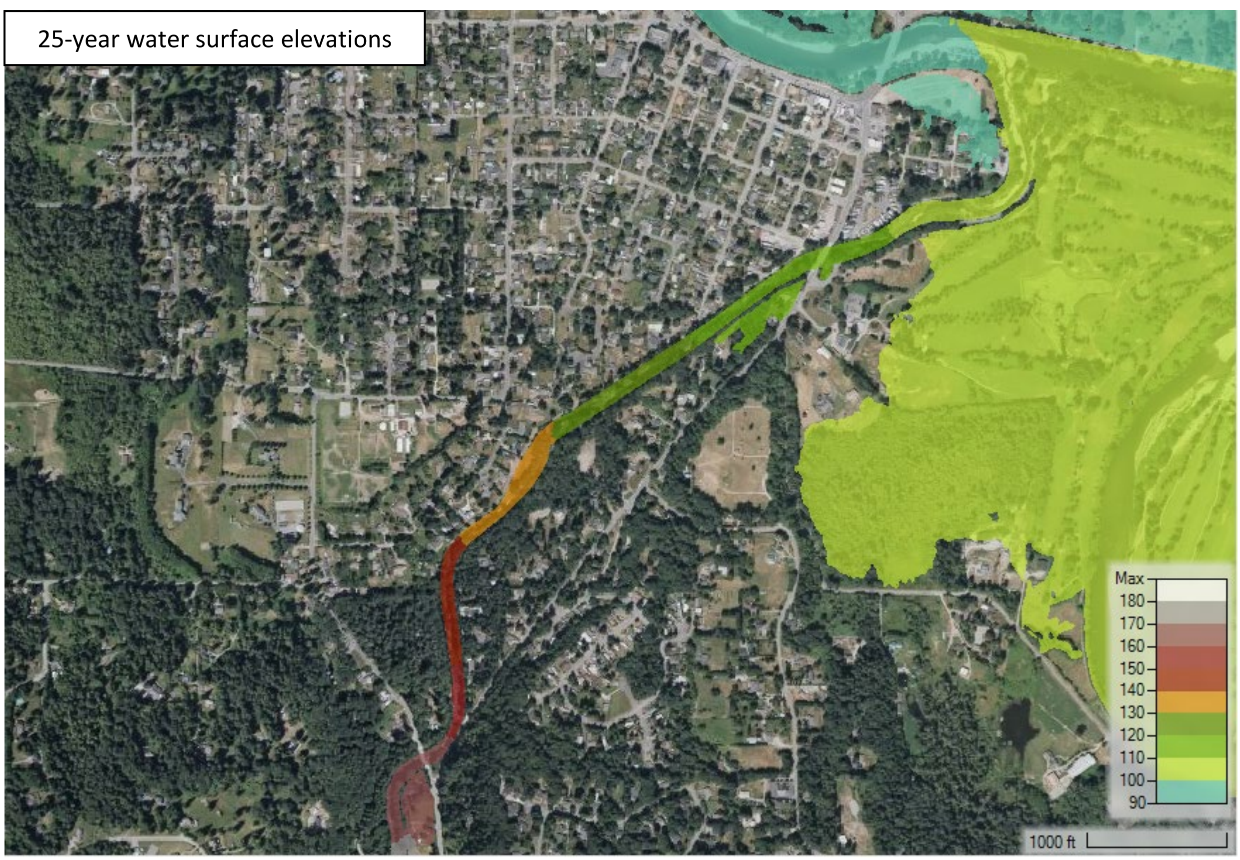

100-year + FC velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft

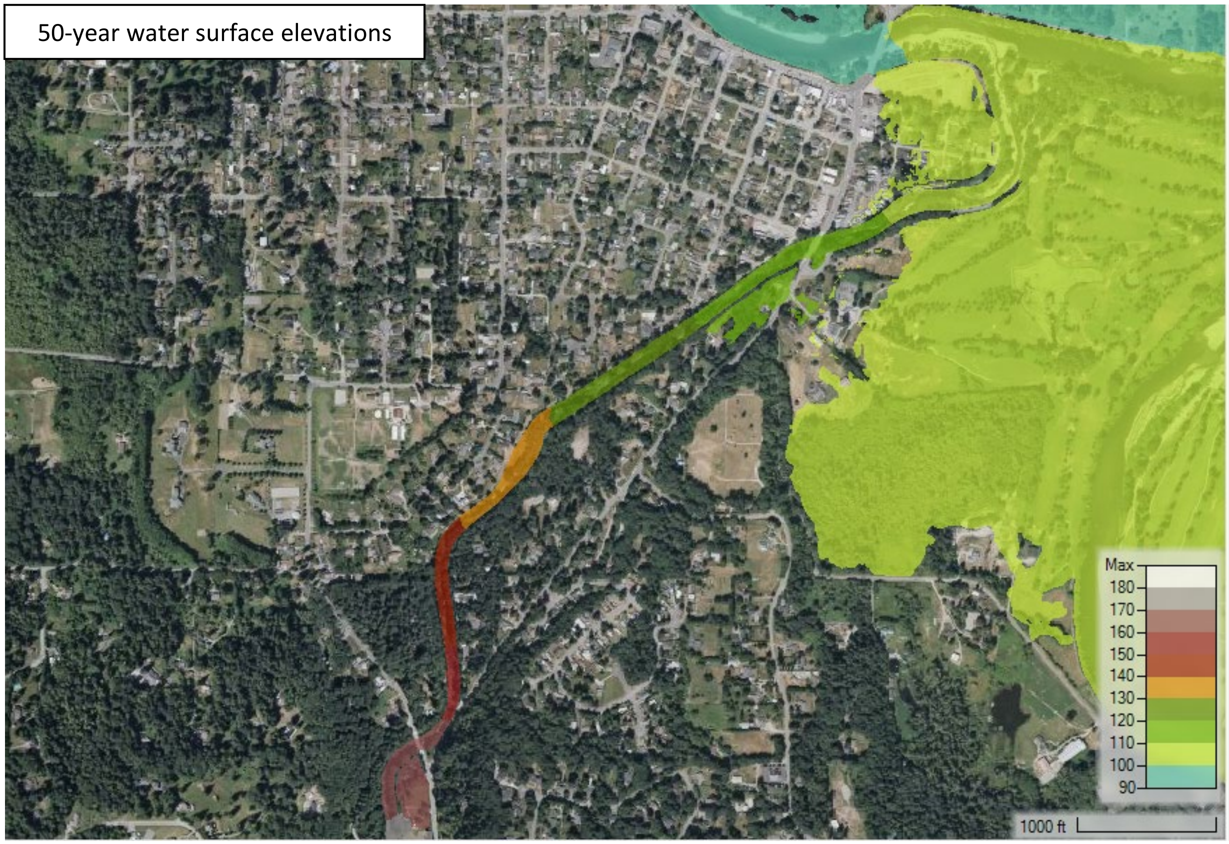

500-year + FC velocities, upstream
Max
15
10
8
6
4
2
0
