DRAFT
SNOQUALMIE RIVER HYDRAULIC STUDY

EVALUATION OF EFFECTS OF SNOQUALMIE FALLS PROJECTS ON DOWNSTREAM FLOODING

HERRERA

King County

Department of Natural Resources and Parks
Water and Land Resources Division

WATERSHED SCIENCE & ENGINEERING

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Prepared for

King County

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The analyses documented in this report were conducted by Watershed Science & Engineering (WSE) on behalf of the King County Department of Natural Resources and Parks, Water and Land Resources Division (King County). This work was done in close coordination with King County to obtain available information and strategize the best technical approaches to answer questions and concerns related to flooding that have been voiced to King County in recent years by many residents and land owners in the Snoqualmie River valley. While King County staff provided guidance and review of the contents presented in this report to ensure the work would meet the study's objectives, all of the technical work of this study was done by WSE. Herrera Environmental Consultants assisted in managing the study and compiling the report contents with editorial support from McAuliffe Technical Editing Services.

An independent technical reviewer representing the interests of Snoqualmie valley residents and stakeholders, Dr. Ed McCarthy, was retained to provide input on the information sources and methods of analysis and to review model results to confirm that the study is technically sound. Dr. McCarthy, a PhD water resources engineer, had complete autonomy in reviewing products and developing comments and recommendations on both interim and final products. He will document his role in the project and his conclusions in a separate memorandum.

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Acronyms and Abbreviations

205 Project	Snoqualmie Flood Damage Reduction Project
cfs	cubic feet per second
County	King County
FEMA	Federal Emergency Management Agency
fps	feet per second
FIS	flood insurance study
HEC-RAS	Hydrologic Engineering Center River Analysis System
HWM	high water mark
LiDAR	light detection and ranging
LOMR	letter of map revision
NAVD88	North American Vertical Datum of 1988
NHC	Northwest Hydraulic Consultants
PSE	Puget Sound Energy
SR	State Route
USACE	US Army Corps of Engineers
USGS	US Geological Survey
WSE	Watershed Science & Engineering

EXECUTIVE SUMMARY

Recently collected data and advanced modeling techniques were used to quantify the effects of two flood reduction projects upstream of Snoqualmie Falls (the Falls) on flows and water levels in the Snoqualmie River valley downstream of the Falls in King County, Washington. Puget Sound Energy (PSE) operates a hydroelectric facility at the Falls, taking advantage of its 268-foot change in elevation. The Snoqualmie Flood Damage Reduction Project (known as the 205 Project) was completed in 2004 by the US Army Corps of Engineers (USACE), King County, and the City of Snoqualmie. It widened the right bank of the Snoqualmie River upstream of PSE's Plant 2 intake structure, widened the left bank of the river downstream from the State Route (SR) 202 Bridge, and removed an abandoned railroad trestle that partially spanned the channel about 0.5 mile upstream from the SR 202 Bridge. River work for the second project, PSE's Snoqualmie Falls Project (PSE Project), was completed between 2010 and 2012. The PSE Project extended the weir that spans the Snoqualmie River just upstream from the Falls, lowered the weir crest, and widened and excavated the left bank to create a transition to the widened weir. Figure ES-1 shows the locations of the two projects.

The hydraulic model used for this study was developed using the USACE's Hydrologic Engineering Center River Analysis System (HEC-RAS) software. Simulations were run to evaluate the effects of the projects on upstream hydraulic conditions and the corresponding effects on downstream flooding. Hydrologic input data were developed from US Geological Survey (USGS) records and Federal Emergency Management Agency (FEMA) floodplain mapping studies. Hydraulic effects of the two projects were evaluated for events ranging from a 2-year flood to a 500-year flood. Detailed results of the simulations are presented for locations upstream and downstream of the Falls. The projects reduce flood water levels upstream of the Falls and generally increase downstream flows and water levels on the rising limb of the flood (i.e., the time period during which discharge, or flow, at the point of measurement is increasing). The magnitude of the downstream increases is dependent on the return period of the flood and the location of interest but the effects are generally greatest closer to the Falls (i.e., at Fall City) and the effects are most pronounced in a 10-year flood.

The 205 Project reduced simulated 100-year-flood water surface elevations in the city of Snoqualmie near Riverview Park by about 0.45 foot. The PSE Project further reduced simulated water levels in Snoqualmie by about 0.96 foot. Together, the combined reduction in peak water level upstream of the Falls is estimated to be about 1.4 foot for the 100-year flood. The lower water levels result from the projects' removal of flow restrictions; the resulting increase in conveyance allows floods to move through the reach upstream of the Falls faster than in the Pre-Projects condition, thereby shifting the timing of the hydrograph earlier.

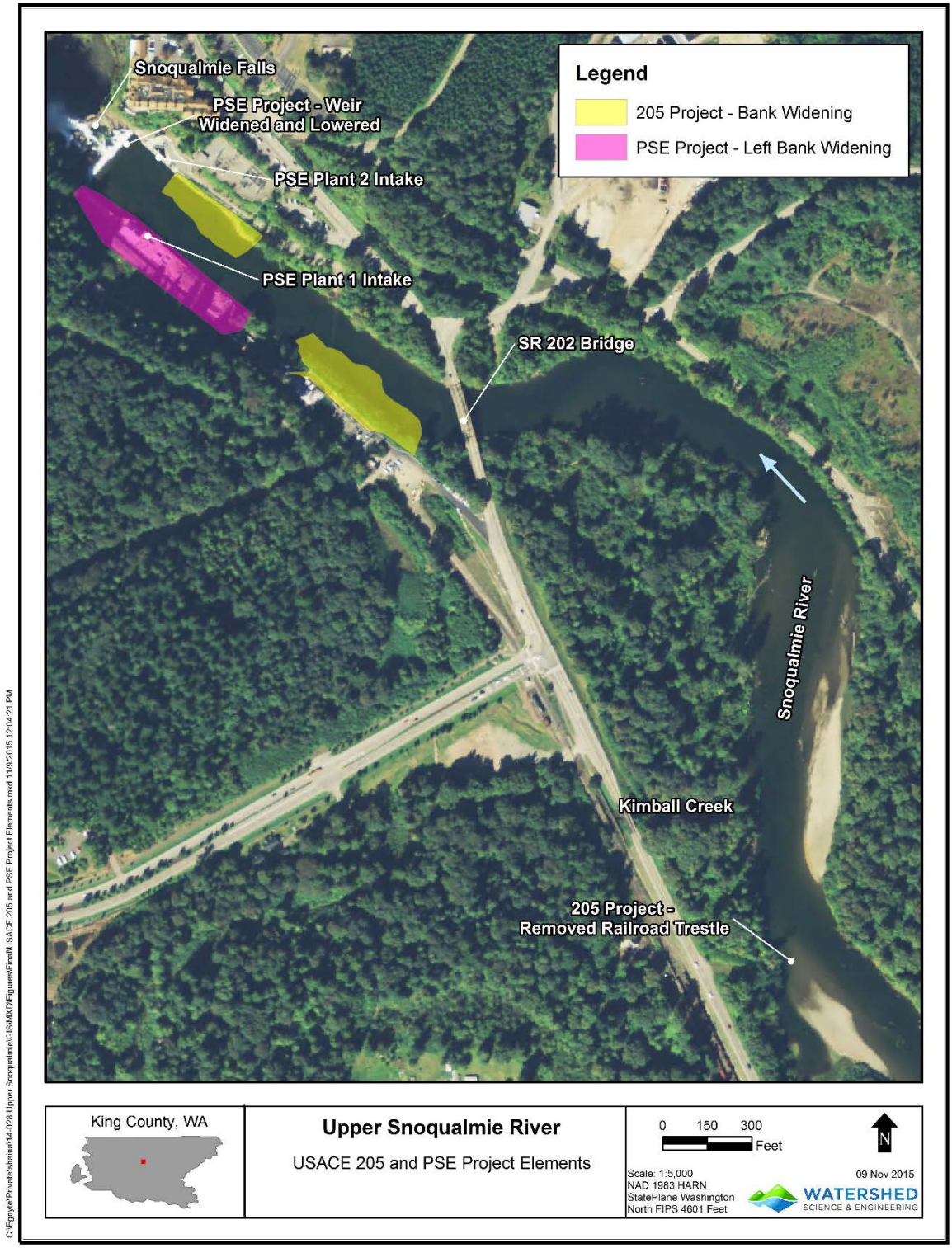

Figure ES-1. USACE 205 and PSE Project Elements for Snoqualmie Falls Projects.

In the Post-Projects condition, the simulated increase in flow at Snoqualmie Falls at the peak of the 100-year flood event is about 1,260 cubic feet per second (cfs) and the peak occurs about 20 minutes earlier than in the Pre-Projects condition. Flow increases on the rising limb of the 100-year flood (before the peak) range as high as 2,600 cfs. After the peak, as the flood is receding, flows at the Falls may be as much as 1,600 cfs lower than in the Pre-Projects condition. Simulated differences in flow at the Falls for the 100-year flood before and after the projects are shown in Figure ES-2.

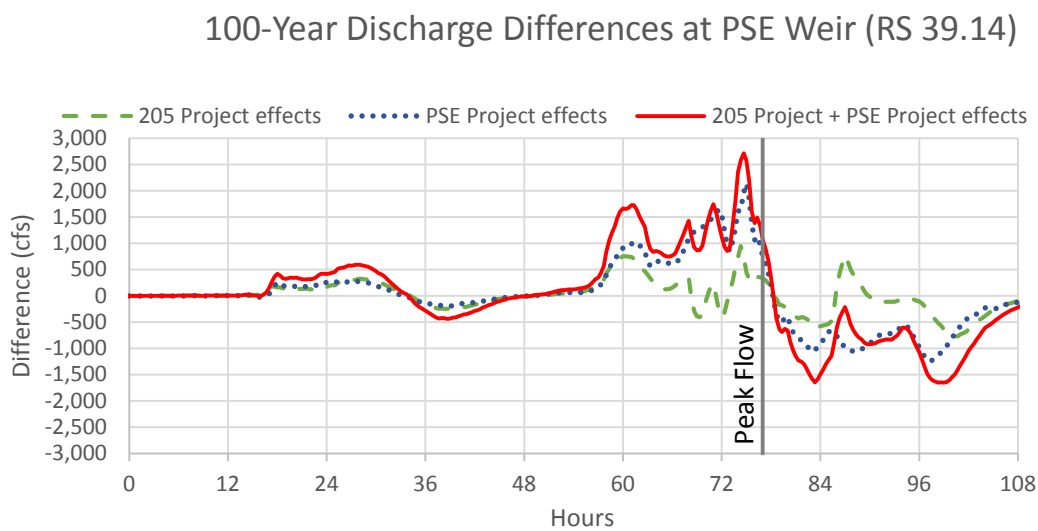

Figure ES-2. Changes in Simulated Flow at Snoqualmie Falls for the 100-Year Flood Resulting from the 205 and PSE Projects.

The increase in upstream conveyance resulting from the 205 and PSE projects causes greater downstream flows and potentially higher water levels in the valley below the Falls. Since the removal of upstream constrictions shifts the timing of the flood event earlier, the largest flow increase relative to conditions before the projects were built occurs prior to the peak of the flood and the largest increase in flood stages (water levels) downstream of the Falls also occurs prior to the peak. Figure ES-3 shows simulated differences in peak stages for a location downstream of the Tolt River confluence near the city of Carnation, approximately 16 river miles below the Falls. The maximum increase in stage is simulated to be 0.15 foot (1.8 inches) on the rising limb of the 100-year flood event at this location. At the time of the peak, however, the modeled increase in stage below the Tolt River confluence resulting from the combined effects of the 205 and PSE projects is about 0.08 feet (less than 1 inch). Following the peak of the simulated 100-year flood event, as the flood flow recedes, water levels below the Tolt River confluence may be up to 0.15 foot (1.8 inches) lower than under the Pre-Projects conditions.

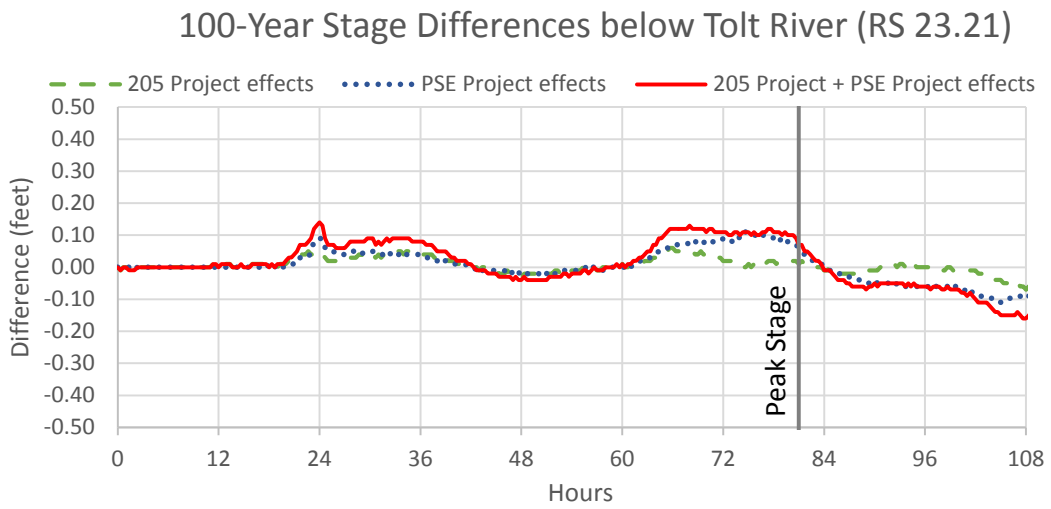

Figure ES-3. Changes Relative to Pre-Projects Condition in Simulated Water Surface Elevation in the Snoqualmie River Below the Tolt River Confluence for the 100-Year Flood.

Figures ES-4 and ES-5 show simulated differences in flood stages for the 10-year flood for a location near Fall City (Figure ES-4) and a location downstream of the Tolt River confluence (Figure ES-5). A comparison of Figure ES-4 to Figure ES-5 shows that the change in stage decreases with increasing distance downstream from the Falls. A comparison of Figure ES-5 (10-year flood event) to Figure ES-3 (100-year flood event) shows how changes in flood stage due to the Snoqualmie Falls projects differ by magnitude of the flood. In general, the largest simulated changes in stage are seen in the 10-year flood, although changes in the 10- to 100-year floods are quite similar. Flow and stage hydrographs at four downstream locations for five simulated flood recurrences (2-, 10-, 50-, 100-, and 500-year) are presented in an appendix to this report.

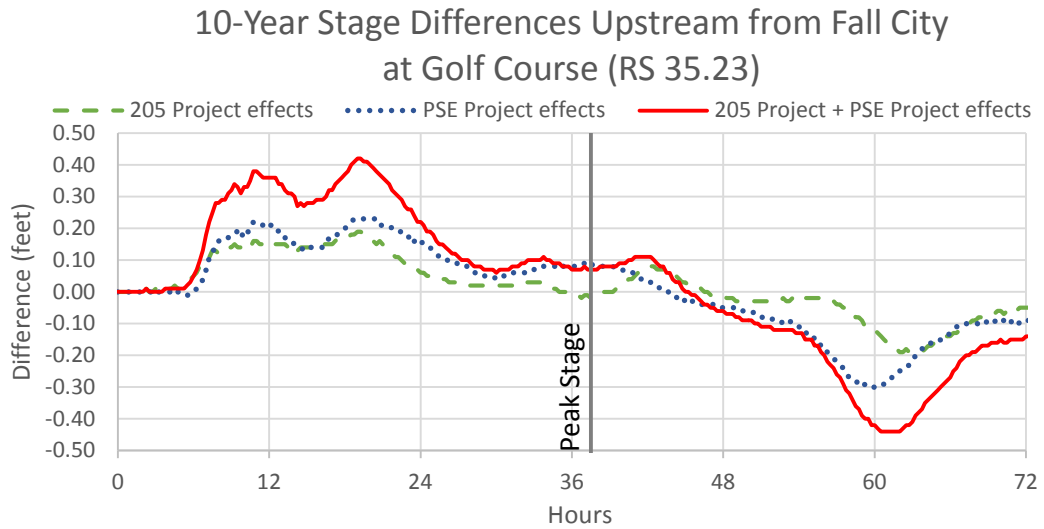

Figure ES-4. Changes in Simulated Water Surface Elevation in the Snoqualmie River Upstream of Fall City between the Snoqualmie Falls and Twin Rivers Golf Courses for the 10-Year Flood.

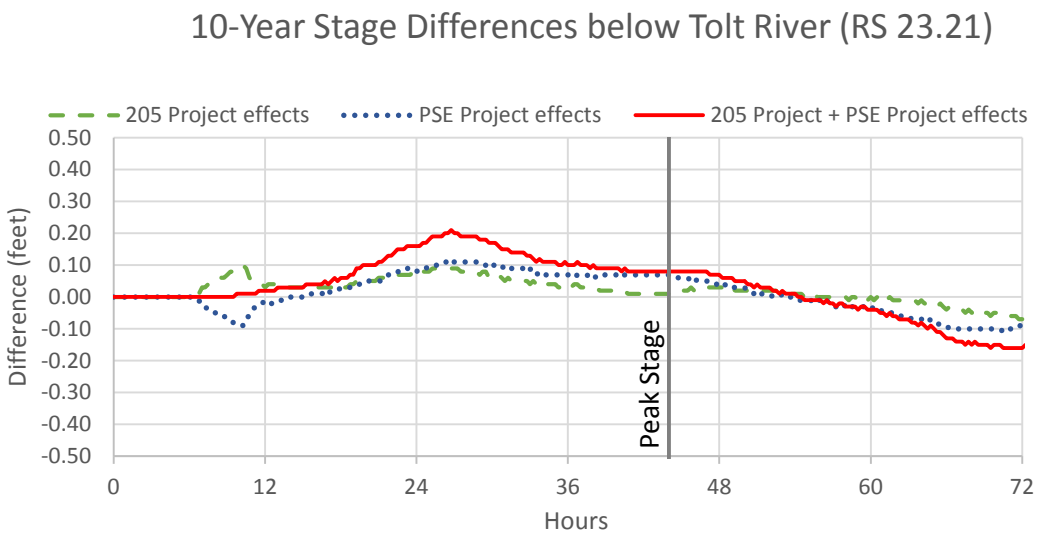

Figure ES-5. Changes in Simulated Water Surface Elevation in the Snoqualmie River Below the Tolt River Confluence for the 100-Year Flood.

By increasing conveyance and reducing flood storage upstream of Snoqualmie Falls, the 205 and PSE projects reduce upstream flooding and flow attenuation. Hydraulic modeling of the 100-year flood event shows that at the peak of the flood, the two projects collectively reduced total flood storage volume by approximately 2,500 acre-feet upstream of the Falls. This volume represents approximately 13 percent of the 18,600 acre-feet of flood storage above the Falls that existed in the Pre-Projects conditions and about 1 percent of the 250,000 acre-feet of flow passing the Falls during the simulated 100-year flood. While the reduction in flood storage upstream of the Falls is significant, the downstream effects of this change are fairly limited because floodplain storage downstream of the Falls is far greater than that upstream of the Falls. The estimated floodplain storage between the city of Duvall and the Falls in a 100-year flood event is 103,450 acre-feet. The simulated loss of flood storage capacity upstream of the Falls is therefore 2 percent of the total storage between Snoqualmie and Duvall.

The effects of the 205 and PSE projects on simulated peak 100-year flood water levels at locations upstream and downstream from Snoqualmie Falls are summarized in Table ES-1. As shown in Table ES-1, both projects reduce water levels upstream of the Falls in the city of Snoqualmie (the reporting location is near Riverview Park) but the effect of the PSE Project is more pronounced. At the PSE weir and downstream of the Falls, the 205 Project does not result in very large changes in peak stages (although changes at other points in the hydrograph are evident, as shown in Figure ES-3). The PSE Project has a greater effect on peak stages at the PSE weir and downstream of the Falls. Comparable peak stage information for the 10-year flood is summarized in Table ES-2. A comparison of Tables ES-1 (100-year flood event) and ES-2 (10-year flood event) shows that changes in downstream peak stages are similar, although slightly greater in the 10-year flood.

Table ES-1. Simulated Change in Peak Stages for the 100-Year Flood.							
		Stage (feet NAVD88) or Change in Stage (feet)					
	River Station	Pre-Projects	Post-205		Post-205 and Post-PSE		
Location	(mile)	Stage	Stage	Change	Stage	Change[a]	Change[b]
City of Snoqualmie	40.65	425.65	425.20	-0.45	424.24	-0.96	-1.41
PSE Weir	39.14	417.97	418.01	0.04	414.16	-3.85	-3.81
Fall City Near Golf Courses	35.23	103.62	103.64	0.02	103.70	0.06	0.08
Patterson Creek	28.00	80.93	80.95	0.02	81.02	0.07	0.09
Below Tolt River Confluence	23.21	74.11	74.12	0.01	74.19	0.07	0.08
City of Duvall	12.40	52.56	52.56	0.00	52.56	0.00	0.00

Notes:

NAVD88 – North American Vertical Datum of 1988

[a] The first column labeled "Change" under the Post-205 and Post-PSE results refers to the change due solely to the PSE Project.

[b] The second column labeled "Change" refers to the combined effects of the 205 Project and the PSE projects.

Table ES-2. Simulated Change in Peak Stages for the 10-Year Flood.							
		Stage (feet NAVD88) or Change in Stage (feet)					
	River Station	Pre-Projects	Post-205		Post-205 and Post-PSE		
Location	(mile)	Stage	Stage	Change	Stage	Change[a]	Change[b]
City of Snoqualmie	40.65	421.04	420.50	-0.54	419.40	-1.10	-1.64
PSE Weir	39.14	414.39	414.40	0.01	410.81	-3.59	-3.58
Fall City Near Golf Courses	35.23	101.48	101.47	-0.01	101.55	0.08	0.07
Patterson Creek	28.00	78.15	78.18	0.03	78.25	0.07	0.10
Below Tolt River Confluence	23.21	71.74	71.76	0.02	71.82	0.06	0.08
City of Duvall	12.40	48.68	48.68	0.00	48.69	0.01	0.01

Notes:

NAVD88 – North American Vertical Datum of 1988

[a] The first column labeled "Change" under the Post-205 and Post-PSE results refers to the change due solely to the PSE Project.

[b] The second column labeled "Change" refers to the combined effects of the 205 Project and the PSE projects.