1000 ft


2-year water surface elevations
Max
180
170
160
150
140
130
120
110
100
90
1000 ft

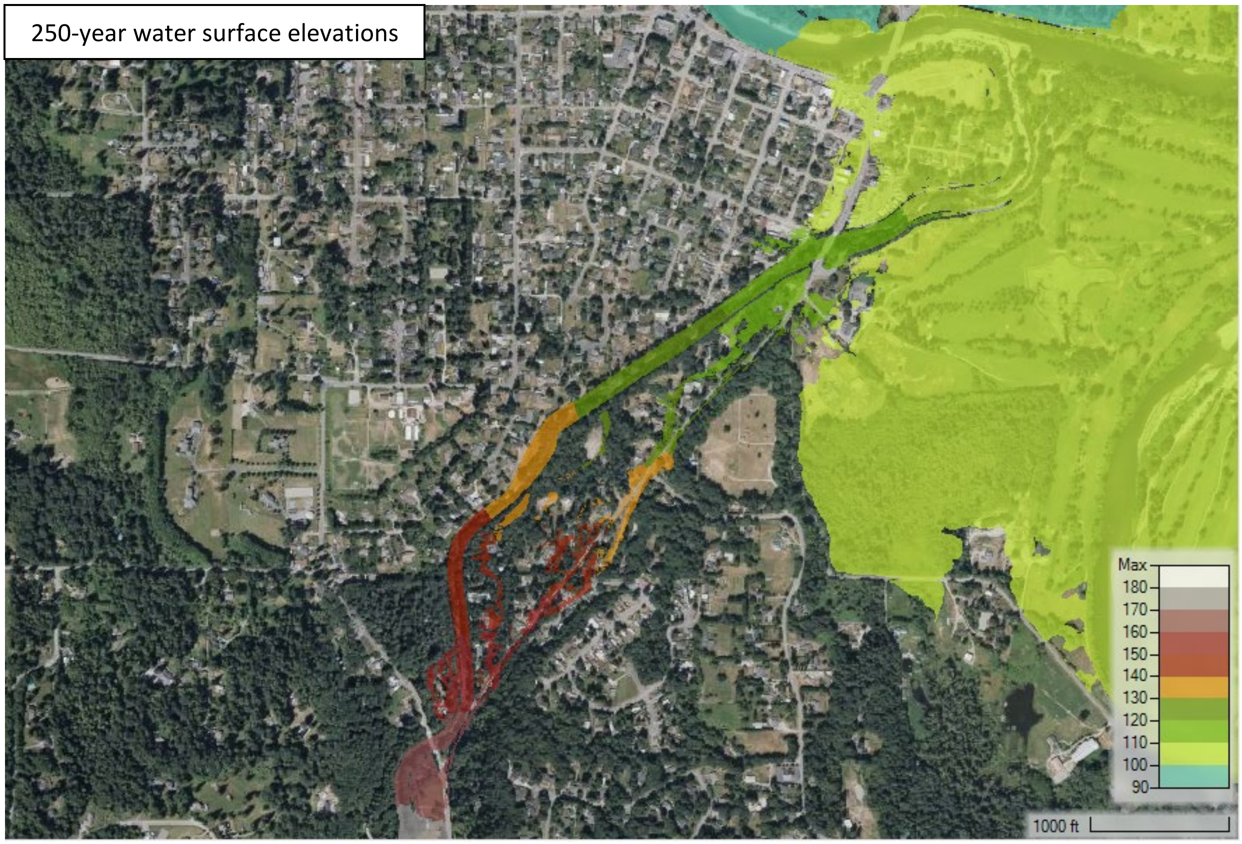

5-year water surface elevations
Max
180
170
160
150
140
130
120
110
100
90
1000 ft