The 2-year flood event was also analyzed for this study because flows of that magnitude can cause flooding in some areas of the lower valley, and they occur with much greater frequency than major floods. For the 2-year flood, the 205 Project reduced the simulated maximum water surface elevation in Snoqualmie (near Riverview Park) by 1.0 foot and the PSE Project further reduced the simulated maximum water level by 1.2 feet for a total reduction of 2.2 feet at that location. At the peak of the 2-year flood event, simulated flows at the PSE weir were essentially unchanged (less than 1 percent difference).Downstream of the Falls, that same pattern is generally seen in the 2-year flood model results, with barely any change in flow at the peak of the hydrograph. However, as with the larger flood events modeled, the 205 and PSE projects result in increased flow rates downstream of the Falls on the rising limb of the 2-year flood event hydrograph compared to Pre-Projects conditions. The greatest increase in simulated flow resulting from the Snoqualmie Falls projects on the rising limb of

the 2-year flood is about 1,000 cfs (upstream of Fall City between the Snoqualmie Falls and Twin Rivers golf courses). For comparison, the increase in flow at the peak of the 2-year flood event is only about 200 cfs at that location. The largest increase in simulated peak 2-year flood stage (0.06 foot) downstream of the Falls is seen downstream of the Tolt River confluence.

1. INTRODUCTION

The purpose of this report is to assess and summarize changes in flow and water surface elevation in the Snoqualmie River since completion of two flood control projects on the river upstream from Snoqualmie Falls (the Falls) in King County, Washington. A project location map is provided in Figure 1. Constructed between 2004 and 2012, the projects were intended to reduce flooding in the city of Snoqualmie, which lies approximately 1 mile upstream of the Falls.

The first project — the Snoqualmie Flood Damage Reduction Project (known as the "205 Project") — was completed jointly in 2004 by the US Army Corps of Engineers (USACE), King County (County), and the City of Snoqualmie. It included three elements upstream from the Falls to reduce flow restrictions, as shown on Figure 2:

- Widening of the right bank[1] of the Snoqualmie River for about 340 linear feet just upstream from one of the intake structures for the hydroelectric facility owned and operated by Puget Sound Energy (PSE) (the Plant 2 intake structure)
- Widening of the left bank for about 475 linear feet just downstream from the State Route (SR) 202 Bridge
- Removal of an abandoned railroad trestle that protruded into the river channel about 0.5 mile upstream from the SR 202 Bridge

The second project was PSE's Snoqualmie Falls Project (the PSE Project). The in-channel work for the PSE Project started in 2010 and was completed in 2012. During this time, PSE modified the weir that spans the Snoqualmie River just upstream from the Falls by adding 37 feet of weir length and lowering the elevation of the entire weir crest by 2 feet (Figure 2). To accommodate the extended weir, PSE also widened and excavated the left bank for about 500 linear feet upstream of the weir.

In removing flow constrictions (for example, by widening the river channel and lowering the weir height) the two projects reduced the ability of the river channel to hold back flood flow in upper reaches ("flood storage") and to slow the flow of floodwater ("flow attenuation"). In 2014, the County decided to quantify these hydraulic effects of the constructed projects, a decision spurred in part by questions from downstream residents about the extent to which the 205 and PSE projects exacerbated the effects of three large floods between November 2006 and January 2009 (see Section 4). These concerns were expressed again following flooding in January 2015, shortly after this study had been initiated. Specifically, the County wanted to evaluate attenuation lost due to the projects and quantify resulting effects on downstream flows and flooding (as measured by water surface elevations).

[1] "Right bank" refers to the bank on the right side of the river, looking downstream. "Left bank" likewise refers to the bank on the left side of the river, looking downstream.

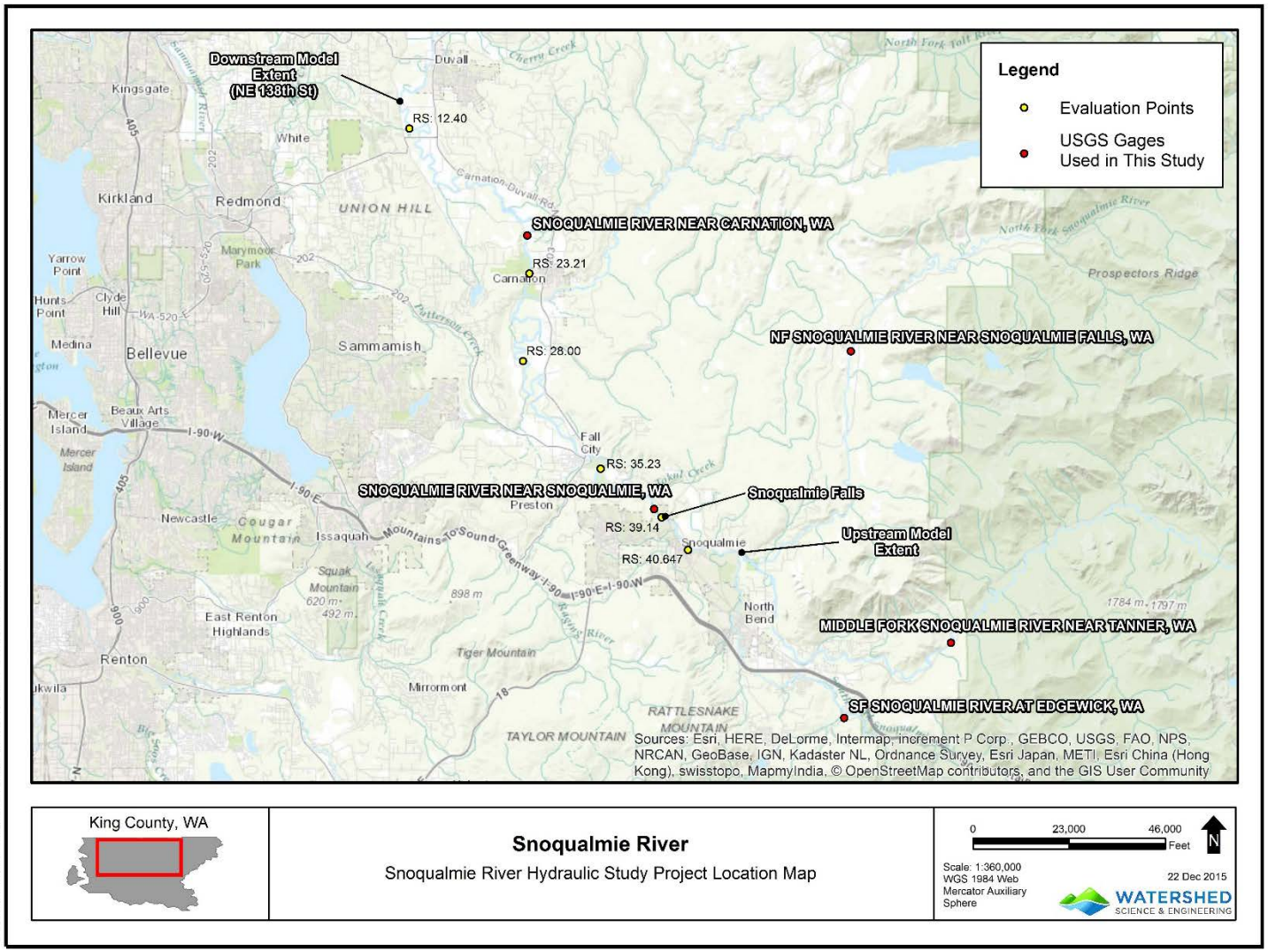

Figure 1. Snoqualmie River Hydraulic Study Project Location Map.

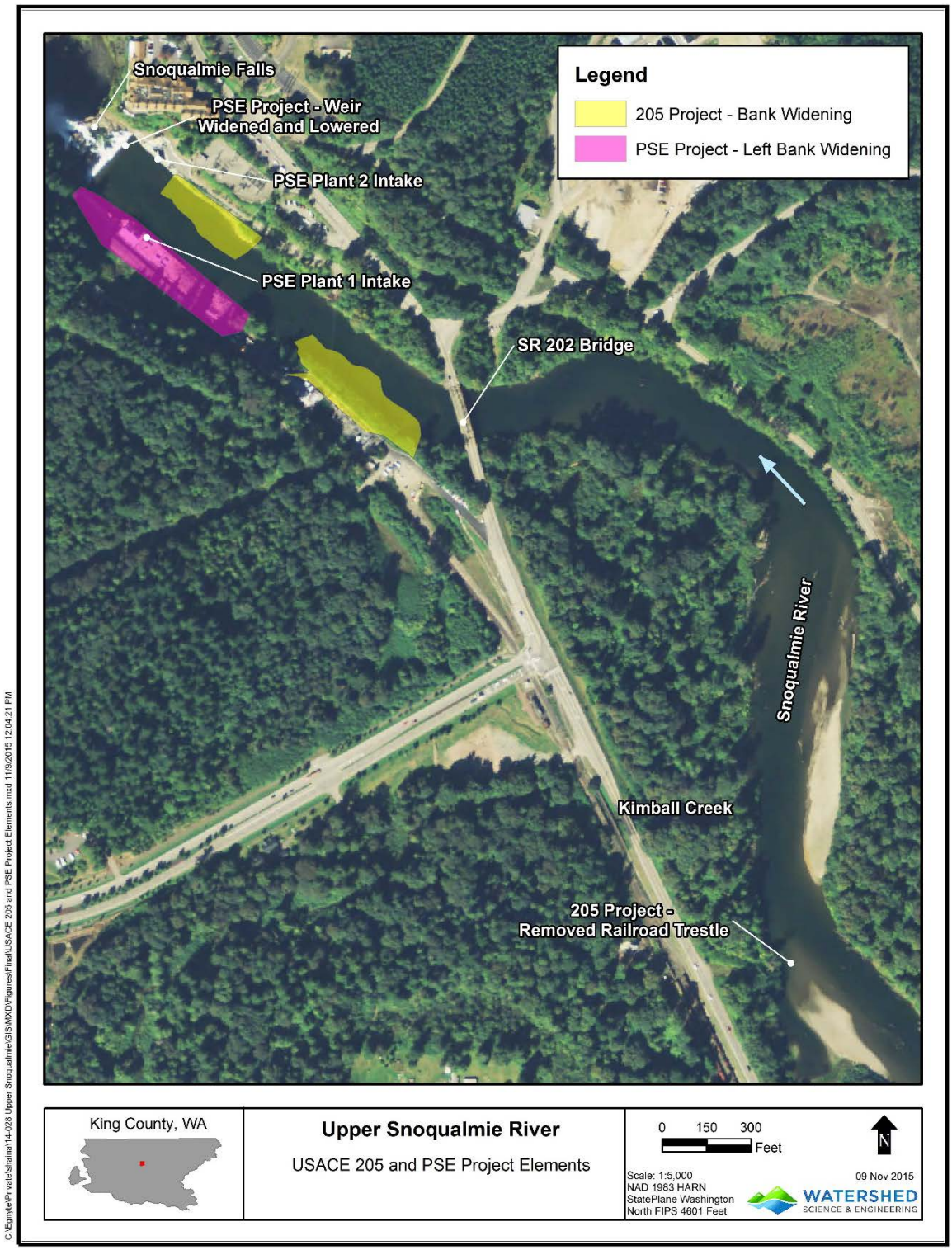

Figure 2. USACE 205 and PSE Project Elements for Snoqualmie Falls Projects.

Accordingly, a numerical hydraulic model of the Snoqualmie River was developed, calibrated, and applied. The model was developed using the USACE Hydrologic Engineering Center River Analysis System (HEC-RAS), which is the software of choice for most riverine flood studies in the United States, including most floodplain mapping studies. Approximately 26 miles of the river were modeled. The upstream limit for the model is just downstream of the confluence of the Middle and South Forks of the Snoqualmie River (river mile [RM] 42.5).[2] The downstream limit for the model is near the city of Duvall, at about Northeast 138th Street (RM 16) (see Figure 1).

Although the model extents are based on existing models, the analyses reported here were evaluated to confirm that the model extents were sufficient to fully describe the downstream hydraulic effects of changes at the Falls. The downstream end of the study area may experience hydraulic backwater effects due to tides (at low flow conditions) and high flows in the Skykomish River[3] (during floods). Puget Sound tides do not affect water levels in the project area during floods; Skykomish River conditions were assumed to be the same for all modeled conditions and therefore do not have any effect on the results reported herein.

Three scenarios were modeled in HEC-RAS: conditions before either project was implemented (Pre-Projects), conditions after the 205 Project was built but before the PSE Project was constructed (Post-205 Project), and conditions after both projects were completed (Post-Projects). The model was run for each condition to allow simulation of the projects' effects on both conveyance capacity and flood storage.

This report describes site investigations (Chapter 2), the modeling approach (Chapter 3), hydrologic input data (Chapters 4 and 5), hydraulic model development and calibration (Chapters 6 and 7), and results (Chapter 8). References cited are listed at the end of the report. Additional detail is provided in appendices. An independent technical reviewer has examined this report to ensure that it provides a reasonable and accurate assessment of the effects of the projects on downstream flooding.

A subsequent phase of work, currently underway as of the time this report was written, will assess whether flood hydrology in the Snoqualmie watershed has changed over time, and, if so, the nature and extent of changes, as well as the level of confidence in the findings. This second phase of investigation will consider trends in peak annual flows and changes in hydrology for smaller but still damaging events (e.g., springtime out-of-bank flows). The work will evaluate key factors that may be influencing changes in Snoqualmie River flood hydrology, including climate change, forest practices, land development patterns, and stormwater management standards.

[2] River miles as measured from the confluence with the Skykomish River.

[3] The Snoqualmie and Skykomish rivers join near Monroe to form the Snohomish River, which enters Puget Sound in the vicinity of Everett.

2. Site Description

About 1 mile downstream from the city of Snoqualmie, the Snoqualmie River drops 268 feet over Snoqualmie Falls. PSE utilizes this natural change in river water surface elevation to generate electricity in two powerhouses. A channel-spanning diversion weir is located about 140 feet upstream from the Falls, creating a backwater that helps divert flows into the facility (PSE Plant 1 Intake on the left bank and Plant 2 Intake on the right bank).

The city of Snoqualmie experiences frequent flooding from the Snoqualmie River. Two recent projects have helped to reduce that flooding. The first of these, the 205 Project, widened the channel along the right bank just upstream from the PSE intake structure to Plant 2, widened the channel along the left bank just downstream from the SR 202 Bridge, and removed an abandoned railroad trestle that partially spanned the channel about 0.5 mile upstream from the SR 202 bridge. The locations of these project elements are shown on Figure 2, with photos taken during a site visit on February 24, 2015, shown in Figures 3 through 5. Additional site photos and descriptions of information sources used to characterize existing site conditions are presented in Watershed Science and Engineering (WSE; 2015a).

Figure 3. Looking Downstream at Location where Right Bank Was Widened as Part of the 205 Project to Reduce Flow Restriction Just Upstream from PSE Plant 2 Intake Structure (Feb. 24, 2015).

Figure 4. Looking Downstream from SR 202 Bridge at Location where Left Bank Was Widened as Part of the 205 Project to Reduce Flow Restriction (Feb. 24, 2015).

Note: The red circle is drawn around trestle remnants on the right bank.

Figure 5. Looking from Left Bank Across to Right Bank at Location where Railroad Trestle Was Removed as Part of the 205 Project (Feb. 24, 2015).

The PSE Project subsequently lowered and widened the weir and rebuilt the intake structures. The weir was lowered by 2 feet and widened by 37 feet to a total length of 253 feet. The intake structure to Plant 2, on the right bank, was reconstructed in essentially the same location, with very little change in the river due to that element of the project. Some modifications were also made to the right bank between the intake structure and the weir. More significant work occurred along the left bank near the intake for Plant 1. The left bank was widened, excavated, and protected (e.g., by riprap and retaining walls) for a distance of about 500 feet upstream of the weir to create a gradual transition into the widened weir. The river channel downstream of the weir was also widened for a short transition distance. Photos of the PSE Project area taken during a site visit on February 24, 2015, are shown in Figures 6 through 8.

Figure 6. Looking from Left Bank Across to Right Bank at New PSE Weir, which is 37 Feet Wider and 2 Feet Lower than Former Weir (Feb. 24, 2015).

Figure 7. Looking Upstream along Left Bank Showing Bank Excavation and New Retaining Wall that Replaced the Former Wood Crib Wall (Feb. 24, 2015).

Figure 8. Looking Downstream along Left Bank at New Retaining Wall, Riprap Slope, and Intake Structure for Plant 1 (Feb. 24, 2015).

3. MODELING APPROACH

To simulate flow attenuation due to the 205 and PSE projects, an "unsteady"[4] hydraulic model is required. An unsteady model simulates an entire flood event (i.e., a hydrograph[5]) as opposed to simply a single, steady-state flow. By simulating the full hydrograph, the model can account for storage in the channel and floodplain, and the resulting reduction in flow passing downstream. It can also simulate the effects of physical changes to the river and floodplain such as the 205 and PSE projects and the resulting differences in flow attenuation.

The one-dimensional hydraulic model HEC-RAS allows simulation of flow in a channel and its overbank areas, including secondary flow paths, and is used in most Federal Emergency Management Agency (FEMA) floodplain mapping studies. HEC-RAS was used for the 2006 floodplain mapping study of the lower Snoqualmie River (Northwest Hydraulic Consultants [NHC], 2006), and also for the 2009 Letter of Map Revision (LOMR) Study for the City of Snoqualmie (Snoqualmie, 2009). For both reaches, these models are currently considered "best available data" by local, state, and federal agencies.

HEC-RAS models consist of two main components: a flow file and a geometry file. The flow file defines boundary conditions, including hydrologic inputs (inflows) to the model. For this analysis, hydrographs corresponding to the 2-, 10-, 50-, 100-, and 500-year return period flood events[6] as well as the January 2009 and January 2015 flood events, were developed as input to the HEC-RAS model.

The geometry file describes physical characteristics of the river. These physical characteristics include the topography of the channel and floodplain (as represented by cross-sections), the length of the channel and overbank flow paths (i.e., reach lengths), elevation differences between cross-sections, channel and overbank hydraulic roughness, structures such as bridges and culverts, and other physical parameters.

Three separate HEC-RAS geometries were developed: one each for Pre-Projects, Post-205 Project, and Post-Projects conditions. The geometries describe the river from the confluence of the Middle and South Forks of the Snoqualmie River at the upstream end of the model to near Northeast 138th Street between the cities of Carnation and Duvall at the downstream end. This coverage allows simulation of the effects of the 205 and PSE projects on flood storage and flow attenuation upstream of the Falls, and the effects of the change in storage on peak flows and water surface elevations in the lower Snoqualmie Valley.

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[4] In this context, "unsteady" means that the flow rate changes with respect to time as characterized by a flow hydrograph.

[5] A hydrograph is a graph showing the rate of flow (or stage) at a specific point in a channel or floodplain versus time.

[6] The return period reflects the average recurrence at which a flood of that magnitude is likely to recur. For example, a flood with a 10-year return period will, on average, occur once in 10 years, but is not guaranteed to occur in any given 10-year period, nor to occur only once within any 10-year period. A different and perhaps more accurate, way of stating this is that a 10-year flood has a 1 in 10 (or 10 percent) chance of occurring in a given year and a 100-year flood has a 1 in 100 (or 1 percent) chance of occurring in any given year.

4. FLOOD FREQUENCY ANALYSIS

Five US Geological Survey (USGS) flow gages (shown on Figure 1) were used to evaluate hydrology for the study reach. USGS gages 12144500 (Snoqualmie River near Snoqualmie) and 12149000 (Snoqualmie River near Carnation) are located downstream of the 205 and PSE projects and downstream of Snoqualmie Falls. Upstream from the Falls, the USGS operates gages on the North, Middle, and South Forks of the Snoqualmie River (gages 12142000, 12141300, and 12143400, respectively). For purposes of this study, flows from the three gages upstream of the Falls were aggregated into a single composite record representing the flows upstream of, and thus unaffected by, the 205 and PSE projects. This composite flow record is termed the "Sum of the Forks" for this study; a full description of the derivation of this record is provided in Appendix B.[7] Flow frequency analysis was conducted for the two downstream gages and the Sum of the Forks to provide insight into trends in basin hydrology that might be affecting downstream flooding.

Concurrent with or following the construction of the two projects (spring 2004 to fall 2012), several large floods occurred on the Snoqualmie River, including November 2006, November 2008, January 2009, and January 2015. To evaluate whether the recent flooding is simply a function of natural variability or long-term changes in hydrology independent of the 205 and PSE projects, flood frequencies were analyzed to see if gages upstream and downstream of the project sites exhibited similar trends. Frequency analyses were conducted using USGS annual peak flows for the two downstream gages, and maximum mean daily flows for the Sum of the Forks. Each gage was evaluated for the period from the start of gaging through water year (WY)[8] 2003 and then again for the period from the start of gaging through WY 2013. The analyses could be taken only through WY 2013 because the USGS had not yet published final peak discharges for WYs 2014 and 2015 at the time of this study. Flood frequency analyses were conducted using the guidelines in USGS Bulletin 17B (USGS 1982). The period of record for each gage is listed in Table 1, and the computed peak flows are summarized in Table 2. Data used in the frequency analyses, and more comprehensive results, are provided in Appendix C.

[7] Note that the "Sum of the Forks" as used in this analysis was derived from mean daily flows at three upstream gages rather than from 15-minute gage data because the latter were not available for all years. This methodological approach is different from the Sum of the Forks data developed and used by King County in its Flood Warning Program, which are based on 15-minute data from each gage for recent years. A more complete description of the derivation of the data used in this study is provided in Appendix B.

[8] The water year runs from October through September. Events in October 2013 and September 2014 would both be included in WY 2014.

Table 1. Period of Record for River Flow Gages Used in This Analysis.		
Gage Number	**Gage Name**	**Period of Record (Water Year)**
12149000	Snoqualmie River near Carnation	1930–2013
12144500	Snoqualmie River near Snoqualmie[a]	1959–2013
12143400	South Fork Snoqualmie River above Alice Creek near Garcia	1961–2013
12142000	North Fork Snoqualmie River near Snoqualmie Falls	1930–2013
12141300	Middle Fork Snoqualmie River near Tanner	1962–2013

[a] Note that although this USGS gage is named Snoqualmie River near Snoqualmie, the gage is actually downstream of the City of Snoqualmie, downstream of Snoqualmie Falls, and downstream of the projects evaluated herein.

Table 2. Flow Frequencies for Snoqualmie River Locations.						
	Peak Flow (cfs)					
	Near Carnation (USGS Gage 12149000)		**Near Snoqualmie (USGS Gage 12144500)**		**Sum of the Forks[a]**	
Return Period	**1930–2003**	**1930–2013**	**1959–2003**	**1959–2013**	**1962–2003**	**1962–2013**
2-year	29,800	30,400	30,100	30,000	17,900	18,300
10-year	51,900	54,400	53,100	52,900	30,300	31,800
50-year	71,100	76,500	72,700	72,800	40,200	43,300
100-year	79,200	86,100	80,800	81,100	44,100	48,100
500-year	98,100	109,300	99,300	100,300	52,800	59,300

[a] See Appendix B for a full description of the Sum of the Forks data.

As shown in Table 2, peak flows during the period of record that includes 2003 to 2013 are generally higher than those in the period of recording ending in 2003; this change is evident at all locations investigated, except for a slight decrease (0.4 percent) in the 2- and 10-year flows estimated for the gage on the Snoqualmie River near Snoqualmie. Upstream of the 205 and PSE projects, the 100-year flow has increased by about 9 percent. Immediately downstream of the projects, at the Snoqualmie River near Snoqualmie gage, the 100-year peak flows are least changed, showing an increase of only 0.4 percent. Farther downstream at the Carnation gage, the 100-year flow shows an increase of almost 9 percent. The incremental difference between the frequency analysis results for the two periods of record increases as the return period increases.

The magnitude and water year of the 10 highest annual peak flows at each location are listed in Table 3. As seen in Table 3, the January 2009 (WY 2009) and November 2006 (WY 2007) flood events are the two largest events recorded at the Carnation gage; the same events are in the top three at the Sum of the Forks, and the top five at the Snoqualmie River near Snoqualmie gage. A complete ranking of peak flows for each location is provided in Appendix C.

Table 3. Ten Highest Observed Annual Peak/Annual Maximum Flows at Two Downstream Gages and at the Sum of the Forks.[a]						
	Carnation Gage 12149000		Snoqualmie Gage 12144500		Sum of the Forks[b]	
Rank	Water Year	Annual Peak Flow (cfs)	Water Year	Annual Peak Flow (cfs)	Water Year	Maximum Mean Daily Flow (cfs)
1	2009	82,900	1991	78,800	2009	44,210
2	2007	71,800	1960	61,000	1991	39,570
3	1991	65,200	2009	60,700	2007	36,730
4	1996	61,600	1987	58,100	1996	36,140
5	1932	59,500	2007	55,000	1976	32,120
6	1933	59,000	1978	53,800	1978	31,130
7	1987	57,100	1976	51,800	2005	29,730
8	1951	52,200	1996	51,700	1987	27,200
9	1976	52,100	1975	48,100	2011	27,140
10	2011	51,600	1990	44,000	1982	25,930

[a] This table only reflects data through water year 2013, the latest year for which quality-controlled data were available from the USGS at the time of the analysis.

[b] Note that the Sum of the Forks data developed and used in this study were derived from mean daily flows because 15-minute data were not available for all historical events. These data are, therefore, slightly different from the Sum of the Forks data developed and used by King County for its Flood Warning Program, which is based on 15-minute data derived from recent events. A full description of the data developed for this study is provided in Appendix B.

Regardless of the period of record for flow frequency analyses, results always have associated uncertainties. Uncertainties arise from factors such as the differences in the period of gaging; missing data for particular historical high flows; errors or uncertainties in the observed flow data; and changes in flows over time due to logging, land development, changes in channel characteristics (e.g., sedimentation, levees), climate change, and similar factors. The greater the return period for which a flow frequency estimate is made, the greater the uncertainty. Estimates of the 100-year or 500-year flow are particularly uncertain as these require extrapolation beyond the available data record.

These uncertainties can also affect the reliability of comparisons between different gage sites. For example, the gages along the Snoqualmie River have different historical periods of record. Two of the ten largest peak flows at Carnation occurred in 1932 and 1933, years not reflected in the periods of record for the other gages. At the Snoqualmie River near Snoqualmie gage, the November 1959 flood (WY 1960) is the second largest event on record; however, data for that event are not available at the Sum of the Forks; and the event is not among the top 10 at the Carnation gage. All three locations include the November 1990 flood (WY 1991) in their record. While this event is the largest at the Snoqualmie River near Snoqualmie gage (by almost 30 percent), it is ranked as only the second or third largest event at the other locations. Considering these variations, and the fact that the peak flow estimate for the November 1990 event at Snoqualmie River near Snoqualmie gage was made using indirect methods having lower than the standard accuracy (post-flood estimates using high water marks [HWMs] and a slope-area calculation), comparisons of flow frequency results among the gages should be made with caution.

However, despite the acknowledged uncertainties, it is reasonable to use the observed historical data to evaluate trends in flows over time. As described above, although flow frequencies downstream of the 205 and PSE projects have increased in recent years, the same level of peak flow increase is also seen upstream of the projects. Further conclusions regarding the effects of any particular change (such as the 205 or PSE project) based strictly on the observed flow data alone would be speculative at best. For that reason a hydraulic model was developed and applied, as described in Chapters 5 and 6, to conduct a detailed evaluation of the hydraulic effects of the two projects.

5. Hydraulic Model Flow Inputs

Based on the flood frequency analysis described in Section 4, it is not possible to conclude that the 205 and PSE projects have caused an increase in downstream flows, as similar increases in peak flows are seen both upstream and downstream from the projects. However, the analysis of peak flows in and of itself does not provide sufficient detail to evaluate the effects of the projects on downstream water levels. That evaluation requires development and application of an unsteady flow hydraulic model. The development of inflow data for the model is described below.

5.1. Flows Used in Previous Modeling Efforts

To evaluate differences in water surface elevations that may be caused by the 205 and PSE projects, a HEC-RAS model was developed for the reach extending from the confluence of the South Fork and Middle Fork Snoqualmie River downstream to approximately Northeast 138th Street between Carnation and Duvall. The model for the study reported in this document combined two previously developed HEC-RAS models, one for upstream of Snoqualmie Falls and one for downstream of the Falls.

The model for the reach upstream of Snoqualmie Falls was originally developed for the City of Snoqualmie's LOMR to FEMA (Snoqualmie, 2009). That model included changes to the Snoqualmie River channel from the 205 Project, but did not include any of the PSE Project elements. The LOMR model was a steady-state HEC-RAS model that used the hydrology from the effective 2005 FEMA Flood Insurance Study (FIS) for the Snoqualmie River near Snoqualmie gage (FEMA, 2005). The flows used in the LOMR model are listed in the middle column of Table 4. The 2005 FIS flows are based on a shorter period of record (1959 to 1978) and thus differ from the flows used in the 2006 FIS listed in Table 2.

Table 4. Inflows at Snoqualmie River Near Snoqualmie Used in Previous Modeling.

Return Period	Effective 2005 FIS[a] Flow (LOMR) (cfs) (upstream of Falls)	Preliminary 2006 FIS[b] Flow (cfs) (downstream of Falls)
10-year	52,300	51,700
50-year	71,000	71,000
100-year	78,500	79,100
500-year	95,500	95,200

[a] FEMA, 2005.

[b] NHC, 2006.

The model for the reach downstream of Snoqualmie Falls was originally developed for a 2006 FIS for the area between Snoqualmie Falls and the confluence of the Snoqualmie and Skykomish rivers (NHC, 2006). The model is an unsteady-state HEC-RAS model, simulating flow hydrographs rather than simply the peak flow. The cross-section farthest upstream in the 2006 model is located at the Snoqualmie River near Snoqualmie gage. The 100-year inflow at

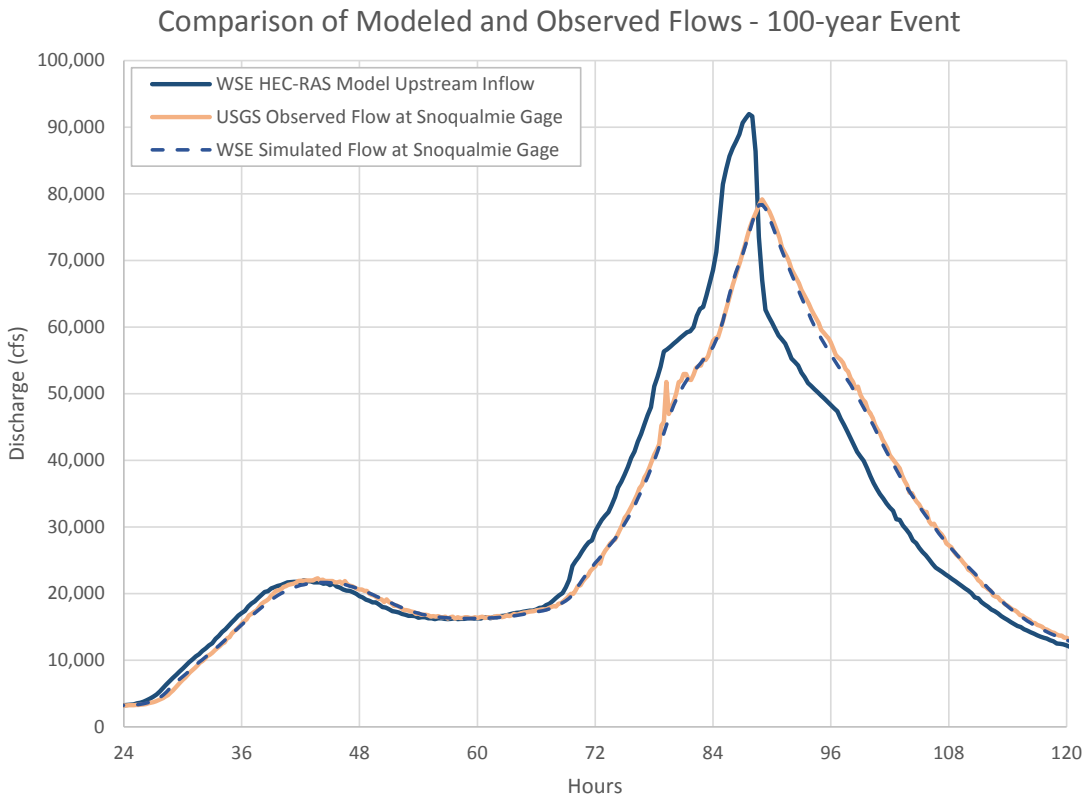
that location was patterned on the USGS observed hydrograph for the November 1990 flood and reduced slightly to match the 100-year peak flow of 79,100 cfs as calculated at that time based on the period of record available then. The 50-year and 500-year inflows for the lower Snoqualmie study used the same pattern hydrograph scaled to match the corresponding peak flows. The 10-year model inflow used a scaled version of the hydrograph for the December 1977 flood, which was closer in magnitude to a 10-year event. Peak flows for each of the upstream inflow hydrographs used in the 2006 FIS model are listed in right-hand column of Table 4.

5.2. Model Inflows for This Study

The purpose of the hydraulic modeling in this study is to evaluate the effect of changes in flow conveyance and flood storage due to the projects on downstream flows and water surface elevations. The baseline HEC-RAS model developed for the present study simulates conditions prior to the 205 and PSE projects. The model was then modified to simulate the Post-205 Project condition and the Post-Projects condition.

The unsteady hydraulic analysis requires an inflow hydrograph at the upstream boundary of the model. This hydrograph was developed using a trial-and-error procedure to match the 2006 FIS (NHC, 2006) flows at the Snoqualmie River near Snoqualmie gage. Flow inputs at the upstream end of the model were adjusted and the flows were iteratively routed to the Snoqualmie gage using the model until the hydrograph at the gage matched the 2006 FIS flow hydrograph. This ensures consistency with the 2006 lower Snoqualmie River study (NHC, 2006). The hydraulic routing analysis was performed using the Post-205 Project condition model, as that model matches the conditions in place above the Falls at the time of the 2006 FIS. The scaled observed and routed hydrographs for the 100-year event at the Snoqualmie River near Snoqualmie gage are compared in Figure 9, along with the final synthesized upstream inflow hydrograph (solid blue line). Similar model inflow hydrographs for the 2-year, 10-year, 50-year, and 500-year events were also developed to allow evaluation of project-related downstream changes over a range of flows.

The same upstream inflow hydrographs were then used with each of the hydraulic model geometries, thus holding the inflows from upstream of the projects unchanged and allowing the effects of project-related changes to be isolated from other possible hydrologic and hydraulic changes. The different project (model) geometries change how flows are stored and attenuated, resulting in differences in downstream flows. The resulting effects on downstream water surface elevations can then be quantified and compared for each modeled scenario.



Note: Upstream inflow to the model is at the confluence of the South Fork and Middle Fork of the Snoqualmie River.

Figure 9. Snoqualmie River 100-Year Hydrographs at Upstream Inflow (for All Model Conditions) and at USGS Gage Near Snoqualmie (for Post-205 Conditions).

6. Hydraulic Model Development

Previously developed HEC-RAS models, which cover the extent of the area potentially affected by the Snoqualmie Falls projects, were used as the basis for the three new HEC-RAS geometries developed for this project (i.e., for Pre-Projects, Post-205 Project, and Post-Projects conditions). The three geometries were updated using available project design drawings; topographic surveys from 2000, 2005, 2011, and 2014; and aerial photos to confirm the extent of changes. Adjustments were made to the models to accurately simulate unsteady flow conditions and river flow patterns occurring under the different flood conditions. The models were then validated using observed flow data and observed high water marks. Appendix A summarizes the sources of the existing conditions models and sources of data used in developing the Post-205 Project and Post-Projects models described below.

6.1. Existing Models

Three existing HEC-RAS models form the basis for the models developed for this study: one for the river and floodplain downstream of Snoqualmie Falls, and two for the reach upstream of the Falls. The downstream model was originally developed by NHC for King County for the 2006 FEMA FIS of the lower Snoqualmie River (NHC, 2006). The upstream extent of this model is near the USGS Snoqualmie River near Snoqualmie gage below the Falls and the model extends downstream to Northeast 138th Street between Carnation and Duvall. The downstream model extent is about 13 river miles downstream from the Tolt River confluence, which is near Carnation. The FEMA model has been updated several times since 2006 to add farm pads and several large projects involving grading in the floodplain that have been permitted and constructed along the Snoqualmie River over the past few years. The current version of the model includes all projects constructed prior to the beginning of 2014. This updated downstream model was used for three scenarios in the present study in order to hold any downstream topographic or other changes constant for the purpose of isolating and clearly comparing the impacts of the two projects at the Falls.

For the area near the 205 and PSE project sites, the USACE provided a HEC-RAS model of the Snoqualmie River upstream of Snoqualmie Falls for the Pre-Projects condition (hereafter called the USACE Pre-Projects model). The model has a downstream boundary located about 180 feet upstream from the PSE weir and an upstream boundary at the confluence of the Middle and South Forks of the Snoqualmie River. The model geometry includes the channel constrictions between the Falls and SR 202 as well as the old railroad trestle downstream of the city of Snoqualmie that existed prior to the 205 Project. The channel cross-sections from the SR 202 Bridge to the downstream extent of the model match channel survey data from 2000 obtained from the USACE. Upstream from the SR 202 Bridge, the source survey data were not available; the vintage of these latter cross-section data could not be verified.

NHC provided WSE with a HEC-RAS model of the Snoqualmie River upstream of Snoqualmie Falls for the Post-205 Project condition. This is an updated version of the model that was originally developed for the City of Snoqualmie 2009 LOMR request. This model has the same upstream extent as the USACE Pre-Projects model, but extends farther downstream to include the PSE weir and three cross-sections between the weir and Snoqualmie Falls. The layout of model cross-sections and key landmarks are shown in Figure 10. The NHC Post-205 Project model incorporates changes to the geometry resulting from the 205 Project, but does not include changes related to the more recent PSE Project.

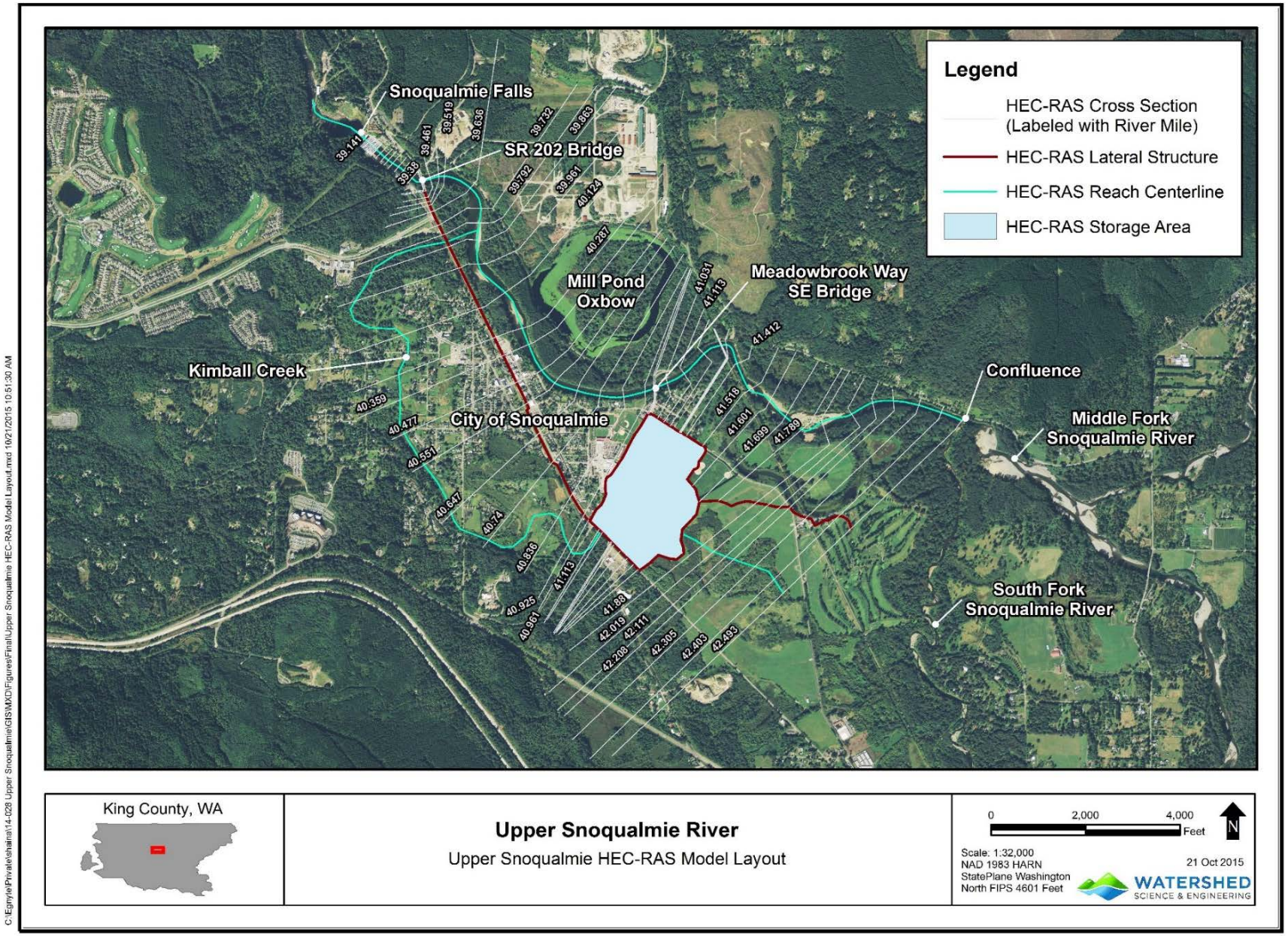


Figure 10. HEC-RAS Model Layout above the Falls Including Key Landmarks and River Mile Stationing.

The NHC Post-205 Project model is an updated version of the model developed for the 2009 LOMR request from the City of Snoqualmie to FEMA (Snoqualmie, 2009). The cross-sections between the weir and SR 202 Bridge were updated by WSE with survey data from 2005; the cross-sections between the SR 202 Bridge and the city of Snoqualmie were updated with 2011 light detection and ranging (LiDAR) data and 2012 in-channel survey data; and the overbank data for some of the cross-sections between the city of Snoqualmie and the upstream model extent were updated with 2011 LiDAR data. For the reach between the city of Snoqualmie and the upstream boundary, this model uses the same in-channel geometry as the USACE Pre-Projects model. Data sources used in this study are documented in Appendix A.

6.2. Development of New Model Geometries

For the present study, cross-sections from the three existing models were modified to support unsteady flow modeling and to better simulate hydraulic complexities in the reach above the Falls. With the exception of a few cross-sections at the junction between the upstream and downstream models, no new cross-sections were added. To the extent possible, in-channel data in each model were updated using available survey data from the period for the condition being modeled. Table 5 lists the project elements included in each model geometry. These are discussed in detail in the following sections.

Table 5. Project Elements Modeled for Each Condition.

Project Elements	Pre-Projects	Post-205 Project	Post-Projects
Weir elevation (feet, NAVD88[a])	400.08	400.08	398.0
Weir length (feet)	216	216	253
Right bank widening upstream of Plant 2	Constriction modeled	Wider channel modeled	Wider channel modeled
Left bank widening downstream of SR 202 Bridge and upstream of Plant 1	Constriction modeled	Wider channel modeled	Wider channel modeled
Abandoned railroad trestle	Included	Removed	Removed
Left bank widening from weir to ~500 feet upstream	Not modeled	Not modeled	Modeled
Channel geometry	2000 survey	2005 survey	2014 survey

[a] NAVD88 – North American Vertical Datum of 1988.

6.2.1. Pre-Projects Geometry

The geometry for the Pre-Projects condition represents the Snoqualmie River upstream of Snoqualmie Falls in about the year 2000. The right bank constriction upstream from PSE Plant 2, the left bank constriction upstream of PSE Plant 1, and the abandoned railroad trestle downstream of the city of Snoqualmie are all included in this geometry. The PSE weir was modeled with a crest elevation of 400.08 feet (North American Vertical Datum of 1988 [NAVD88]).

From the weir to about 180 feet upstream (an area not altered by the 205 Project), the Pre-Projects geometry uses the NHC Post-205 Project model cross-sections. Although the Pre-Projects model represents conditions prior to the 205 Project, the Pre-Projects model

obtained from the USACE does not have any cross-sections in this reach and cross-section surveys preceding the 205 Project were not available. Because this reach was not modified in any way by the 205 Project, the Post-205 data accurately portray conditions prior to the 205 Project. The model cross-section just upstream from the weir was also adjusted slightly to reflect the 216-foot length of the former weir as shown in PSE drawings versus the 201-foot length used in the NHC Post-205 Project model.

From about 180 feet upstream of the weir to the SR 202 Bridge, the Pre-Projects model geometry uses the USACE Pre-Projects model cross-sections. These cross-sections are based on survey data from 2000 and include the former channel constrictions. Upstream of the SR 202 Bridge, the Pre-Projects model uses the channel geometry from the USACE Pre-Projects model, except that the overbank topographic data were updated to match the NHC Post-205 Project model. This refinement was made because the overbank areas in the USACE Pre-Projects model were delineated very coarsely, and the NHC Post-205 Project model incorporates much more detailed topography from 2011 LiDAR survey data. Note that the data used from 2011 are all outside of the channel and, therefore, not physically modified by the projects. Consequently, those topographic data were used to improve accuracy and were held constant in all three scenarios.