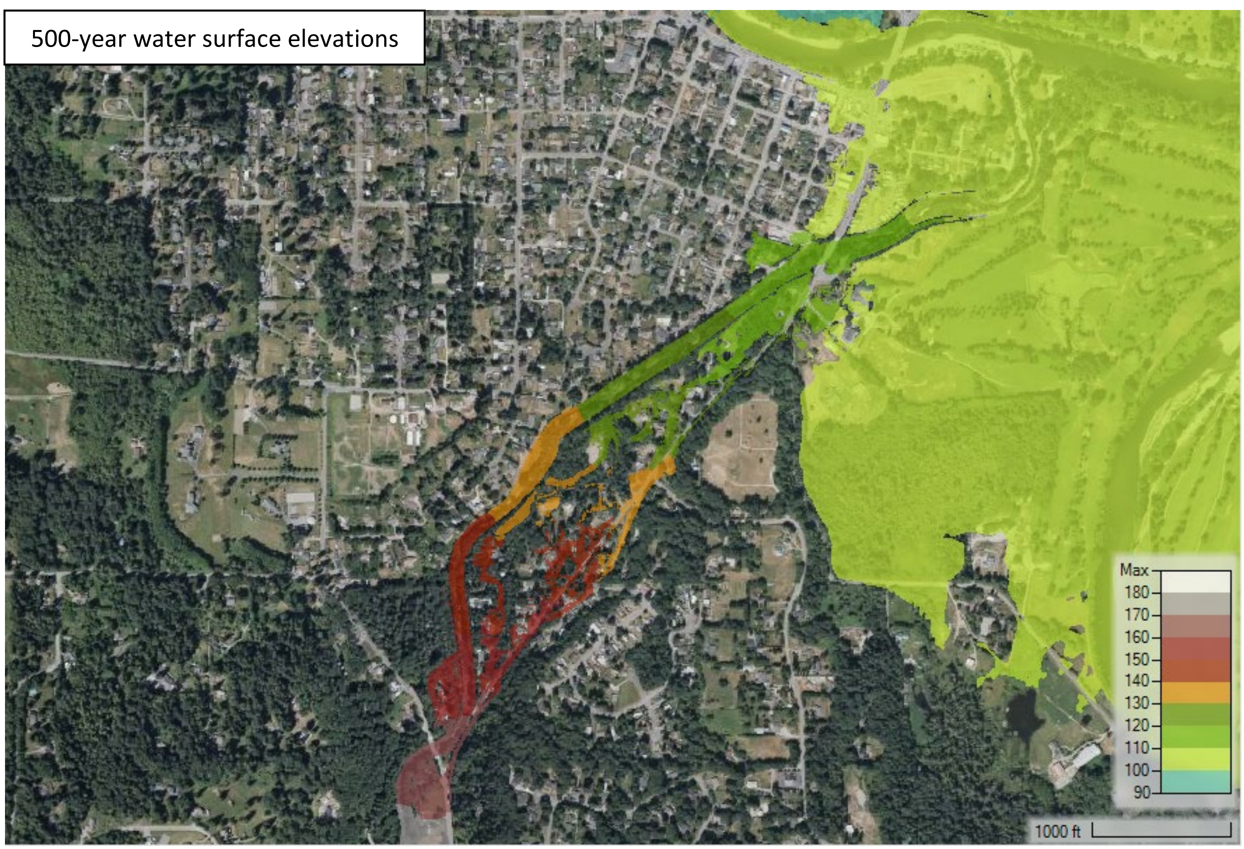

10-year water surface elevations
Max
180
170
160
150
140
130
120
110
100
90
1000 ft


25-year water surface elevations
Max
180
170
160
150
140
130
120
110
100
90
1000 ft

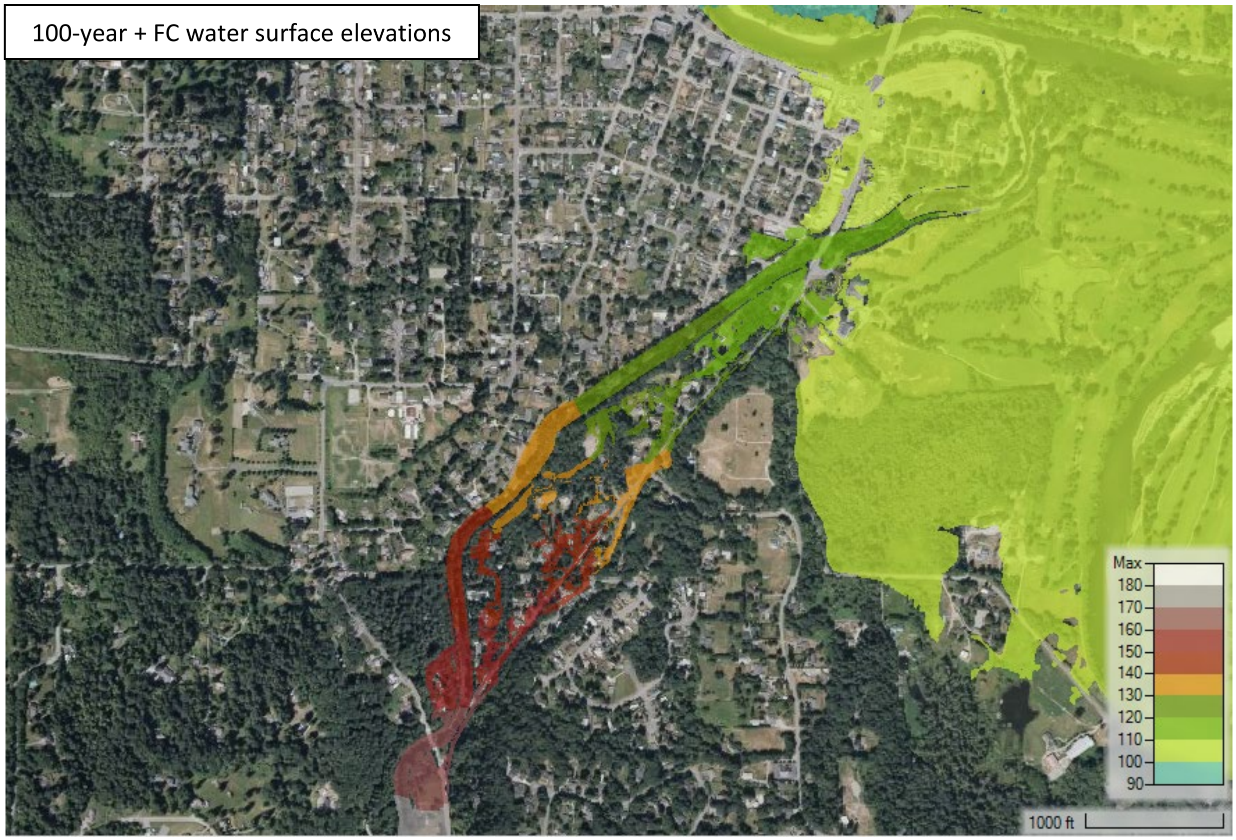

50-year water surface elevations
Max
180
170
160
150
140
130
120
110
100
90
1000 ft