6.2.2. Post-205 Project Geometry

The geometry for the Post-205 Project condition represents the Snoqualmie River upstream of the Falls in about the year 2005 and reflects completion of all in-channel work for the 205 Project. Where possible, Post-205 Project model channel geometry is based on survey data from 2005. The Post-205 Project geometry includes the channel widening of the right bank upstream from PSE Plant 2, the channel widening along the left bank upstream of PSE Plant 1, and removal of the railroad trestle. The PSE weir was modeled at an elevation of 400.08 feet (NAVD88), as for the Pre-Projects condition. The Post-205 Project geometry uses the NHC Post-205 Project model cross-sections from the PSE weir to the upstream boundary of the model. The adjustment to the cross-section immediately upstream of the PSE weir to correct for weir length, as described above for the Pre-Projects condition, was also made for the Post-205 Project geometry. Between SR 202 and the city of Snoqualmie, the Post-205 Project geometry replaces the channel topographic data with 2005 survey data collected by PSE, wherever these data are available.

6.2.3. Post-Projects Geometry

The geometry for the Post-Projects condition represents conditions as of 2014 upstream of Snoqualmie Falls. The Post-Projects geometry includes the Post-205 Project geometry, as described above, for all the cross-sections from the PSE weir to the upstream boundary of the model. Geometry changes were made to reflect the new (lowered) elevation of the weir (398 feet NAVD88) and the new longer weir length (253 feet). Because the weir modifications included widening of the left bank area, adjustments from Post-205 Project conditions were made in 16 cross-sections upstream from the PSE weir, based on design drawings provided by PSE. In addition, the Post-Projects geometry replaces channel data for the reach from about 300 feet upstream of the weir to the city of Snoqualmie with 2014 survey data collected by PSE, where available.

6.3. Connecting Upper Basin to Lower Basin

Each of the three model geometries was connected to the existing HEC-RAS model for the Snoqualmie River downstream of Snoqualmie Falls. The models were connected to provide a single, comprehensive tool for use in evaluating the downstream impacts of changes upstream of the Falls associated with the 205 and PSE projects. The upstream extent of the lower basin model is the USGS Snoqualmie River near Snoqualmie gage, and the downstream extent of the upper basin models is the PSE weir above the Falls. The horizontal gap between these boundaries is about 1,600 feet. To fill in part of this gap, five cross-sections were added to the combined model between the Falls and the USGS gage using 2002 survey data. In the absence of bathymetric[9] data for this reach, the channel geometry was approximated using an assumed 100-foot-wide "pilot channel" (which is how HEC-RAS simulates it).

The remainder of the gap—the 160 feet between the Falls and the PSE weir—was ignored in the combined model. This approximation was made after testing alternative approaches, as described below. The NHC Post-205 Project model included three cross-sections between the Falls and the weir. To help stabilize model computations in the present study, the weir alone was explicitly modeled in HEC-RAS as a single structure directly connected to the cross-section at the base of the Falls (i.e., no cross-sections were included between the weir and the Falls). This approach was taken because upstream hydraulics are controlled by the weir, and the channel grade between the weir and the Falls is sufficiently steep to prevent any significant tailwater effects (influences from downstream) on the weir. A sensitivity analysis was performed to test the effects of this simplification on upstream flows and water levels, under both the former weir geometry and the lowered and extended weir. At the 100-year flow, there were no differences in computed upstream water surface elevations with and without cross-sections between the weir and the Falls.

6.4. Modification of the Upstream Model for Unsteady Flow

The two existing upstream models used as a basis for the combined model for this study were developed as steady-state models. However, as noted in Section 3 and elsewhere, the analysis for this study requires unsteady modeling to simulate the effects of the 205 and PSE projects on conveyance capacity and flood storage in the upstream reach of the river.

Certain features in the HEC-RAS geometry need to be handled differently in steady and unsteady simulations. One relevant example for the present analysis is the "ineffective flow limits" on model cross-sections. In a steady-state model, ineffective flow areas can be used to define areas where water will not flow. These areas could similarly be represented using "levee cards" or "blocked obstructions." In a steady-state simulation, these methods would all have similar effect on the results. In an unsteady-state simulation, however, the three methods can produce widely differing results. In an unsteady model, ineffective flow limits define areas where water will not move, but can still be stored. Levee data can be used to define areas where water cannot reach until a predetermined water surface elevation is attained during the unsteady model run, at which point storage and flow conveyance become possible. Blocked obstructions, on the other hand, define areas where neither flow conveyance nor storage occur under any conditions.

[9] Bathymetry is underwater topography.

In unsteady models, failure to define these parameters appropriately can yield erroneous results for storage (or lack thereof) and corresponding effects on downstream flow. All ineffective flow limits, levees, and blocked obstructions in the models used as a basis for the current study were, therefore, reviewed to ensure appropriate configuration for unsteady flow. Consistently for all three model geometries, changes were made as necessary to accurately represent the physical conditions.

The three model geometries were developed and the 100-year flow hydrograph (see Chapter 4) was run through each model. For the Pre-Projects condition, flow overtopped SR 202 or high ground on the left bank of all cross-sections through the city of Snoqualmie. In the Post-Projects condition, maximum water surface elevations are lower through the city of Snoqualmie, and approximately 10 cross-sections in the model do not show overtopping.

The left overbank through the city of Snoqualmie includes several remnant side channels.[10] If flow overtops the banks in this area, it may be conveyed in one of these side channels and not return to the main Snoqualmie River channel for a considerable distance downstream. HEC-RAS cannot not simulate this complex flow pattern if the river is modeled as a single reach (as was done in the USACE and NHC models). To better simulate the complex flow splits that occur in this reach, the model cross-sections from the Kimball Creek confluence upstream to the upper model boundary were divided into two adjacent reaches separated by what HEC-RAS calls "lateral structures"; these structures are features of the natural or built landscape that function to direct flow from one flow path to another. The primary reach includes the main channel and right floodplain, while the other reach represents overflows on the left floodplain generally following the alignment of Kimball Creek. The lateral structures follow SR 202 or the high ground along the overbank area between Kimball Creek and the Snoqualmie River. The lengths and elevations of these lateral structures were measured in GIS[11] from the 2011 LiDAR data.

The lateral structures allow HEC-RAS to calculate how much water flow leaves or returns to the main stem Snoqualmie River channel from the left floodplain between each pair of cross-sections. When configured with adjacent reaches connected by lateral structures, HEC-RAS can simulate different water surface elevations in the left overbank area and the main stem channel of the Snoqualmie River as well as the exchange of water between them. This enhancement allows more accurate simulation of the hydraulic conditions under each project, including changes in overflows to Kimball Creek, flooding of side channels, and the resulting storage effects on downstream flows.

Because of the complexity of flow conditions on the left overbank upstream from the intersection of Meadowbrook Way Southeast and Railroad Avenue in Snoqualmie, a storage area was added to the model in this location (see Figure 10). Flows in this area can follow different paths depending on relative differences in water levels (stages) between the river and the floodplain. The storage area allows the model to capture these localized two-dimensional flow patterns within the one-dimensional HEC-RAS formulation. Lateral structures provide connectivity between the main channel and this storage area.

[10] Remnant side channels are former locations of the main Snoqualmie River channel abandoned at some time in the past.

[11] Geographic Information System – a mapping and analysis tool used for spatial data

7. HYDRAULIC MODEL CALIBRATION AND VALIDATION

Numerical hydraulic models are most useful when they have been "tuned" (or calibrated) to match a set of high water mark observations and then tested (or validated) with an independent set of observed data. The HEC-RAS models were run for two recent flood events and compared to observed HWMs to demonstrate that they adequately simulate existing conditions. The Post-205 Project model was calibrated using HWMs from the January 2009 flood event cited in Section 4. The calibrated model parameters were then validated by comparing Post-Projects simulations to HWMs from the January 2015 flood event. It was not necessary to calibrate or validate the existing downstream model because it had been calibrated for the FEMA (2006) FIS and was used in the present study without modification.

7.1. January 2009 Flood

The January 2009 flood had a peak flow of 60,700 cfs at the Snoqualmie River near Snoqualmie gage (see page C-11 in Appendix C). This flow is about 900 cfs less than a 20-year return period event based on the historical flow record at that gage. The USGS collected HWMs upstream from Snoqualmie Falls for this event (Mastin et al., 2010). The locations and elevations of these data are shown in Appendix D.

An upstream inflow hydrograph for the HEC-RAS model corresponding to the January 2009 flood event was developed, as described in detail by WSE (2015c). At the time of that flood, the 205 Project had been completed but construction of the PSE Project had not yet begun. The upstream model inflow hydrograph was defined such that the routed flow at the USGS Snoqualmie River near Snoqualmie gage from the Post-205 Project HEC-RAS model approximated the observed USGS flow record for the January 2009 flood. Modeled maximum water surface elevation results for this event were then plotted against the observed HWMs, as shown in Figure 11.

Notwithstanding the significant scatter in HWMs reported by the USGS for this flood event, simulated maximum water surface elevations from the Post-205 Project model were generally slightly above the reported values in two reaches: between the PSE weir and SR 202 Bridge (RM 39.1 to 39.4), and upstream from the Meadowbrook Way Southeast Bridge (RM 40.95). Initial model results for the intervening reach (i.e., between the SR 202 Bridge and the Meadowbrook Way Southeast Bridge) were lower than reported HWMs. To help improve the calibration, Manning's roughness coefficients ("n" values) for the channel and overbank areas were increased between these two bridges. The results that appeared to best fit the measured HWMs are based on Manning's n-values in the between-bridges reach that are 10 percent higher than those used in the previous modeling. The final Manning's n-values in this reach are 0.0385 for the channel and from 0.132 to 0.187 in the overbank areas. These values are considered reasonable given the conditions through the study reach. The weir

coefficient[12] for the PSE weir was not changed from the previously modeled (NHC, 2006) value of 3.2. Discussion of the reasonableness of this calibration for the purposes of this study is provided below in Sections 7.3 and 7.4

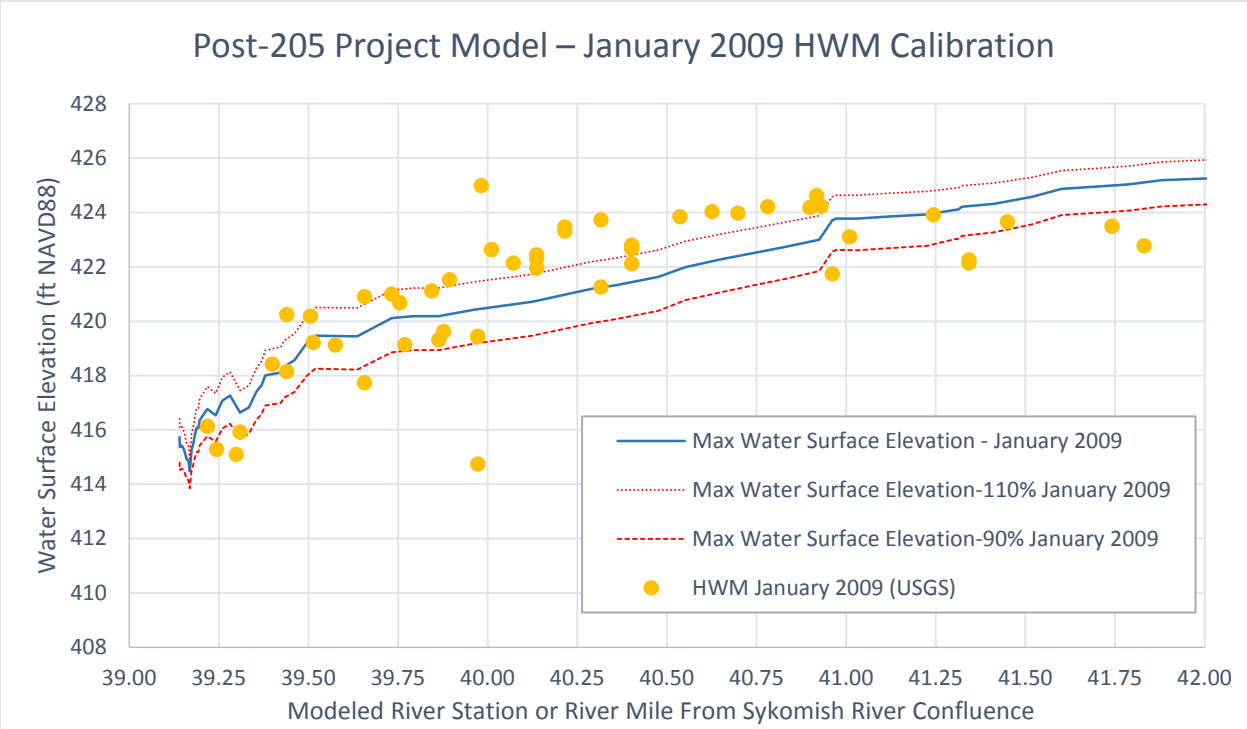


Figure 11. Post-205 Project Modeled Maximum Water Surface Elevations for January 2009 Flood.

The same changes to the calibrated n-values were applied to the Pre-Projects and the Post-Projects model geometries, and the latter was then used to validate the modeling as described below.

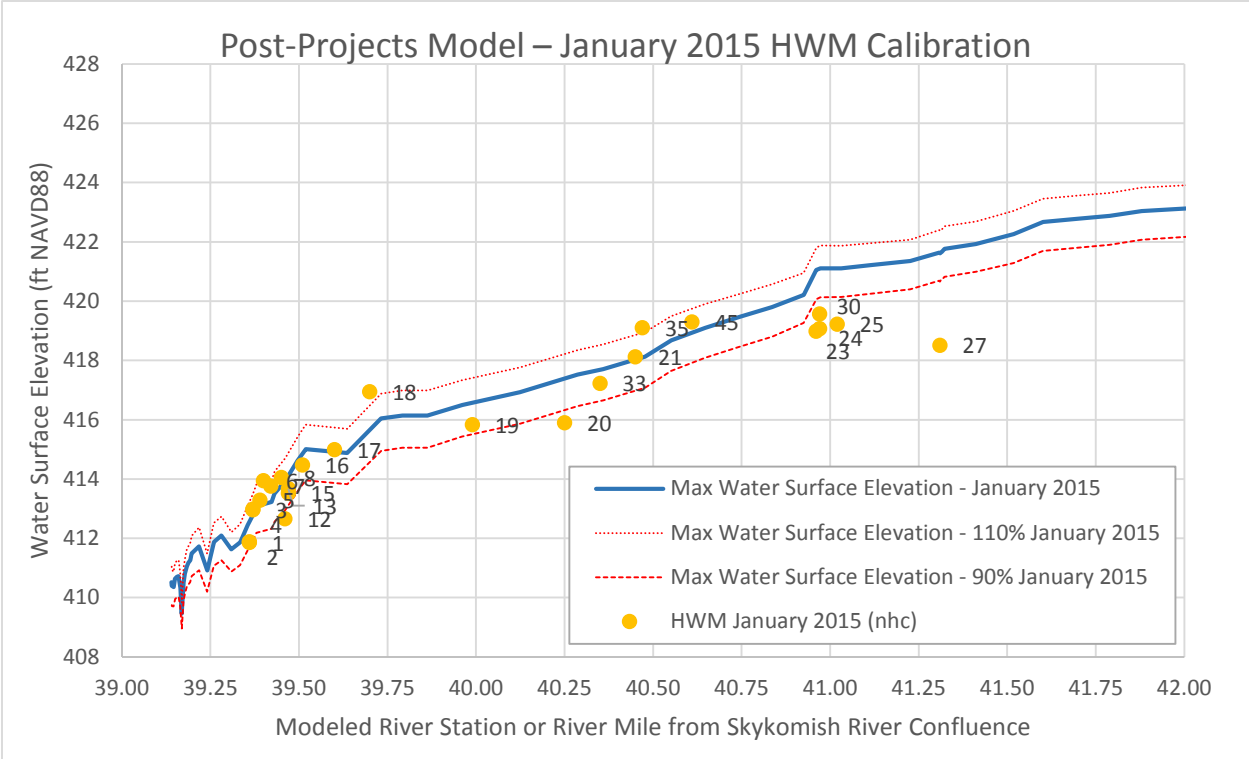
7.2. January 2015 Flood

The January 2015 flood had a peak flow of 50,300 cfs at the USGS Snoqualmie River near Snoqualmie gage. This flow is about 2,600 cfs less than the estimated 10-year flood peak discharge. NHC collected HWMs for this event upstream from Snoqualmie Falls at locations and elevations as shown in Appendix D.

An upstream inflow hydrograph for the HEC-RAS model corresponding to the January 2015 flood event was developed, as described in detail by WSE (2015b). At the time of the January 2015 flood, both the 205 and PSE projects had been completed. Therefore, the inflow hydrograph was configured such that the routed flow at the USGS Snoqualmie River near Snoqualmie gage from the Post-Projects HEC-RAS model matched the observed USGS flow

[12] Flow over a weir is typically computed using an empirical formula ($Q = C_d L H^{3/2}$) where Q is flow, C_d is a weir coefficient based on the type of weir, L is weir length, and H is the depth of flow over the weir.

record for the January 2015 flood. The Post-Projects modeled maximum water surface elevation results for this event were then plotted against the observed HWMs, as shown in Figure 12.



Note: HWM numbers as per NHC (2006); see Appendix D.

Figure 12. Post-Projects Modeled Maximum Water Surface Elevations for January 2015 Flood.

In general, the Post-Projects model maximum water surface profile matches well with the higher HWMs. Adjustments to Manning's n-values were tested to improve the fit to some of the lower HWMs. However, when these same adjustments were tested in the January 2009 event simulation, the calibration to high water marks for that event was much poorer. Therefore, Manning's n-values were left unchanged. The weir coefficient for the extended and lowered PSE weir was increased to 3.4 (from the 3.2 value used for Pre-Projects and Post-205 Project models) to better match the observed HWMs just upstream of the weir. The different weir coefficient is presumably a result of improved hydraulic conditions (less constriction) at the expanded and lowered weir constructed as part of the PSE Project. Given the new PSE Project weir configuration, the use of a different weir coefficient for the Post-Projects scenario is appropriate.

7.3. Uncertainties in Model Calibration

As shown in Figures 11 and 12, the HEC-RAS model calibration is reasonable but not perfect. The intent of the current study was to use HEC-RAS models that had already been developed and used for previous Snoqualmie River hydraulic investigations (FEMA, 2005; NHC, 2006).

Significant refinements as described previously (e.g., conversion to unsteady flow, addition of overflow reaches, addition of storage areas) were made to these models to better simulate hydraulic complexities upstream of the Falls. While these refinements allowed for better model calibration and improved simulation of actual conditions, points remain where the model does not accurately match the reported high water marks. Potential reasons for this include errors and uncertainties in the HWM data, uncertainties in the observed flow data, and hydraulic complexities that are still not fully captured in the HEC-RAS model.

The HWM data display significant scatter. For instance, reported HWMs measured in close proximity to one another following both the 2009 and 2015 flood events vary by as much as 10 feet. Such significant changes in water elevation are not physically possible, and likely show the effects of waves, physical separation of HWMs from the flow, locally ponded areas from a different flood source, or simply mistakes in setting or surveying the HWMs. No hydraulic model can reproduce such dramatic water surface elevation changes. Furthermore, upstream HWMs in many locations were reported to be lower than downstream HWMs for the same event. Such results do not appear to be reasonable (assuming flow is headed downstream) and thus the model cannot be expected to fully match these data.

Uncertainty is also associated with the USGS flow data. The USGS states that its gages are typically accurate to within 10 percent (USGS, 1992). If this range of accuracy is assumed for the calibration events, actual flows could vary substantially from the reported flows. To represent this uncertainty, the calibration and validation models were rerun with a ±10 percent inflow adjustment, and plotted with the maximum HWMs for comparison (see Figures 11 and 12). Changes in the assumed inflows result in shifts in the predicted maximum water surface profile on the order of ±1 foot or more. A majority of the reported HWMs fall within this widened range of simulated maximum surface water elevations.

A final uncertainty in the model calibration is the inability of HEC-RAS to capture all of the complex hydraulic conditions seen near the city of Snoqualmie. As a one-dimensional model, HEC-RAS is best suited to situations where flow directions can be predicted with certainty and flow patterns do not change significantly at higher flows. Single-thread channels and broad, undeveloped, and uniformly sloped floodplains generally meet these criteria. Upstream of the SR 202 Bridge, however, numerous features, as shown on Figure 10, make flow patterns more complex and more difficult to model. These features include the Mill Pond oxbow, railroad tracks, road fills, and natural and constructed levees and berms, all of which can cause flow to go in different directions at different flow levels. Refinements to the hydraulic models, including the addition of the secondary overflow path along the south side of SR 202 and the addition of the modeled storage area near the Meadowbrook Way Southeast Bridge, greatly improve the model's ability to simulate these complex conditions. However, capturing all of the hydraulic complexities and significantly improving the model calibration would have required a far greater degree of model refinement (or a different model altogether), beyond the scope of the current study.

7.4. Effect of Uncertainty on Model Application

To test the sensitivity of the model results to the calibration, the model was also calibrated to the extremes of the observed data rather than to a best fit of the data. By making all the Mill Pond area and much of the left bank upstream from the SR 202 Bridge ineffective (in

terms of the model; see Section 6.4) and by raising the Manning's n-values, the January 2009 flood water surface elevation profile can be forced to better match the highest measured data points in the reach between RMs 40 and 40.75 (see Figure 10). These changes resulted in an increase in the simulated water surface profile for the January 2015 event. Simulated impacts on flows and water surface elevations downstream of the Falls during the 100-year event were lower for the model calibrated to the upper HWM extremes than for the model calibrated to the best fit of the data. Thus, the results using the calibrated model shown in Figures 11 and 12 provide a conservative analysis of downstream impacts. Despite the calibration uncertainties discussed above, the models are appropriate for the purposes of this study.

8. Summary of Results

The hydraulic models described above were used to quantify the effects of the 205 and PSE projects on flows and water levels in the lower Snoqualmie Valley. Data from detailed simulations of the 2-, 10-, 50-, 100-, and 500-year design flood events were extracted and evaluated for locations upstream and downstream of Snoqualmie Falls. The flood events in January 2009 and January 2015 were also simulated, and results were extracted and evaluated for locations at and upstream of Snoqualmie Falls. Analyses for areas upstream of the Falls focused on changes in flood storage and flow attenuation caused by the 205 and PSE projects. Downstream evaluations focused on changes in flow and water levels caused by the loss of upstream storage.

8.1. Changes Upstream of Snoqualmie Falls

The 205 Project reduced simulated 100-year water surface elevations in the city of Snoqualmie (at Riverview Park) by about 0.45 foot. The PSE Project further reduced simulated water levels in the city of Snoqualmie by about 0.96 foot. Together, the combined reduction in peak water level upstream of the Falls due to the two projects is estimated to be 1.4 feet in the 100-year flood. The lower water levels result from increased conveyance of flood flows due to the removal of constrictions upstream of the Falls. In turn, this change accelerates the timing and increases the magnitude of downstream flows and results in higher water levels in the valley downstream of the Falls. At the peak of the simulated 100-year flood event, the incremental flow increase at Snoqualmie Falls (relative to the Pre-Projects condition) is about 1,260 cfs and the peak of the flood occurs about 1 hour earlier. On the rising limb of the flood (before the peak), the increase ranges as high as 2,600 cfs. By contrast, after the peak as the flood is receding, flows at the Falls may be as much as 1,600 cfs lower than in the Pre-Projects condition.

For the 2-year flood, the 205 Project reduced the simulated maximum water surface elevation in Snoqualmie (at Riverview Park) by 1.0 foot and the PSE Project further reduced the simulated maximum water level by 1.2 feet for a total reduction of 2.2 feet. At the peak of the 2-year flood event, simulated flows at the PSE weir were essentially unchanged (less than 1 percent difference). Compared to the Pre-Projects condition, Post-205 Project simulated flows increased by up to 1,000 cfs on the rising limb of the flood and decreased by up to 1,500 cfs on the falling limb.

Maximum water surface elevations simulated at the PSE weir, at Riverview Park in Snoqualmie, and at the upstream end of the model are presented in Table 6. In general, simulated maximum water surface elevations at Riverview Park decrease successively with implementation of each project. Between Pre-Projects and Post-205 Project conditions, the biggest difference occurs through the city of Snoqualmie. Between the Post-205 Project model and Post-Projects model, the difference is greatest at the PSE weir, due primarily to its longer length and lower height.

Table 6. Simulated Upstream Maximum Water Surface Elevations.									
Return Period or Event	Maximum Water Surface Elevation (feet NAVD88)								
	At PSE Weir			City of Snoqualmie Riverview Park (RS 40.647)			Upstream Cross-Section		
	Pre-Projects	Post-205 Project	Post-Projects	Pre-Projects	Post-205 Project	Post-Projects	Pre-Projects	Post-205 Project	Post-Projects
2-year	410.7	410.7	407.3	416.3	415.3	414.1	420.7	420.3	420.0
10-year	414.4	414.4	410.8	421.0	420.5	419.4	425.2	425.0	424.6
50-year	417.0	417.1	413.4	424.4	424.0	423.2	428.1	428.1	427.8
100-year	418.0	418.0	414.2	425.7	425.2	424.2	428.9	428.7	428.6
500-year	419.9	419.9	415.9	428.0	427.6	426.6	430.4	430.3	430.1
Jan. 2009	415.7	415.7	412.0	422.7	422.3	421.1	426.4	426.3	425.9
Jan. 2015	414.1	414.1	410.5	420.8	420.1	419.0	424.9	424.6	424.3

Changes in flows for the simulated events resulting from the 205 and PSE projects are plotted in the figures presented in Appendix E, and the corresponding peak flows at the PSE weir are listed in Table 7. In general, simulated peak flows are slightly higher for the Post-205 Project condition than for the Pre-Projects condition. More significant increases are simulated for the Post-Projects condition for all flood events modeled. The increased discharges result from lower upstream water surface elevations (and corresponding loss of flood storage), which are due primarily to the modified PSE weir. The simulated increase in peak flow from Pre- to Post-Projects conditions at Snoqualmie Falls ranges from 740 cfs to 1,850 cfs, with the greatest differences simulated for the 50-year flood event.

Table 7. Simulated Maximum Discharge at the PSE Weir.			
Return Period or Event	Flow at PSE Weir (cfs)		
	Pre-Projects	Post-205 Project	Post-Projects
2-year	29,840	29,870	30,010
10-year	51,470	51,500	52,350
50-year	70,590	71,360	72,440
100-year	78,870	79,220	80,050
500-year	94,820	94,580	95,560
January 2009	60,600	60,900	61,100
January 2015	49,300	49,400	50,370

At Riverview Park in Snoqualmie, the simulated changes in peak stage resulting from the 205 and PSE projects follow a trend similar to that seen at the PSE weir, with the greatest reduction in water surface occurring as a result of the modified weir. Simulated stage decreases at Riverview Park across all return periods are generally about 0.5 foot due to the 205 Project and about an additional 1 foot due to the PSE Project. At the upstream end of the model, near the confluence of the Middle and South Forks, simulated changes in stage are generally small (less than 0.3 foot for the larger design flood events).

The increased conveyance upstream of Snoqualmie Falls due to the 205 and PSE projects reduces upstream flooding and flow attenuation. The total volume of flow passing Snoqualmie Falls during the simulated 100-year flood event is about 250,000 acre-feet. The volume of water held in storage upstream of the Falls in the Post-Projects condition is approximately 2,500 acre-feet less than under Pre-Project conditions, or 1 percent of the 100-year flood volume. This reduction in flood storage represents approximately 13 percent of the simulated Pre-Project flood storage (18,600 acre-feet) in the 100-year event upstream of the Falls.

8.2. Changes Downstream of Snoqualmie Falls

Conditions resulting from the 205 and PSE projects were evaluated at the PSE weir and four downstream locations:

- The Fall City golf course
- Near the confluence of Patterson Creek
- Below the Tolt River confluence
- Near Duvall

For each modeled condition, flow and stage hydrographs at each location were plotted, as were differences between the Pre-Projects, Post-205 Project, and Post-Projects conditions. Results for peak stages simulated during the 100-year event are summarized in Table 8 and results for the 2-year event are summarized in Table 9. Flow and stage hydrographs and difference plots for the 100-year flood event simulations for the PSE weir and the four downstream locations are provided in Figures 13 through 21 (note that figures are not provided for flows at the Patterson Creek location because the model geometry is split into several parallel flow paths at that location and it is not possible to extract a combined flow hydrograph from the HEC-RAS model). Flow and stage hydrographs and difference plots for the 2-year flood event simulations at these same five locations are provided in Figures 22 through 30. Model results plots for the other flood events (i.e., the 10-, 50-, and 500-year flood events) are provided in Appendix E.

Table 8. Simulated Change in Peak Stages for the 100-Year Event.

Location	River Station (mile)	Stage (feet NAVD88) or Change (feet)					
		Pre-Project	Post-205		Post-205 and PSE		
		Stage	Stage	Change from Pre-Project	Stage	Change from Post-205	Total Change
City of Snoqualmie	40.65	425.09	424.67	-0.42	423.62	-1.05	-1.47
PSE Weir	39.14	417.97	418.01	+0.04	414.16	-1.85	-1.81
Fall City Golf Course	35.23	103.62	103.64	+0.02	103.70	+0.08	+0.10
Patterson Creek	28.00	80.93	80.95	+0.02	81.02	+0.07	+0.09
Below Tolt River Confluence	23.21	74.11	74.12	+0.01	74.19	+0.07	+0.08
City of Duvall	12.40	52.56	52.56	+0.00	52.56	+0.00	+0.00

Table 9. Simulated Change in Peak Stages for the 2-Year Event.							
		Stage (feet NAVD88) or Change (feet)					
		Pre-Project	Post-205		Post-205 and PSE		
Location	River Station (mile)	Stage	Stage	Change from Pre-Project	Stage	Change from Post-205	Total Change
City of Snoqualmie	40.65	416.27	415.33	-0.94	414.09	-1.23	-2.18
PSE Weir	39.14	410.72	410.73	+0.01	407.32	-3.41	-3.40
Fall City Golf Course	35.23	98.17	98.18	+0.01	98.22	+0.04	+0.05
Patterson Creek	28.00	75.60	75.62	+0.02	75.65	+0.03	+0.05
Below Tolt River Confluence	23.21	69.31	69.33	+0.02	69.37	+0.04	+0.06
City of Duvall	12.40	44.82	44.81	+0.01	44.80	+0.01	+0.02

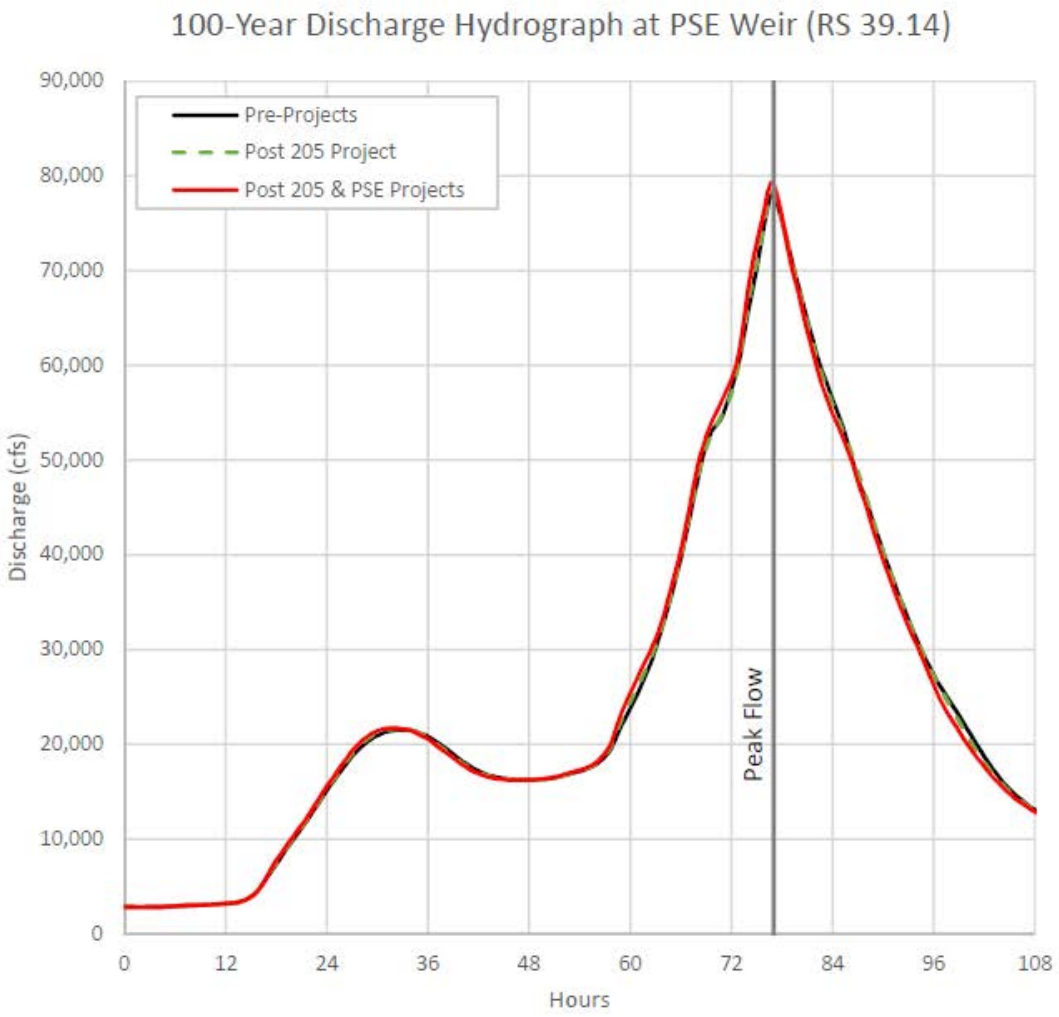
The timing of the greatest differences in stage between Pre- and Post-Projects conditions does not generally coincide with the peak stage in the event. For example, the maximum stage increase for the 100-year event below the Tolt River is 0.15 foot but this occurs before the peak of the flood (see Figure 19). At the peak of the flood, the increase in the 100-year stage is 0.08 foot as shown in Table 8. The maximum stage increase at any location downstream of the Falls under any of the simulated events is seen near Fall City, where the stage prior to the peak is increased by as much as 0.4 foot in the 10-year event (see plotted model results for the 10-year event in Appendix E). At the time of the peak, however, the increase in stage at this location is only about 0.10 foot (i.e., about an inch) (Table 8).

The hydraulic modeling also indicates that the timing of arrival of peak flows and stages downstream of Snoqualmie Falls was slightly accelerated by the 205 and PSE projects. The analysis shows that downstream flows and stages may peak 15 to 30 minutes earlier because of the projects. Similar differences in timing are seen in each of the modeled flood events as shown in the plots in Appendix E.

In addition to the 2- through 500-year events, simulations were conducted for the January 2009 and January 2015 floods. Simulations for these two floods could only be carried as far as Snoqualmie Falls, as a comprehensive analysis of basin hydrology for these more recent events has not yet been completed and corresponding hydrologic inputs for locations downstream of the Falls are not available. Note, however, that the January 2009 event was between a 10- and 50-year flood (for which comprehensive results are provided in Appendix E) and the January 2015 flood event was very close to a 10-year flood (for which comprehensive results are provided in Appendix E). Figures are also included in Appendix E showing conditions at a location near Riverview Park in downtown Snoqualmie (RM 40.647) and at the PSE weir for these two recent flood events. As seen in the hydrographs for the January 2015 flood, the simulated flow at the weir increased by as much as 2,500 cfs during the rising limb of the flood hydrograph but the simulated increase in flow at the peak of the event is only about 1,070 cfs (about 2 percent of the event peak) compared to the Pre-Projects condition. In the January 2009 event, simulated flows increase by as much as 2,000 cfs during the rising limb of the hydrograph but the model results indicate that the peak

flow in that event only increased by about 500 cfs (or less than 1 percent) compared to the Pre-Projects condition.

In addition to questions or concerns about flood flows and water levels, landowners downstream of the Falls have raised questions about the effect of the projects on flow velocities. Given the relatively small increases in simulated peak discharges due to the 205 and PSE projects, it is logical to assume that changes in flow velocities during flood events are small as well. Peak in-channel and overbank flow velocities simulated at all model cross-sections downstream of Snoqualmie Falls were tabulated and reviewed in HEC-RAS. Simulated changes in peak flow velocities are on the order of 0.01 to 0.02 foot per second (fps) for all locations downstream of Fall City and simulated changes between Snoqualmie Falls and Fall City are less than about 0.1 fps (the higher differences were seen only at locations where simulated flow velocities were 7 fps or higher). Given that these differences are less than 1 percent of the simulated existing conditions flow velocities, model results plots for flow velocities were not created.



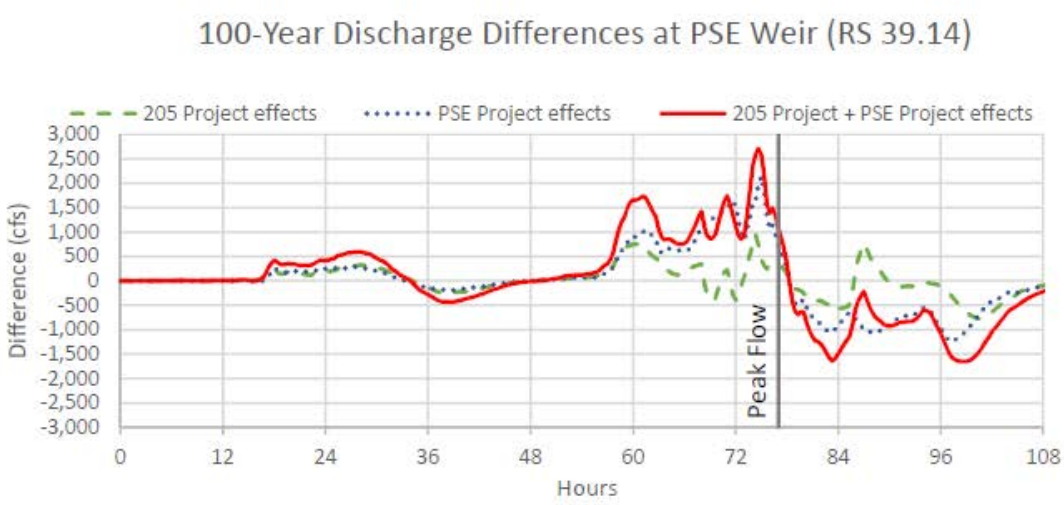
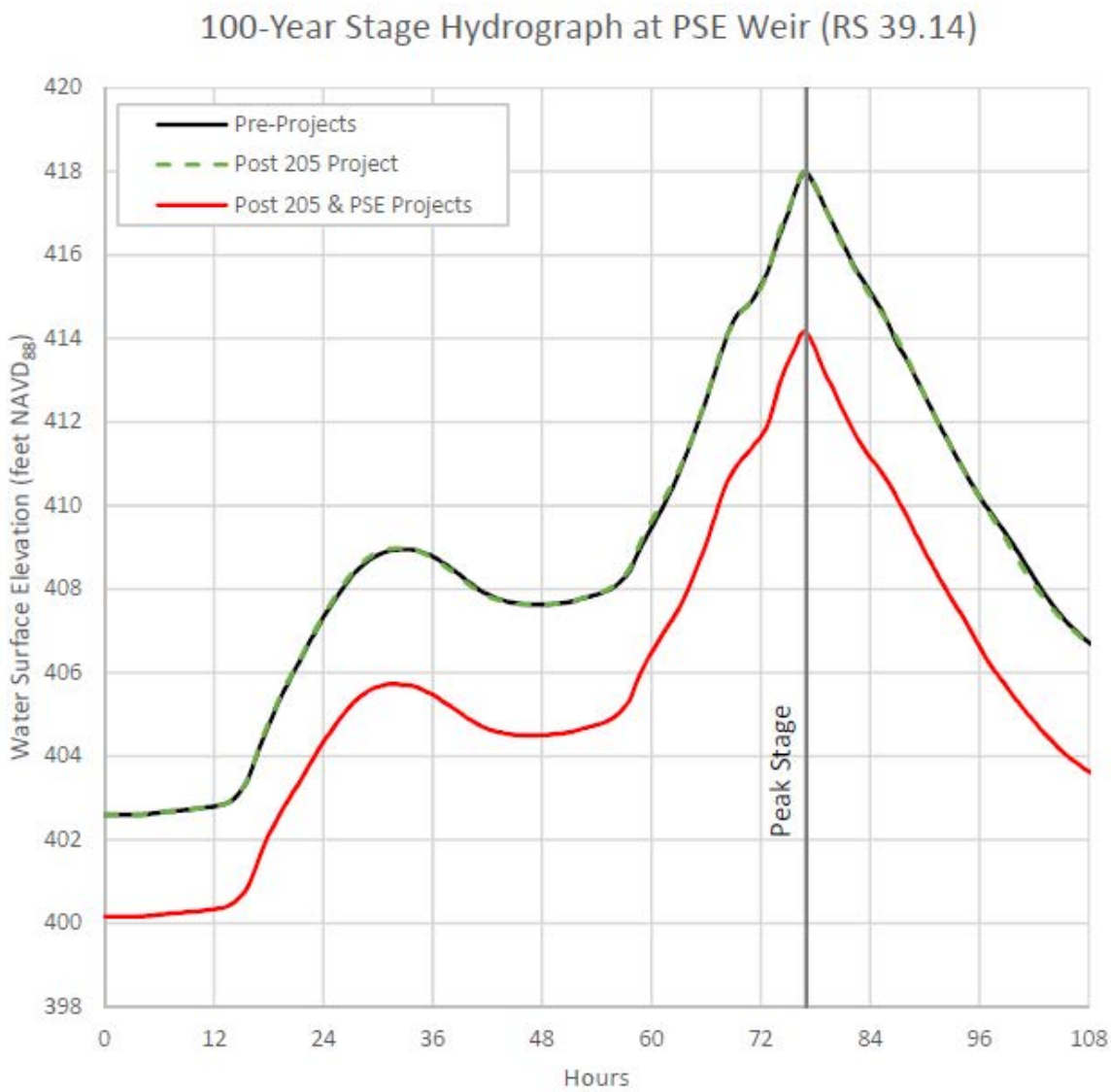


Figure 13. Comparison of 100-Year Flood Discharge Hydrographs and Modeled Differences Caused by the Projects at the PSE Weir.



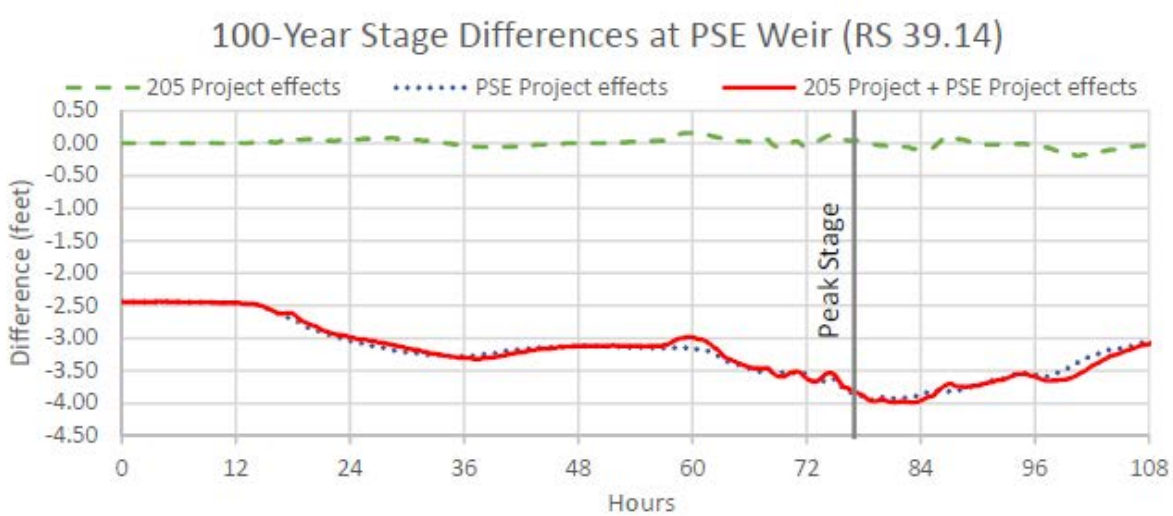


Figure 14. Comparison of 100-Year Flood Stages and Modeled Differences Caused by the Projects at the PSE Weir.