100-year water surface elevations
Max
180
170
160
150
140
130
120
110
100
90
1000 ft

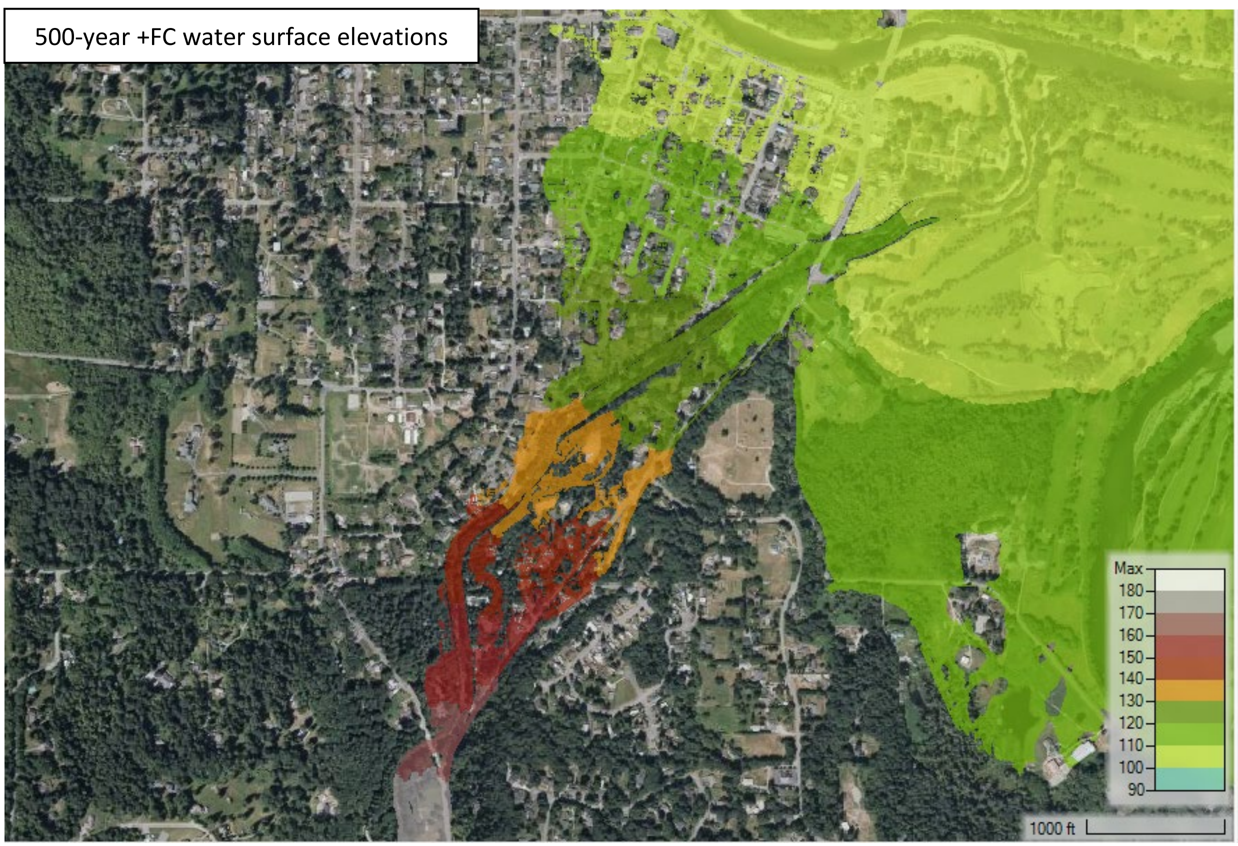

250-year water surface elevations
Max
180
170
160
150
140
130
120
110
100
90
1000 ft