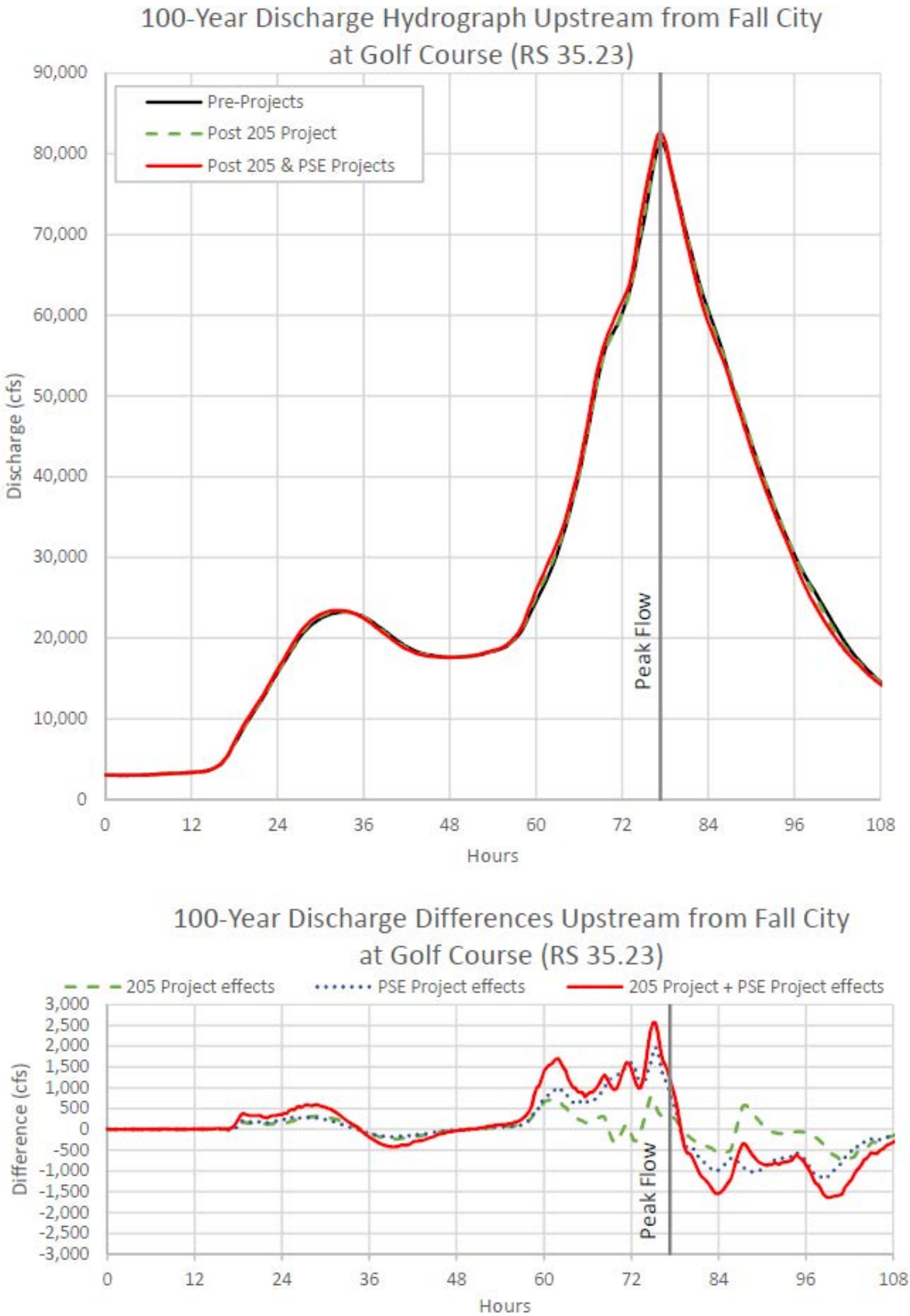


Figure 15. Comparison of 100-Year Flood Discharge Hydrographs and Modeled Differences Caused by the Projects Upstream from Fall City at the Golf Course.

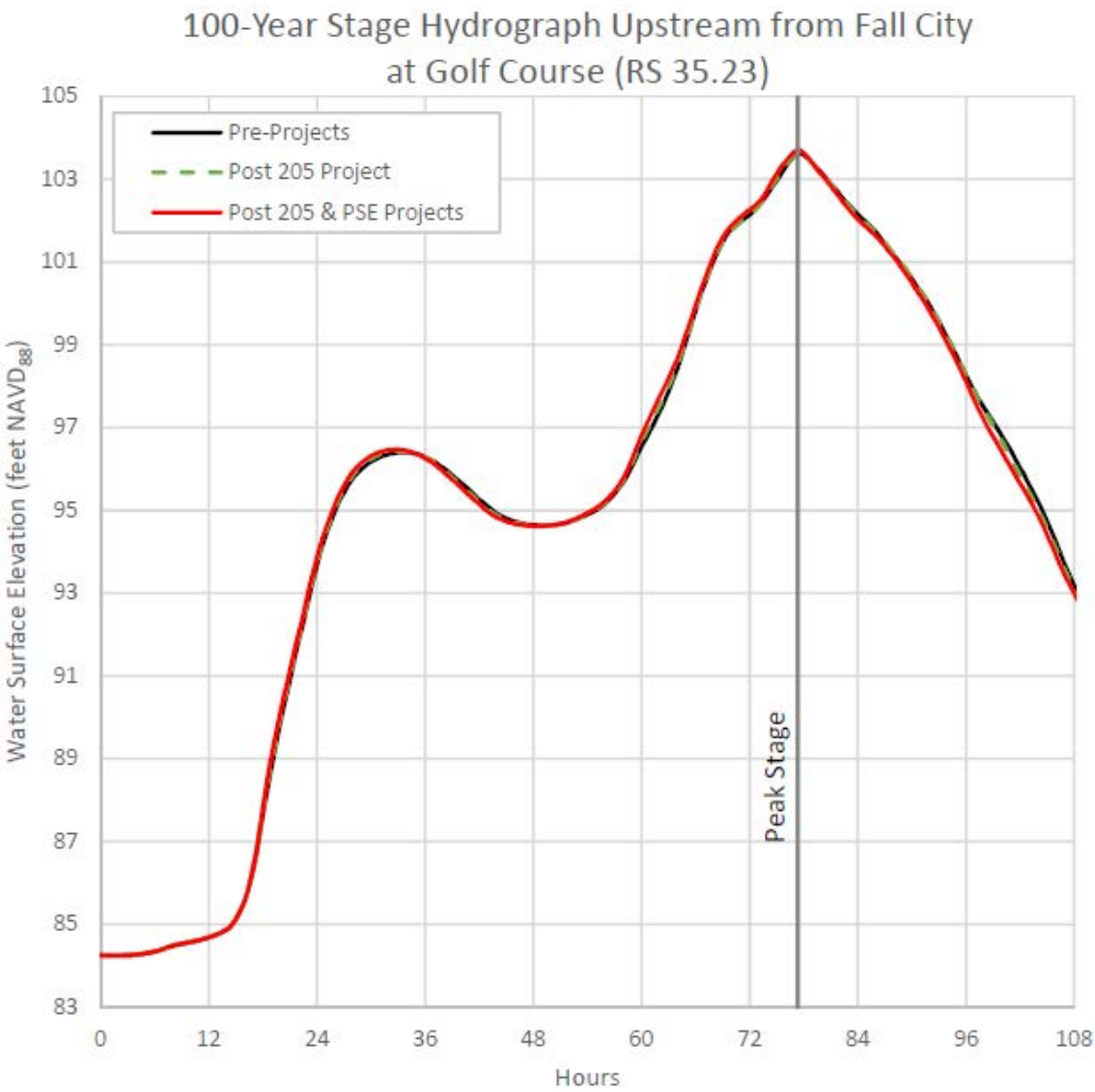


Figure 16. Comparison of 100-Year Flood Stages and Modeled Differences Caused by the Projects Upstream from Fall City at the Golf Course.

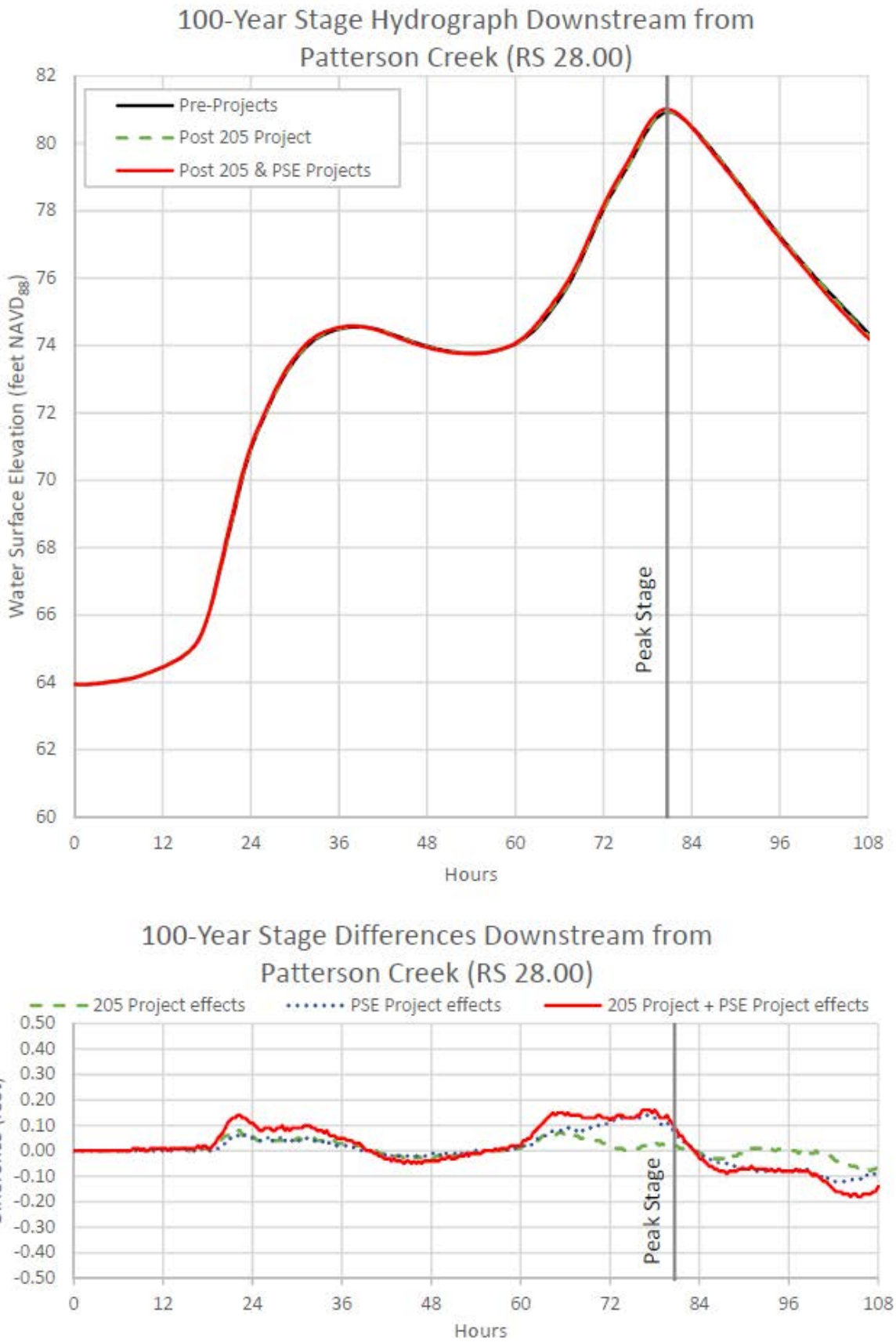


Figure 17. Comparison of 100-Year Flood Stages and Modeled Differences Caused by the Projects Downstream of the Patterson Creek Confluence.

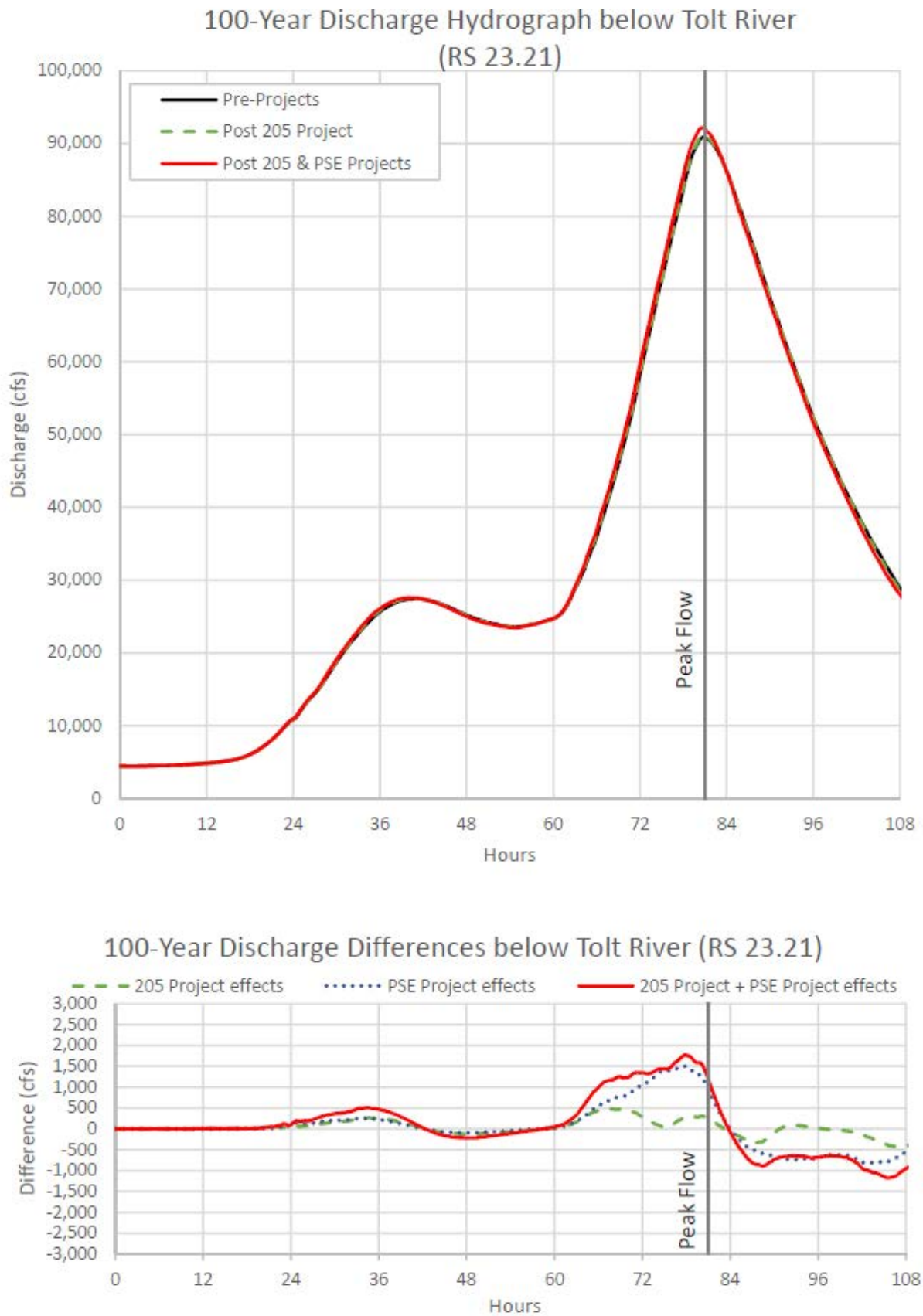
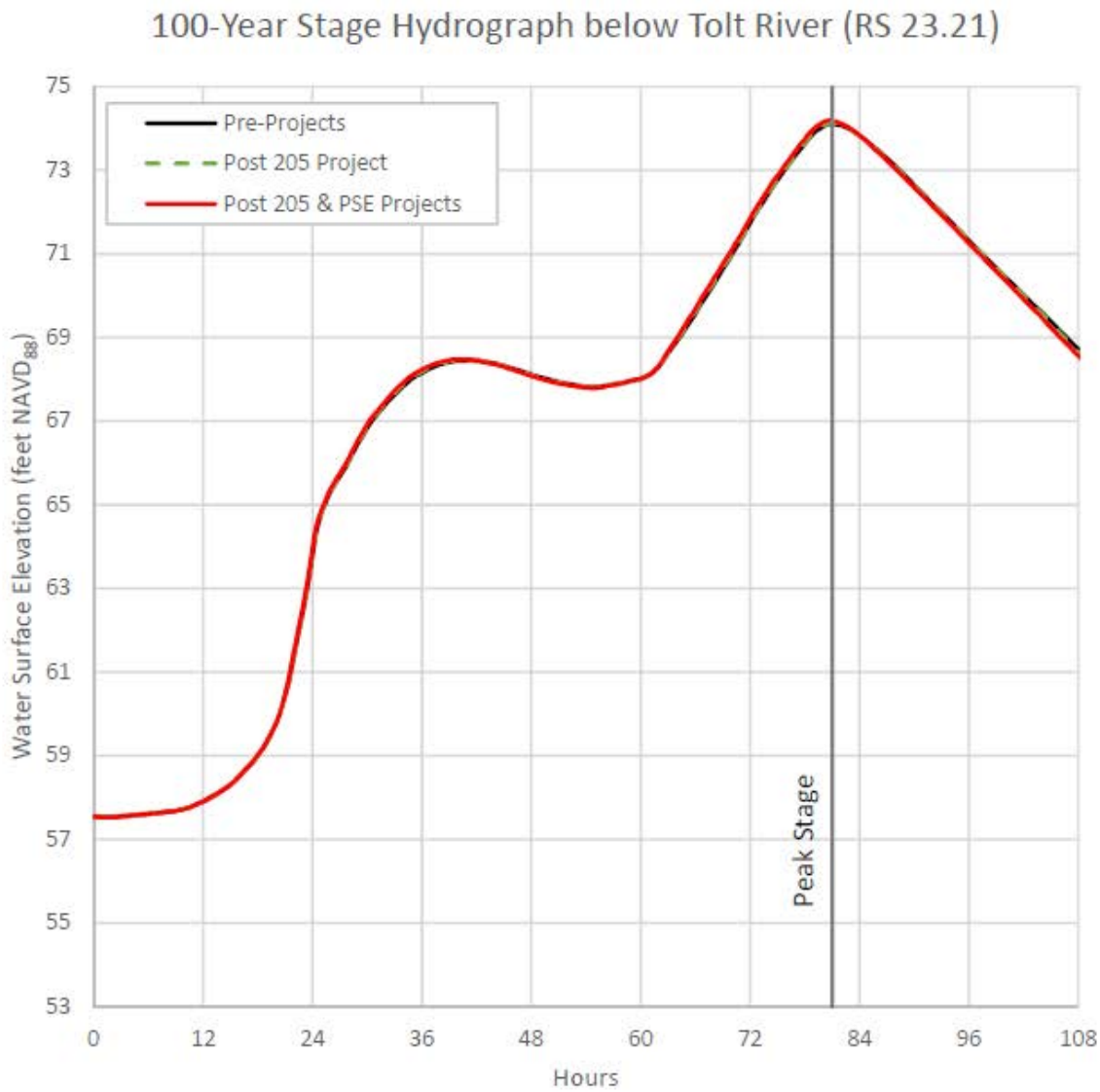


Figure 18. Comparison of 100-Year Flood Discharge Hydrographs and Modeled Differences Caused by the Projects Below the Tolt River Confluence.



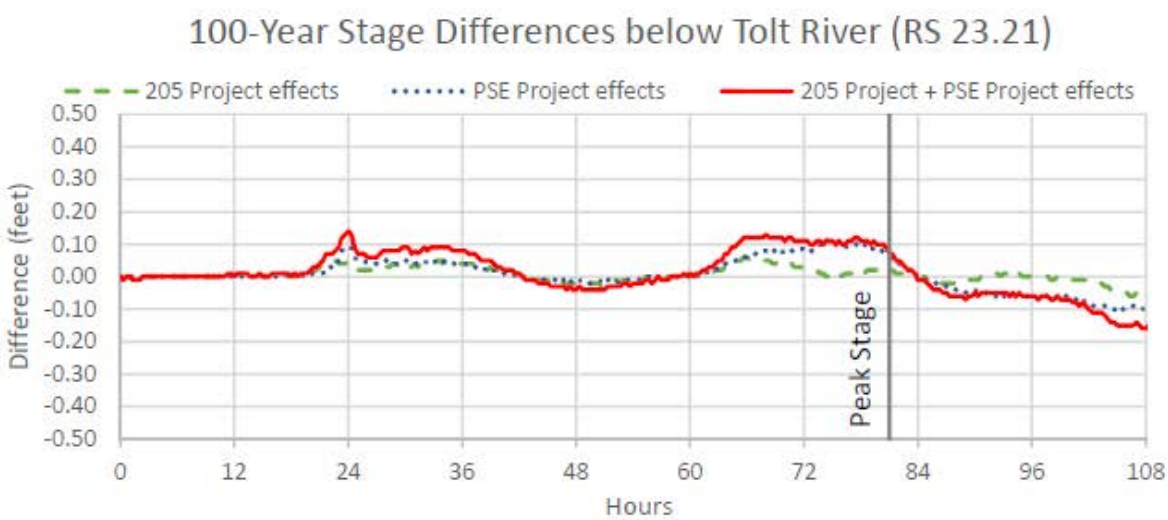


Figure 19. Comparison of 100-Year Flood Stages and Modeled Differences Caused by the Projects Below the Tolt River Confluence.