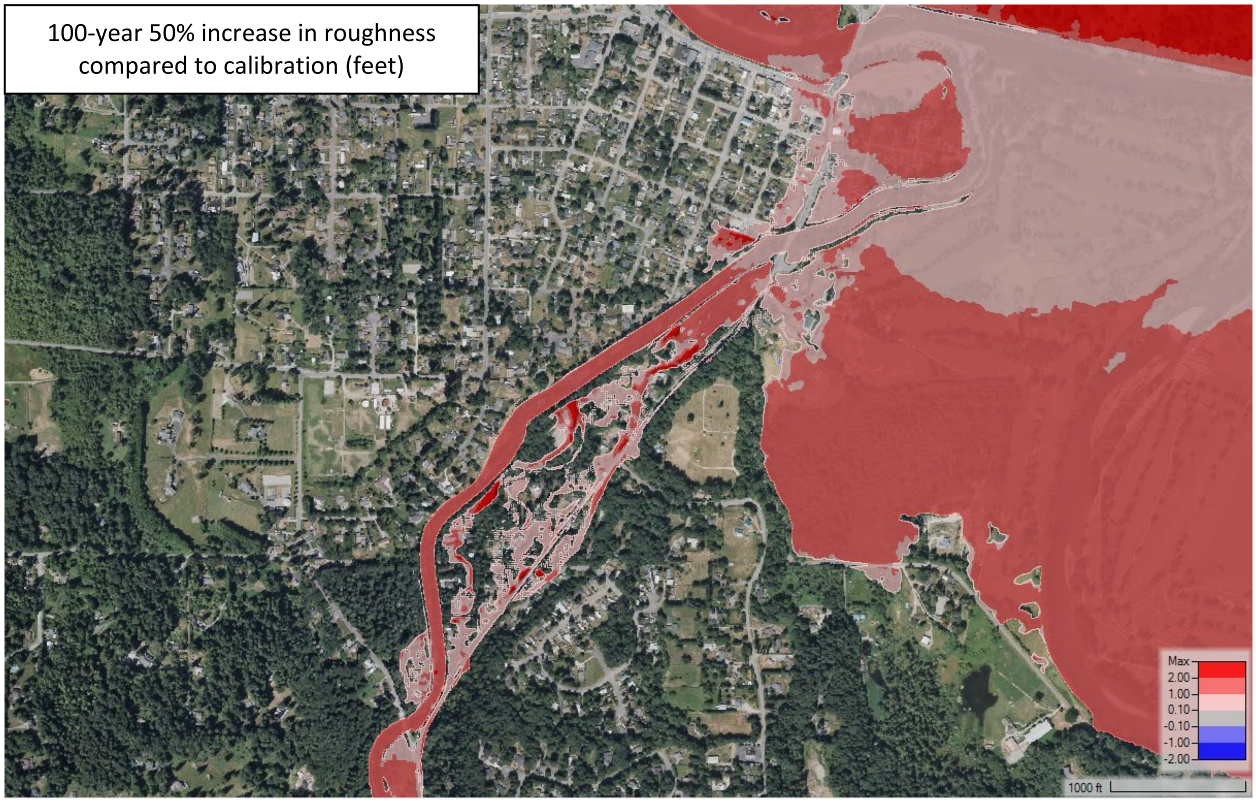

500-year water surface elevations
Max
180
170
160
150
140
130
120
110
100
90
1000 ft