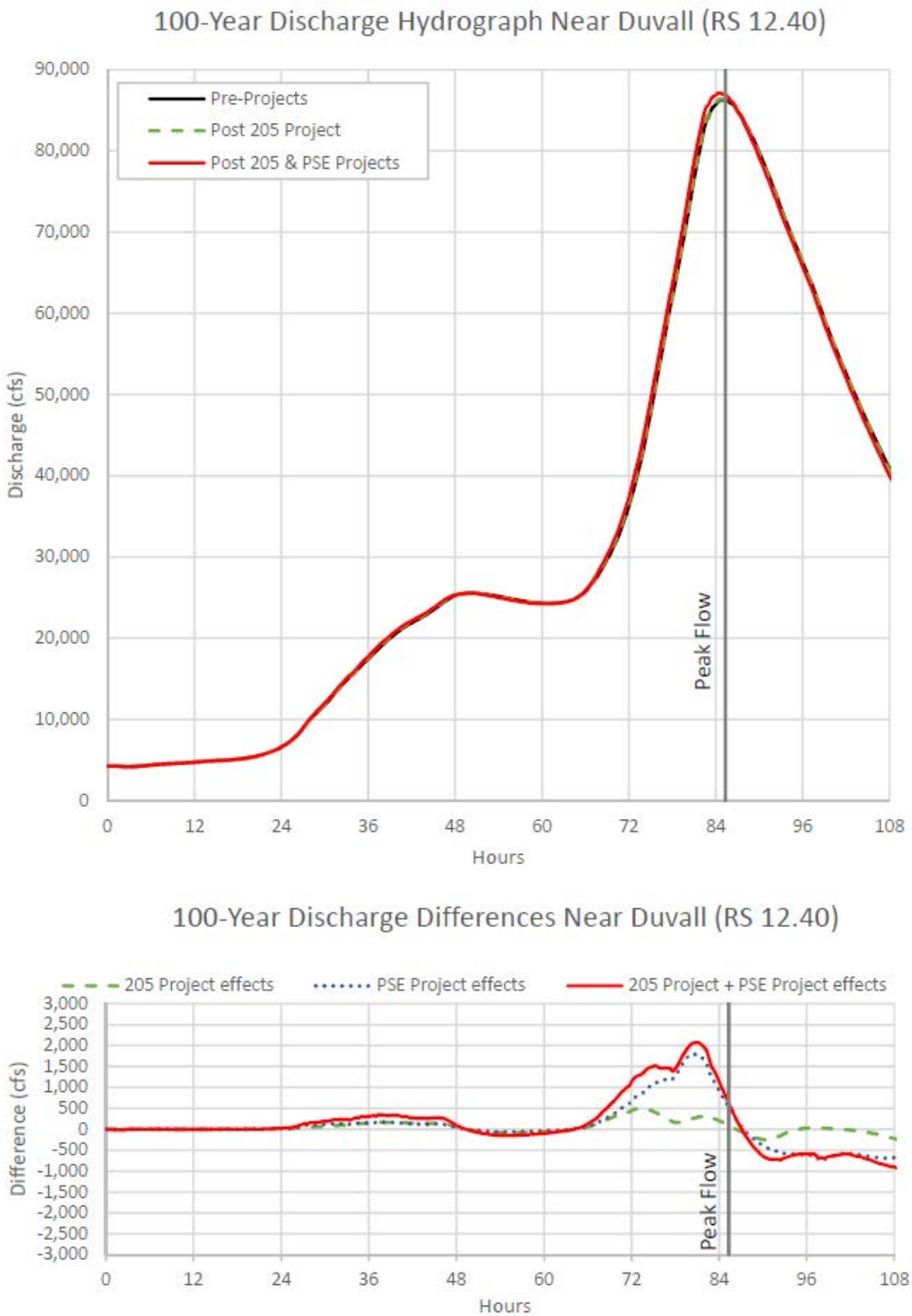
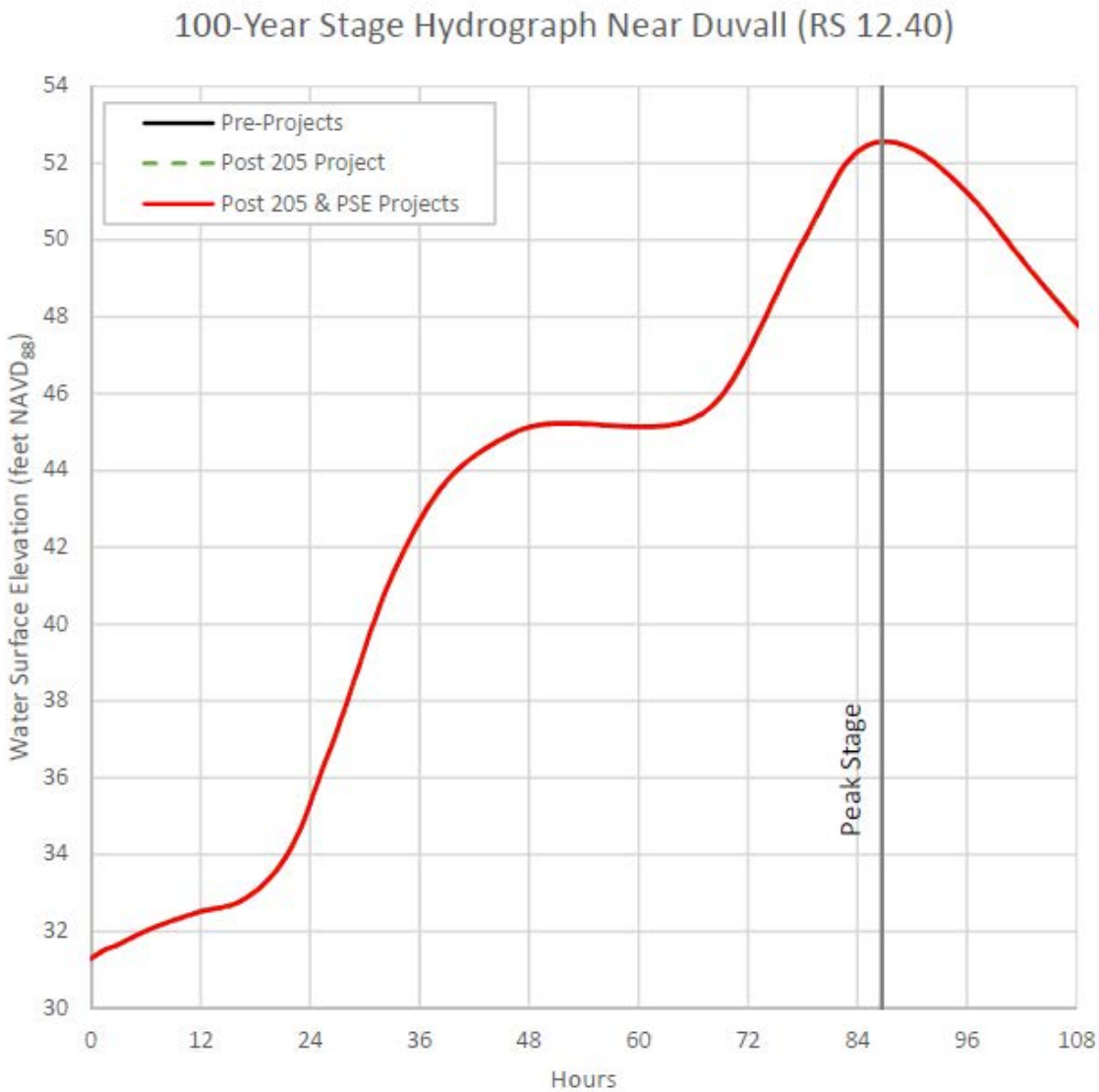


Figure 20. Comparison of 100-Year Flood Discharge Hydrographs and Modeled Differences Caused by the Projects near Duvall.



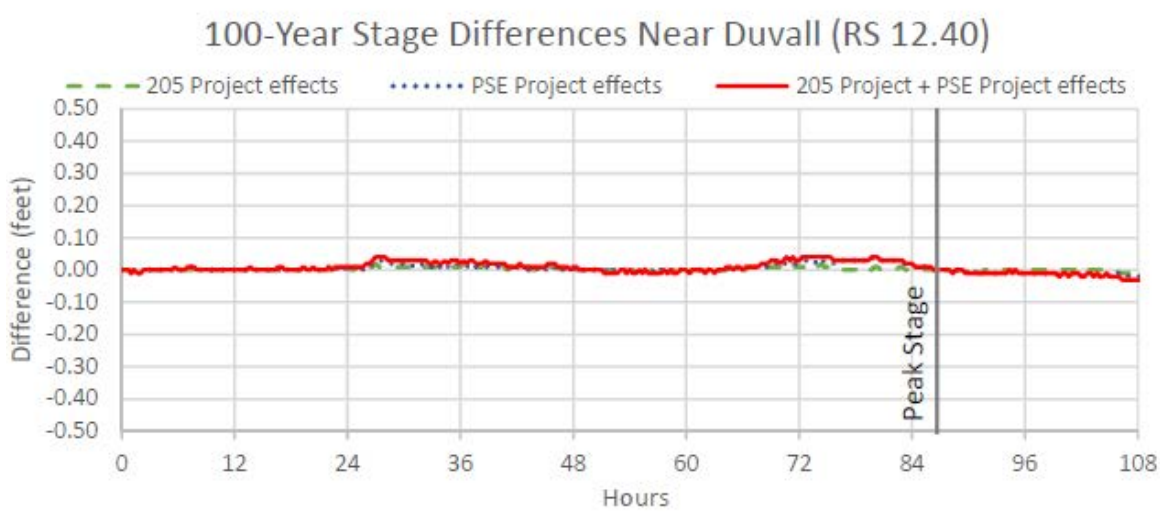


Figure 21. Comparison of 100-Year Flood Stages and Modeled Differences Caused by the Projects near Duvall.

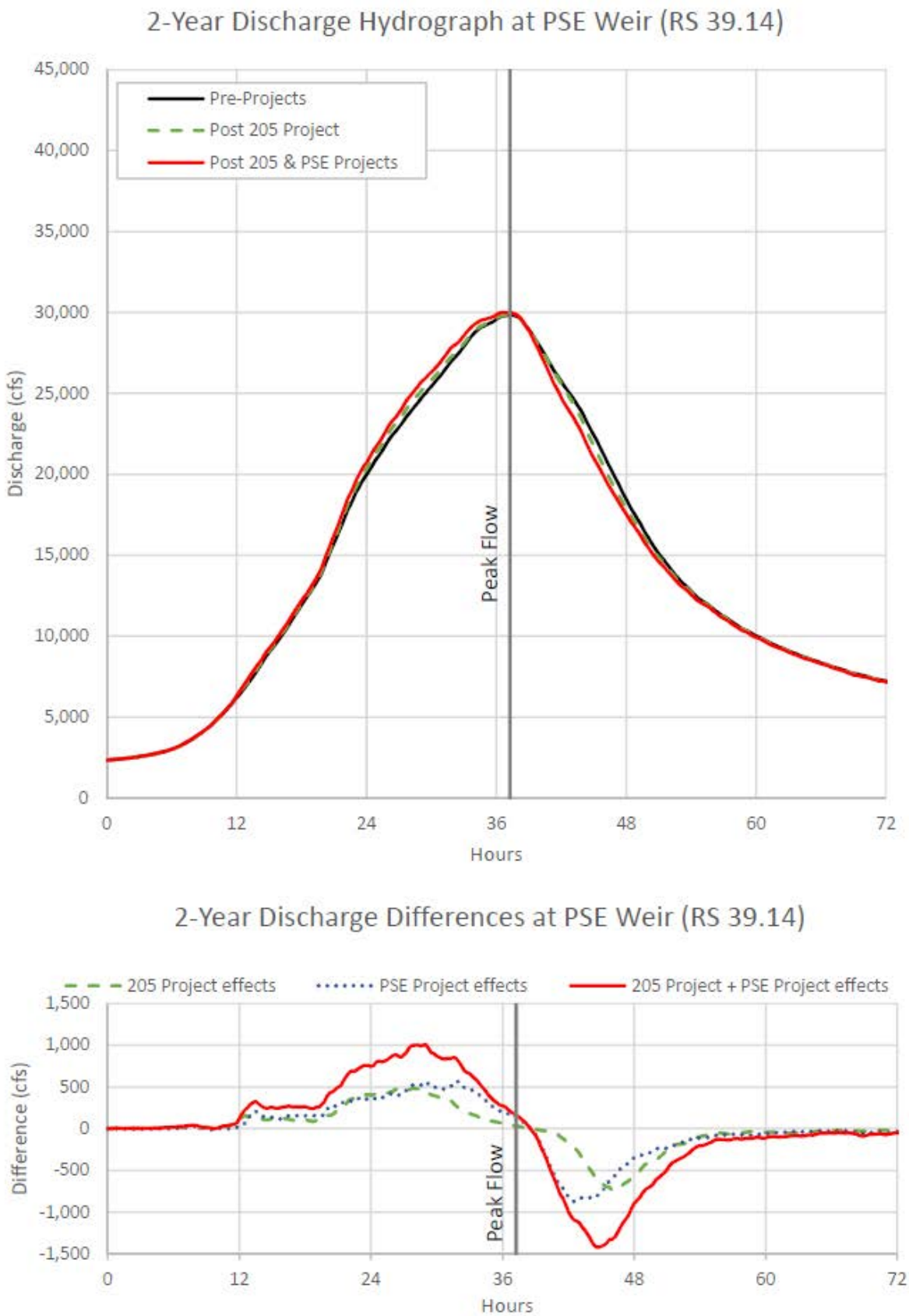


Figure 22. Comparison of 2-Year Flood Discharge Hydrographs and Modeled Differences Caused by the Projects at the PSE Weir.

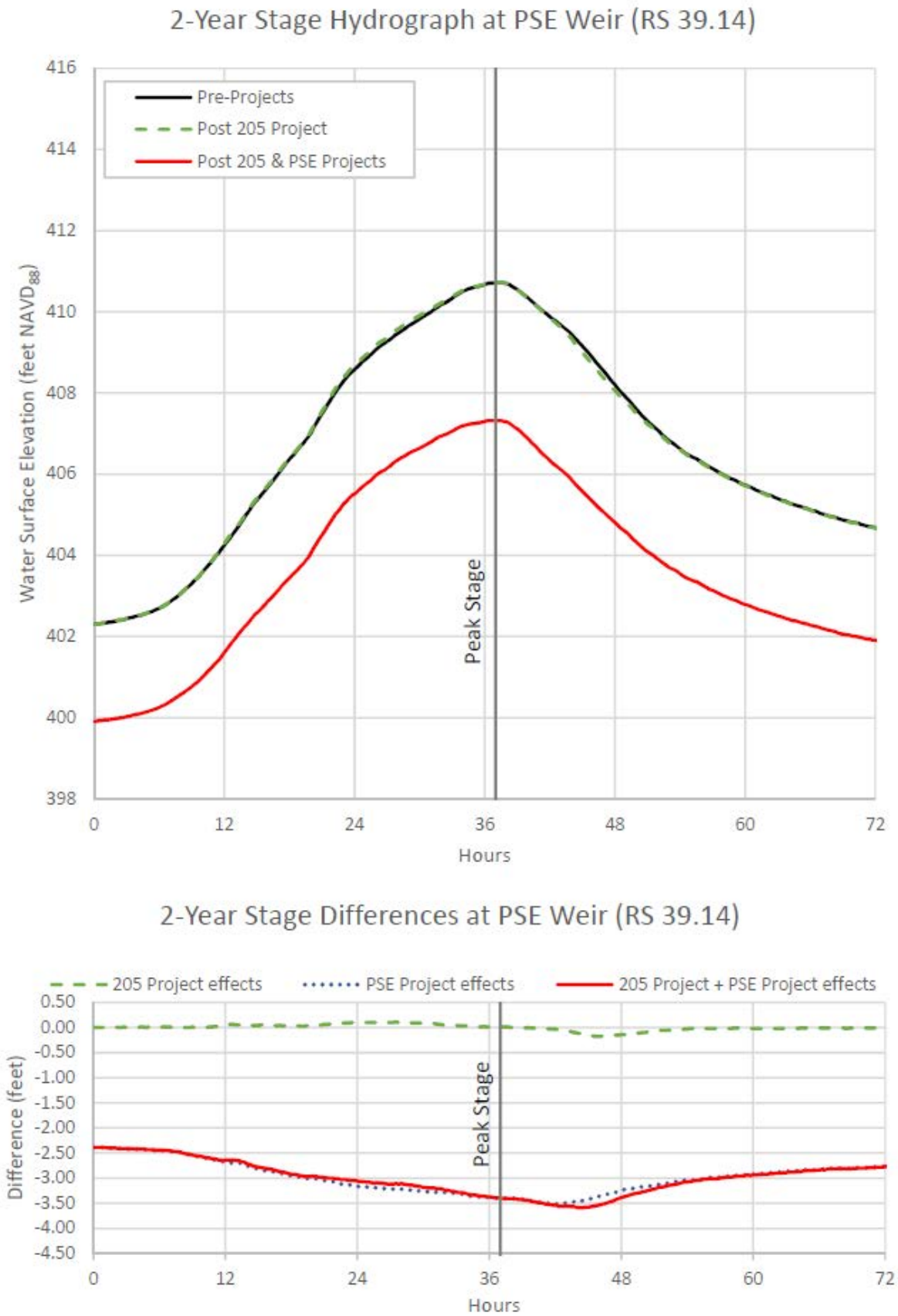
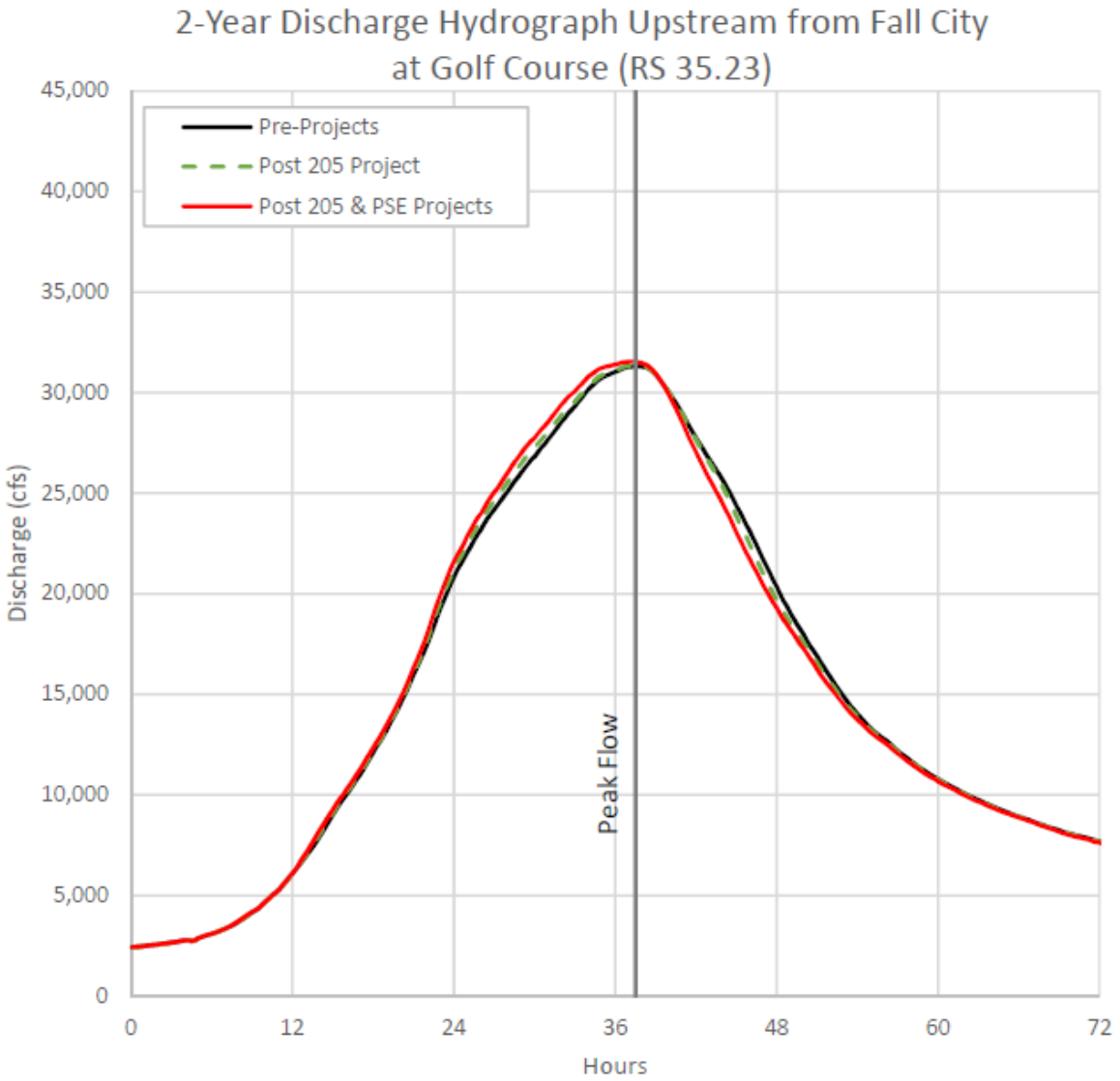


Figure 23. Comparison of 2-Year Flood Stages and Modeled Differences Caused by the Projects at the PSE Weir.



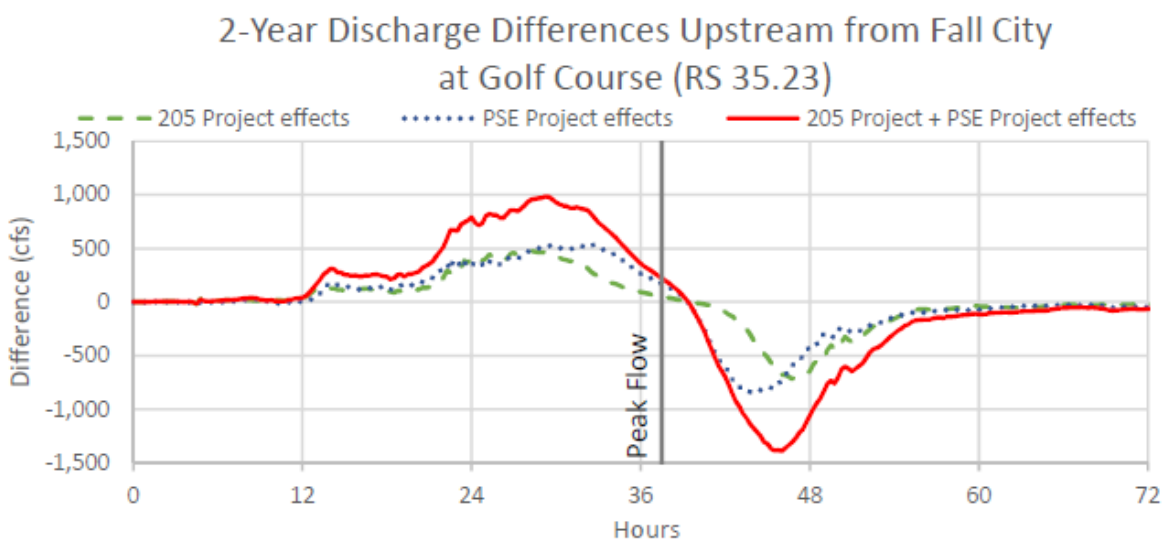


Figure 24. Comparison of 2-Year Flood Discharge Hydrographs and Modeled Differences Caused by the Projects Upstream from Fall City at the Golf Course.