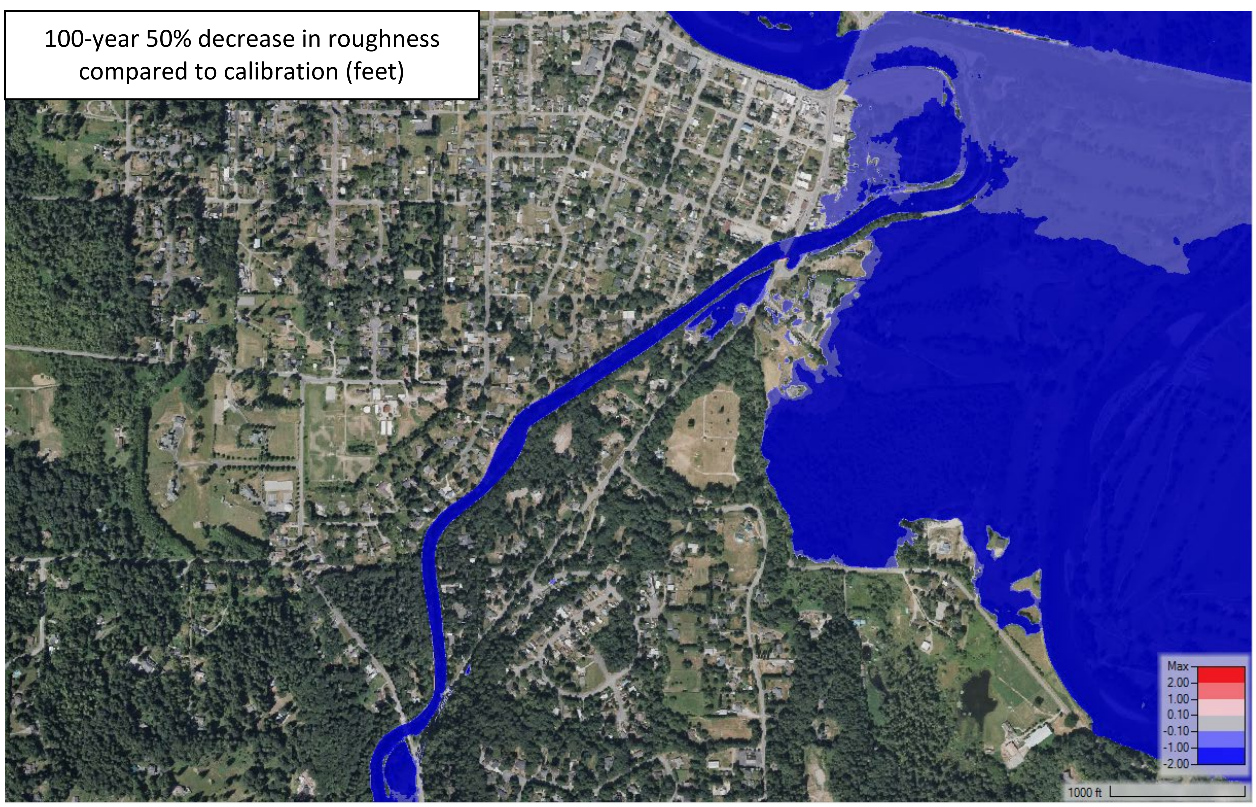
100-year + FC water surface elevations

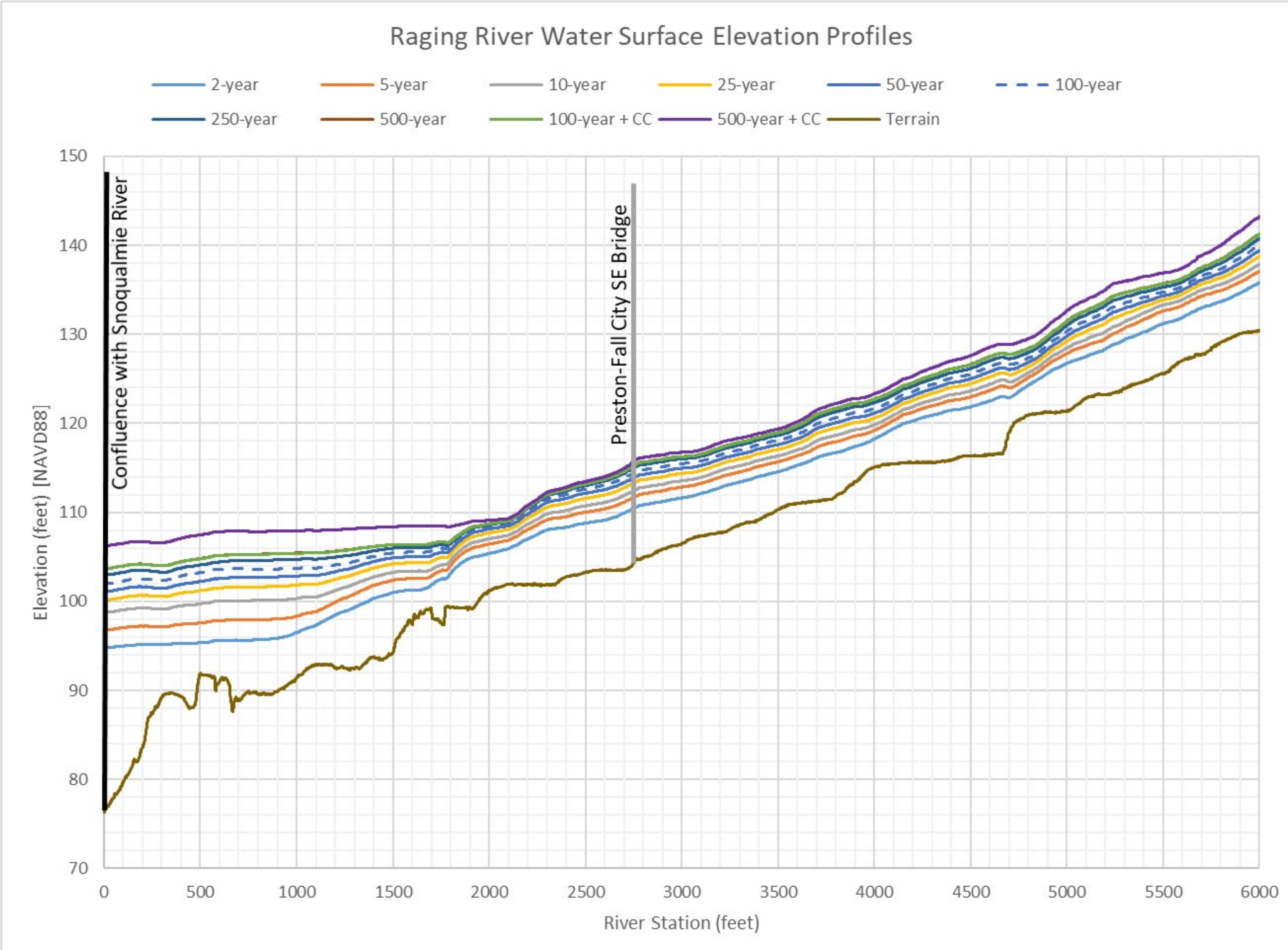


500-year +FC water surface elevations

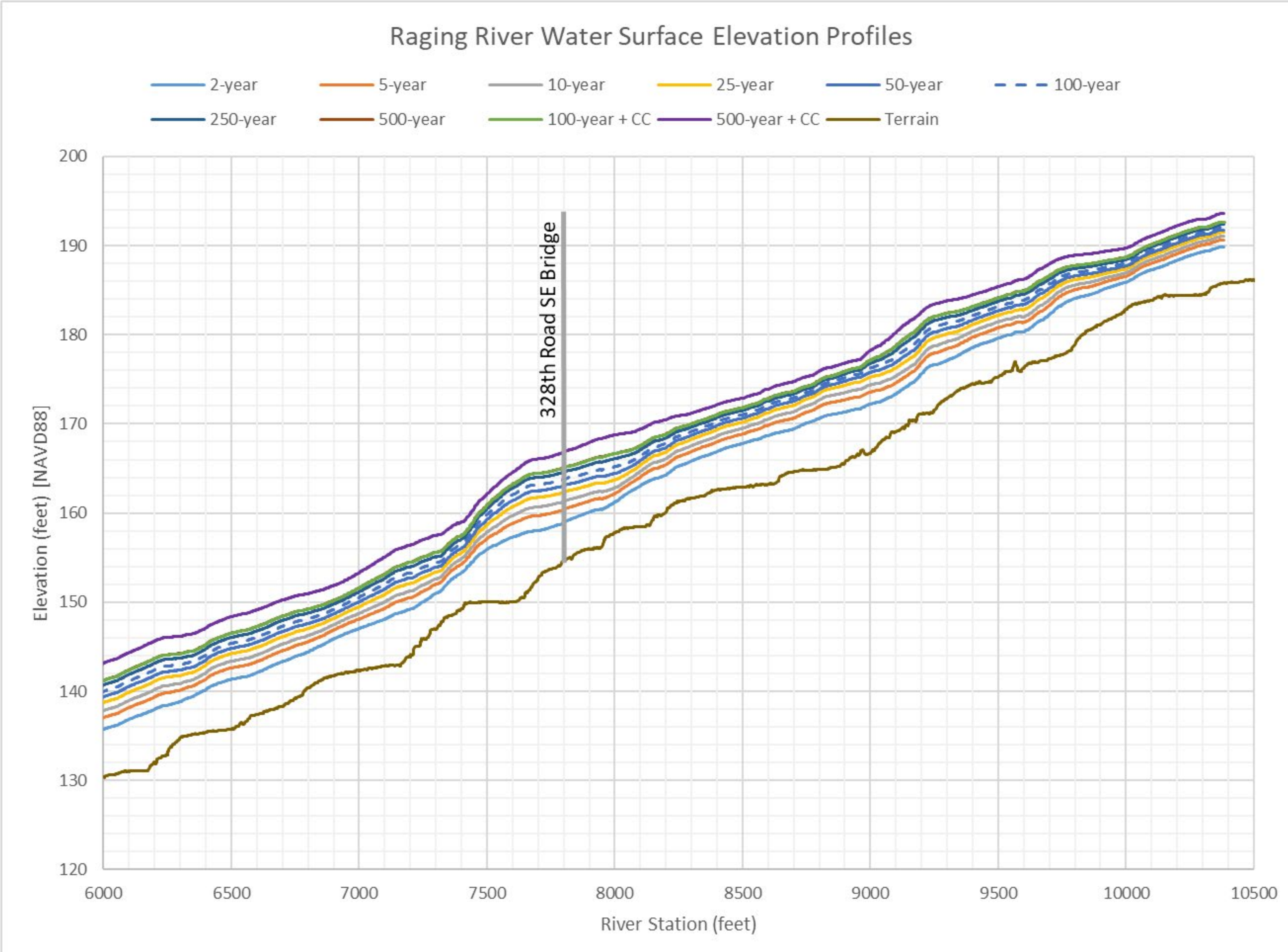



100-year 50% increase in roughness compared to calibration (feet)
Max
2.00
1.00
0.10
-0.10
-1.00
-2.00
1000 ft


100-year 50% decrease in roughness compared to calibration (feet)
Max
2.00
1.00
0.10
-0.10
-1.00
-2.00
1000 ft



Raging River Water Surface Elevation Profiles
2-year
5-year
10-year
25-year
50-year
100-year
250-year
500-year
100-year + CC
500-year + CC
Terrain
Confluence with Snoqualmie River
Preston-Fall City SE Bridge
Elevation (feet) [NAVD88]
River Station (feet)
150
140
130
120
110
100
90
80
70
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
6000



Raging River Water Surface Elevation Profiles
2-year
5-year
10-year
25-year
50-year
100-year
250-year
500-year
100-year + CC
500-year + CC
Terrain
328th Road SE Bridge
Elevation (feet) [NAVD88]
River Station (feet)
200
190
180
170
160
150
140
130
120
6000
6500
7000
7500
8000
8500
9000
9500
10000
10500

APPENDIX D

QUALITY CONTROL

2D HYDRAULIC MODEL REVIEW CHECKLIST

PROJECT:	2004712	**REVIEWER:**	Todd Bennett, P.E.
CLIENT:	Shannon & Wilson for King County	**MODELER:**	Kristin Kramer
RIVER:	Raging River	**DATE:**	
MODEL FILE PATH:	Q:\2004712_King_County_Levee_Breach_Analysis\300_Task300_Raging\06_H		

REVIEW ITEM	COMMENT	ACTION NEEDED (blank = none)	RESPONSE TO COMMENT	DATE RESOLVED	COMMENT CLOSED (Reviewer Initials)
Model/Background Data					
Version of SMS/SRH/RAS	RAS Version 6.1.0. Text should be updated accordingly in the description portion of the main RAS program window.	Y	Text added to main RAS window specifying the version of HEC-RAS this model was developed with.		THB
Project Vert Datum	NAVD88	N			
Project Horz Datum	NAD_1983_StatePlane_Washington_North_FIPS_4601_Feet	N			
Metadata included in model files	Not reviewed	Y	Metadata will be provided in final model		THB
Topography					
Source/Date					
Datums verified					
Bathymetry					
Source/Date					
Datums verified					
Additional Survey					
Source/Date					
Datums verified					
Bridge/Culvert/Structure Data					
Source/Date					
Datums verified					
Terrain Data Review					
Data consistency (datums/projections/merging)	Terrain not reviewed as part of this evaluation	N	No response needed.		THB
Does the final surface accurately represent the site? (are hydraulic controls represented)	Terrain not reviewed as part of this evaluation	N	No response needed.		THB
Were voids filled? If so does the void filling look reasonable?	Terrain not reviewed as part of this evaluation	N	No response needed.		THB
Confirm terrain breaklines used where necessary	Terrain not reviewed as part of this evaluation	N	No response needed.		THB
Mesh					
Are the number of elements reasonable?	The number of elements is reasonable.	N	No response needed.		THB
Mesh quality (odd shaped elements, orientation to flow)	Meshing generally look approapriate. The mesh is aligned to flow.	N	No response needed.		THB
Is the upstream mesh limit sufficient?	The downstream limit seems sufficient.	N	No response needed.		THB
Is the downstream mesh limit sufficient?	The upstream limit seems sufficient.	N	No response needed.		THB
Are the lateral extents sufficient?	The highest simulated flow (500-year with climate change) is contained within the mesh limit	N	No response needed.		THB
Are key project features correctly represented?	Meshing looks appropriate around key features.	N	No response needed.		THB
Confirm mesh breaklines used where necessary	Breakline look appropropriate.	N	No response needed.		THB
Materials/Land Cover					
Source of Materials/Land Cover	The data source is documented in the report.	N	No response needed.		THB
Are material types correctly assigned?	Values are assigned correctly.	N	No response needed.		THB
Are appropriate n values used?	Generally assigned Manning n values seem appropriate. Though it likely doesn't affect results, a Manning n of "10" seems high for "buildings".	Y	Sensitivity to this value was evaluated.		THB
Boundary Conditions					
Steady/Unsteady and is this appropriate?	Unsteady state (though two of the boundary conditions use steady state flows)	N	No response needed.		THB
Upstream BCs- verify correct type and implementation on mesh	The three upstream boundary conditions span the active flow channel.	N	No response needed.		THB
Upstream BCs- data source	Computed or from FEMA.	N	No response needed.		THB
Upstream BCs- check values used, their development if derived, whether they match source if historical	Upstream peak flow boundary condition data were checked against values referred to in the draft documenation. The 72-hour hydrographs were only simulated for 12-hours around the peak. Are simulation results senstivite to running the entire 72-hours?	Y	Sensitivty to running the full 72-hour hydrograph did not make a significant difference in the simulated water surface elevation or the amount inundated area.		THB
Downstream BCs- verify correct type and implementation on mesh	Rating curves were specified at the downstream boundary conditions.	N	No response needed.		THB
Downstream BCs- data source	Results from prior reach wide hydraulic model (King County Farm Pads in NAVD88).	N	No response needed.		THB
Downstream BCs- check values used, their development if derived, whether they match source if historical	Compared downstream rating curves to values provided in 2D_RAS_Documentation.xlsx. The largest simulated flows extend beyond the values defined in the rating curves (~50k cfs on the left overbank and ~100k cfs simulated in the main channel).	Y	It was assumed the a linear extension of the rating curve data beyond the listed data values was sufficient for these simulations.		THB
Hydraulic Structures					
How many hydraulic structures are represented? What types?	Preston-Fall City Rd. and SR 202 crossings were modeled using 1-D bridges, there is one culvert, and the model includes several weirs (representing the channel levees).	N	No response needed.		THB
Bridges					
Is the bridge geometry correct?	Compared Preston-Fall City Rd. model geometry to survey. Drawings labeled as NAVD88.	N	No response needed.		THB
Are piers modeled correctly?	Both Preston-Fall City Rd. and SR 202 bridges use Energy Method. Since both these bridges have piers, were Momentum and Yarnell Methods considered, and do they make a significant difference in the results?	Y	Both Momentum and Yarnell were considered, but neither yields a significant difference in the simulated water surface elevation.		THB

2D HYDRAULIC MODEL REVIEW CHECKLIST					
PROJECT:	2004712		**REVIEWER:**		Todd Bennett, P.E.
CLIENT:	Shannon & Wilson for King County		**MODELER:**		Kristin Kramer
RIVER:	Raging River		**DATE:**		
MODEL FILE PATH:	Q:\2004712_King_County_Levee_Breach_Analysis\300_Task300_Raging\06_H				
REVIEW ITEM	**COMMENT**	**ACTION NEEDED (blank = none)**	**RESPONSE TO COMMENT**	**DATE RESOLVED**	**COMMENT CLOSED (Reviewer Initials)**
Is pressure flow accounted for correctly (geom and calculation method)?	The 500-year climate change simulated water surface, the largest of the flows reviewed, is just below the low cord on the upstream side of the Preston-Fall City bridge, and well below the maximum low cord for the SR 202 bridge.	N	No response needed.		THB
Culverts					
Are the culvert geometries correct?	One culvert is included in the model (at 2D Connection Brdg-Bridg_RB_A) and the geometry matches what is listed in the report.	N	No response needed.		THB
Are coefficients are parameters reasonable?	Culvert parameters are reasonable.	N	No response needed.		THB
Obstructions					
Are obstructions used in the model? If so are they applied correctly?	Bridges are the only obstructions modeled within the main channel.	N	No response needed.		THB
Hydraulic Analysis					
Model Controls and Simulations					
Are simulation settings reasonable?	Reviewed computation and tolerance settings. Are model results sensitive to using the turbulence model?	Y	Using turbulence with reasonable parameters results in Raging River stage increasing by 0.2 ft and velocities decreasing by 0.5 fps (for 100-yr event). The Raging River is a fairly uniform channel, and some numerical diffusion is present in the solution algorithm, so don't feel it is necessary to add additional turbulent diffusion in this case.		THB
Are simulations labeled correctly and include the correct components?	Simulation plan names are intuitive.	N	No response needed.		THB
Model Results					
Confirm model stability at locations of interest	Any observed instabilities had oscilations less than 0.1 feet.	N	No response needed.		THB
Confirm volume conservation	For the simulations checked, RAS reported overall volume accounting error of less than 1 acre-foot and approximately "zero" percent.	N	No response needed.		THB
Do results contain any oddities (check WSE, velocity, Depth, Shear, Fr)	No oddities were obsereved.	N	No response needed.		THB
Calibration					
Was the model calibrated? If so does the calibration appear adequate?	There is little reported data calibration data available. For the Nov. 2006 event, a visual comparison was made between simulated and observed flooding. Consider a similar qualitative assessment for other events, and discuss with County if anecdotal information are available for other known inundated areas and flood flow paths and conduct additional comparisons.	Y	All Raging River observed flooding was within the channel banks so out of bank flooding could not be compared. NHC will follow up with the County to confirm that there is no anecdotal information on observed Raging River levee overtopping.		THB
If no calibration was performed, was a sensitivity analysis performed, if so what was determined?	Sensitivity analysis was conducted assessing change in flood depth and inundated area.	N	No response needed.		THB
General Comments					