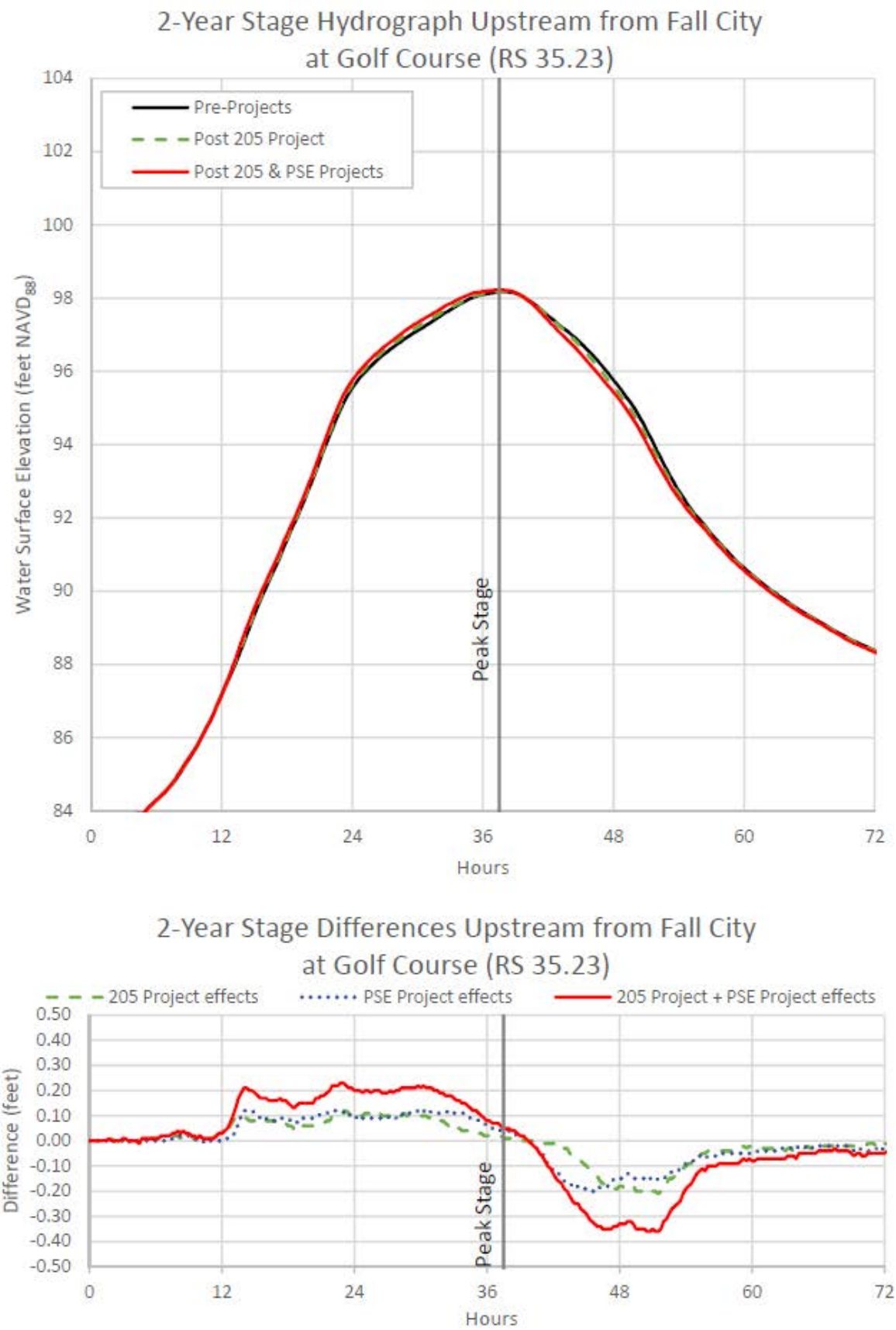
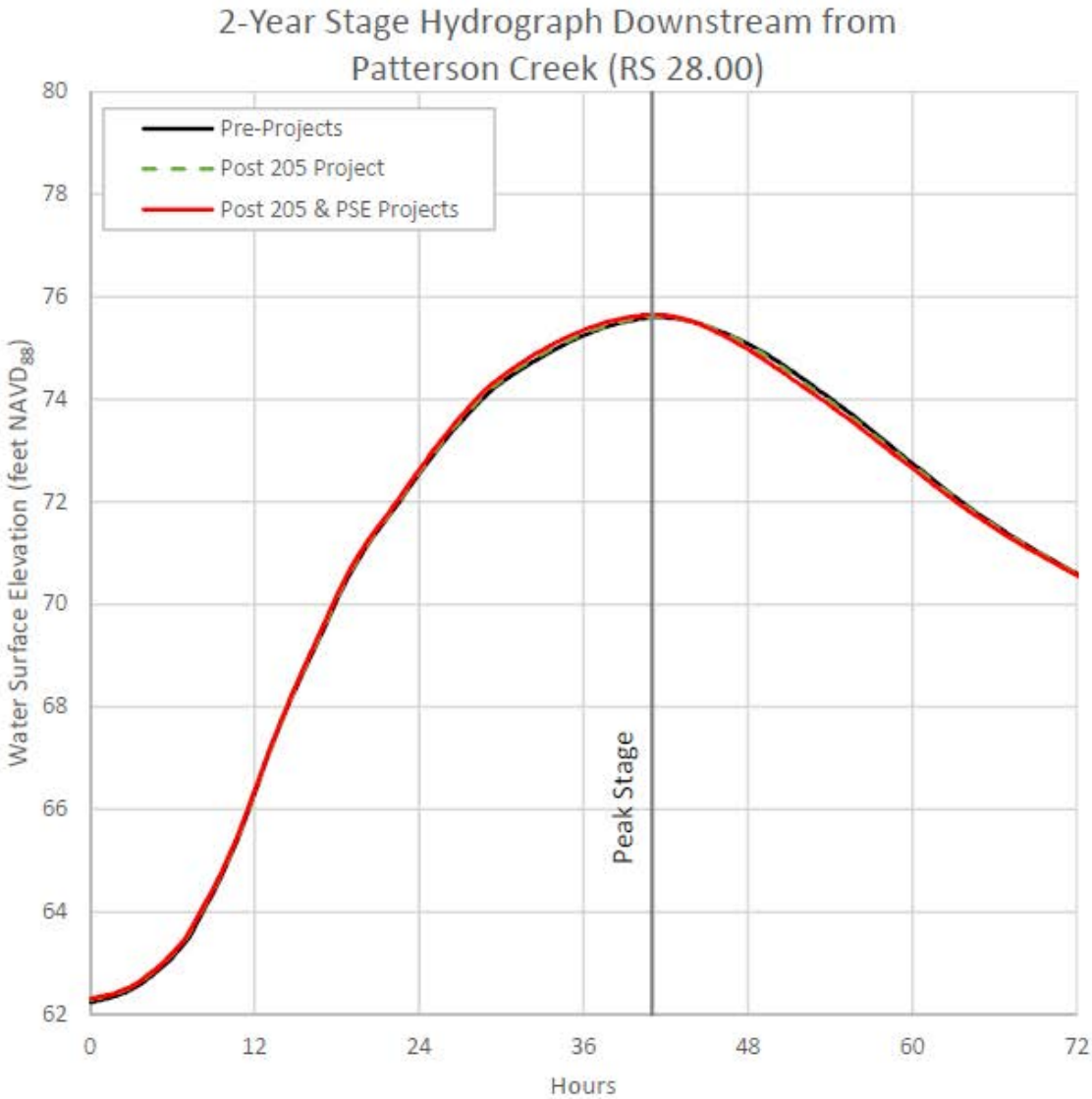


Figure 25. Comparison of 2-Year Flood Stages and Modeled Differences Caused by the Projects Upstream from Fall City at the Golf Course.



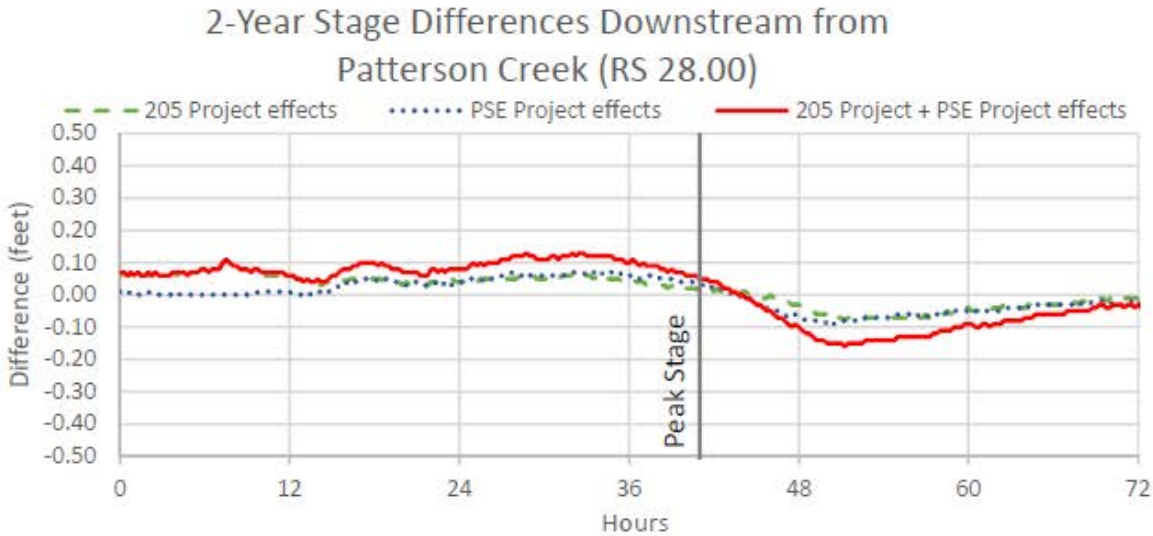


Figure 26. Comparison of 2-Year Flood Stages and Modeled Differences Caused by the Projects Downstream of the Patterson Creek Confluence.

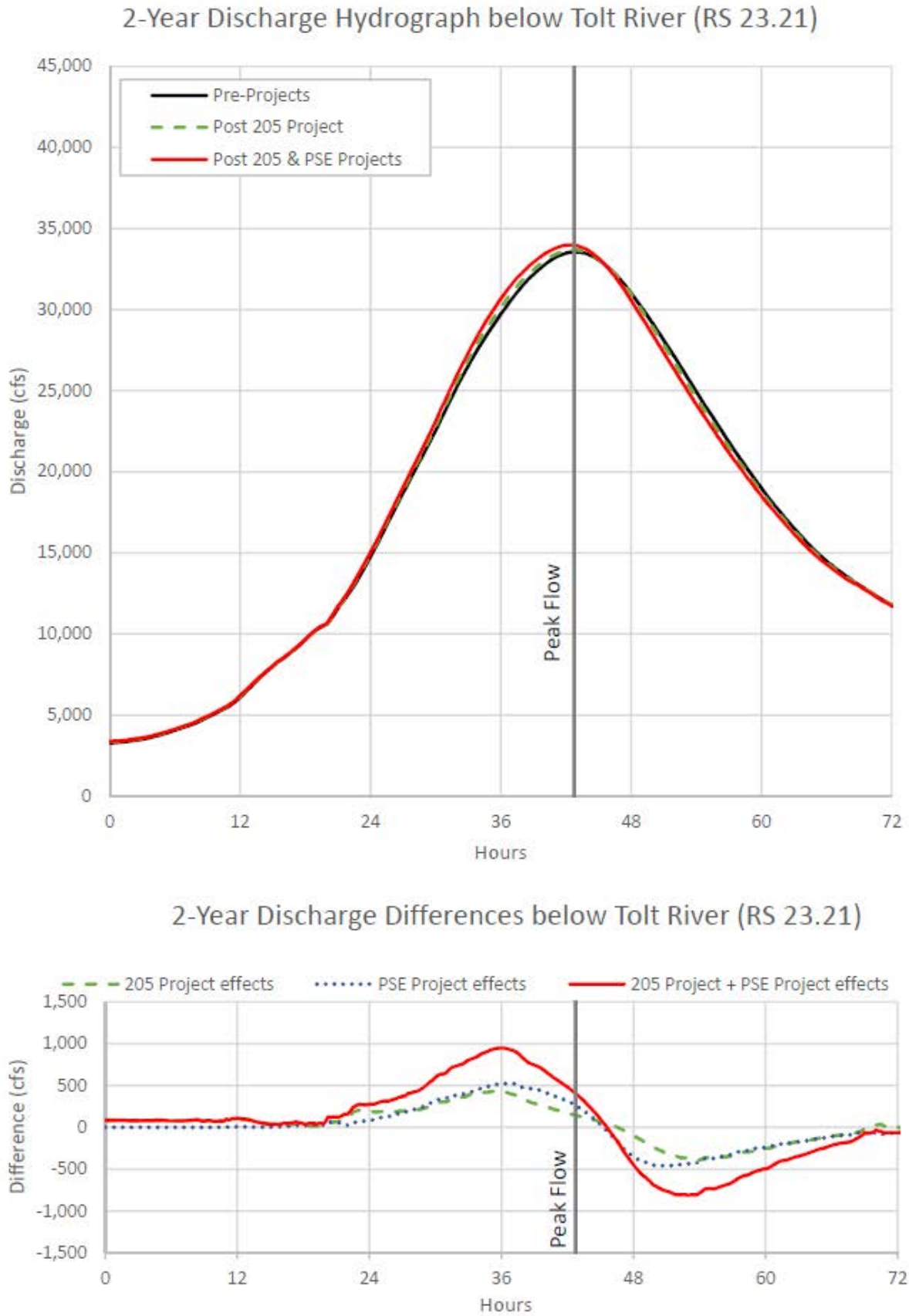
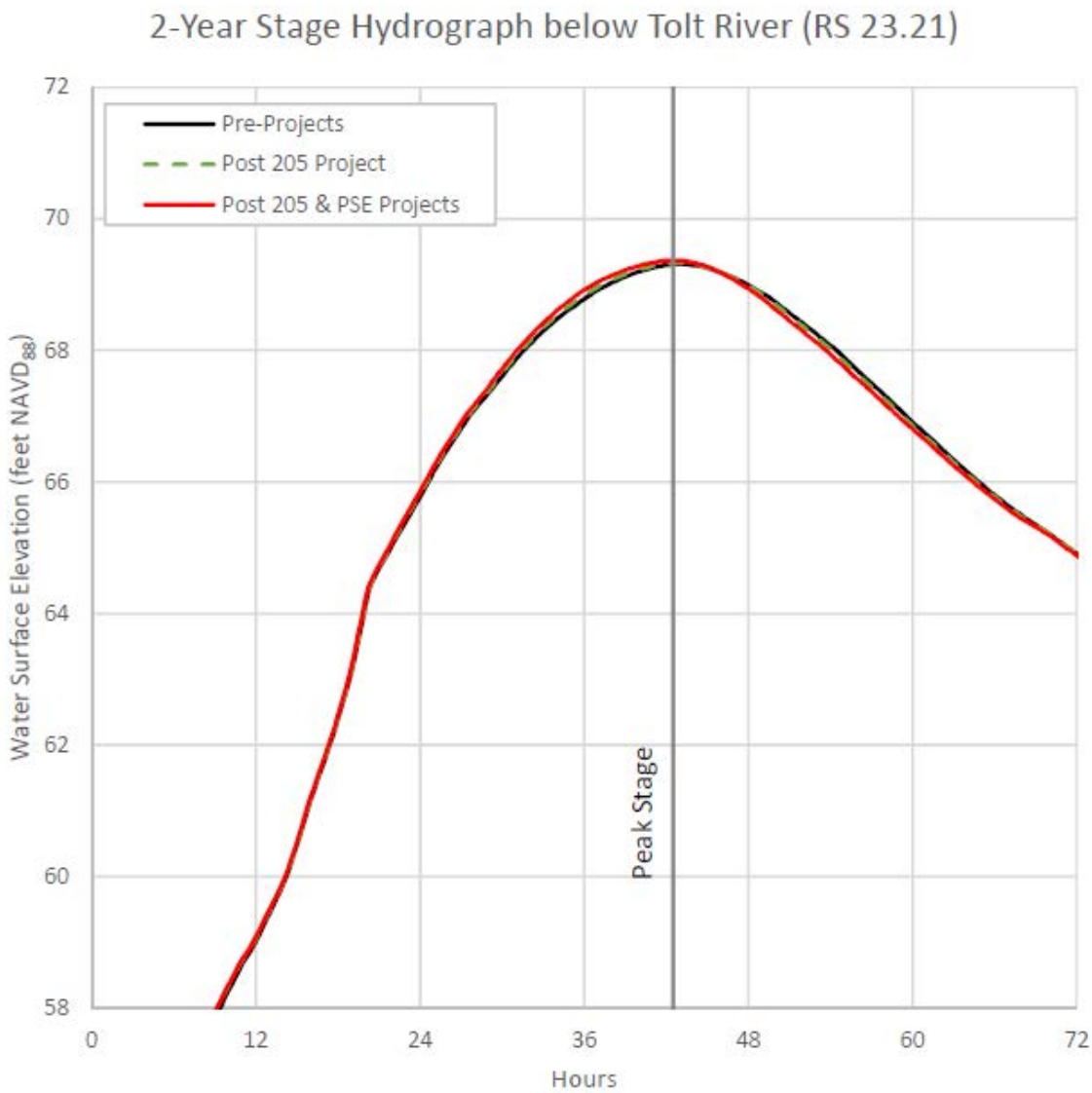


Figure 27. Comparison of 2-Year Flood Discharge Hydrographs and Modeled Differences Caused by the Projects Below the Tolt River Confluence.



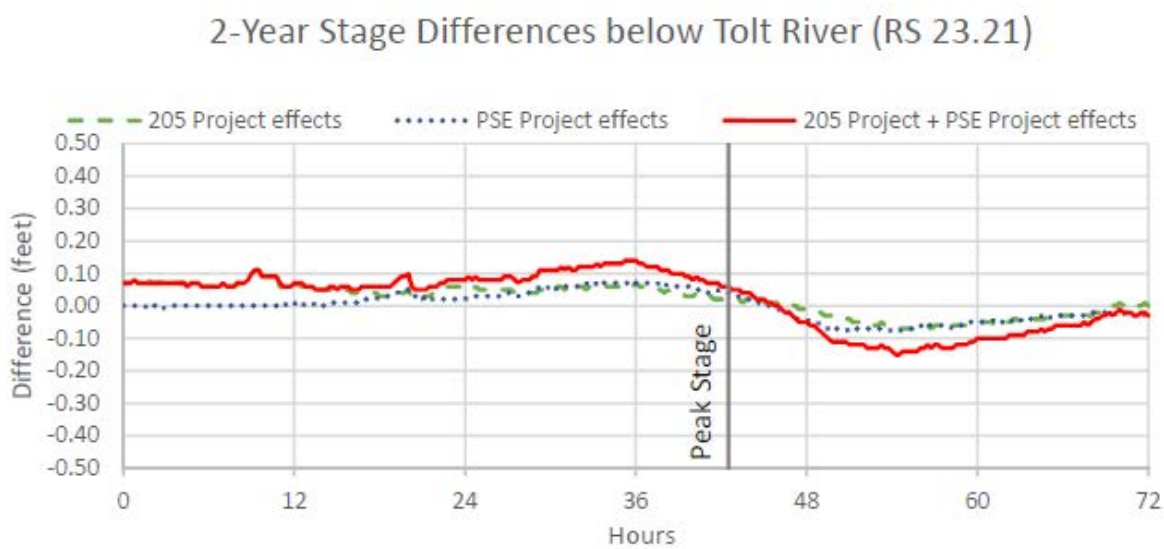


Figure 28. Comparison of 2-Year Flood Stages and Modeled Differences Caused by the Projects Below the Tolt River Confluence.

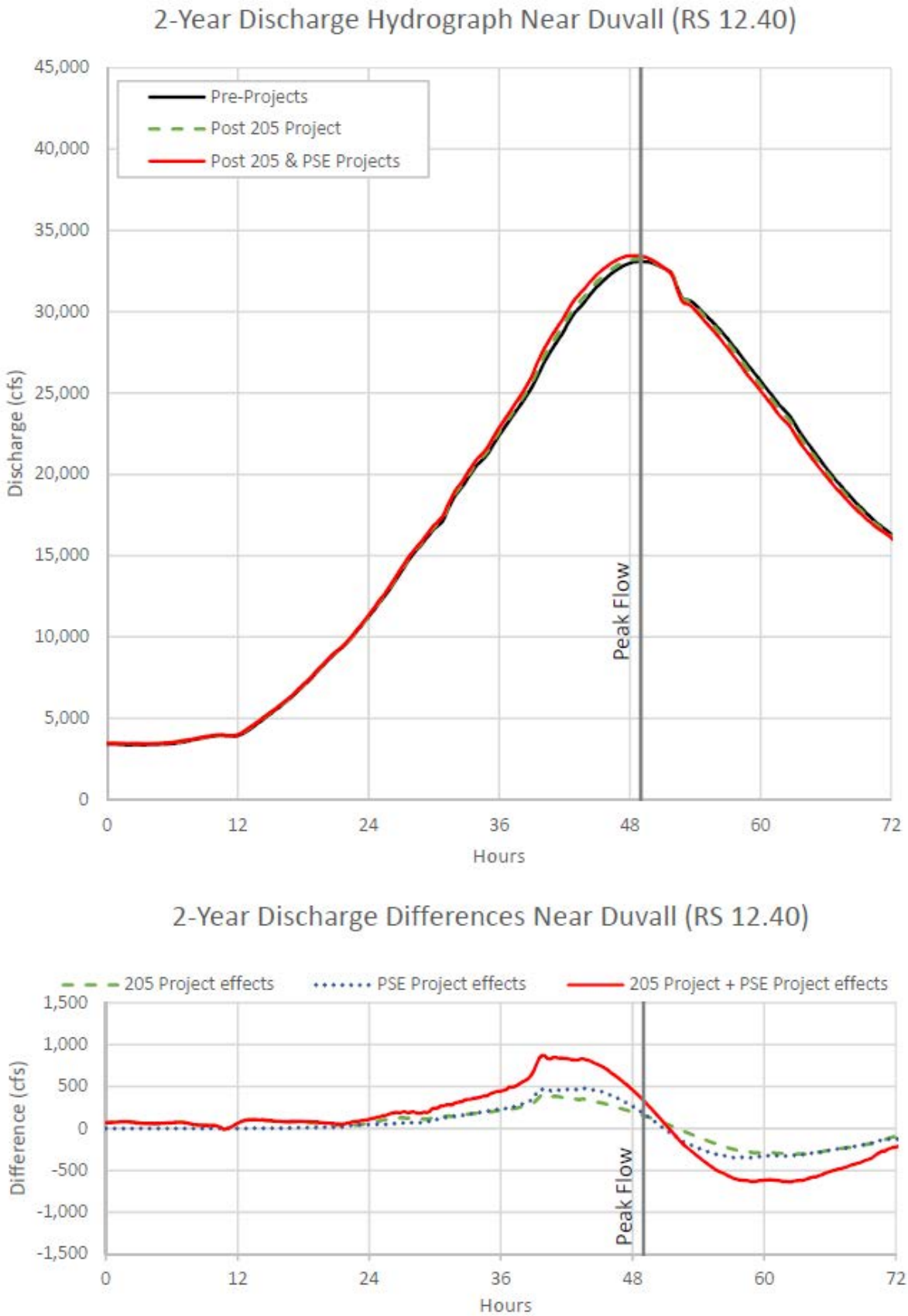
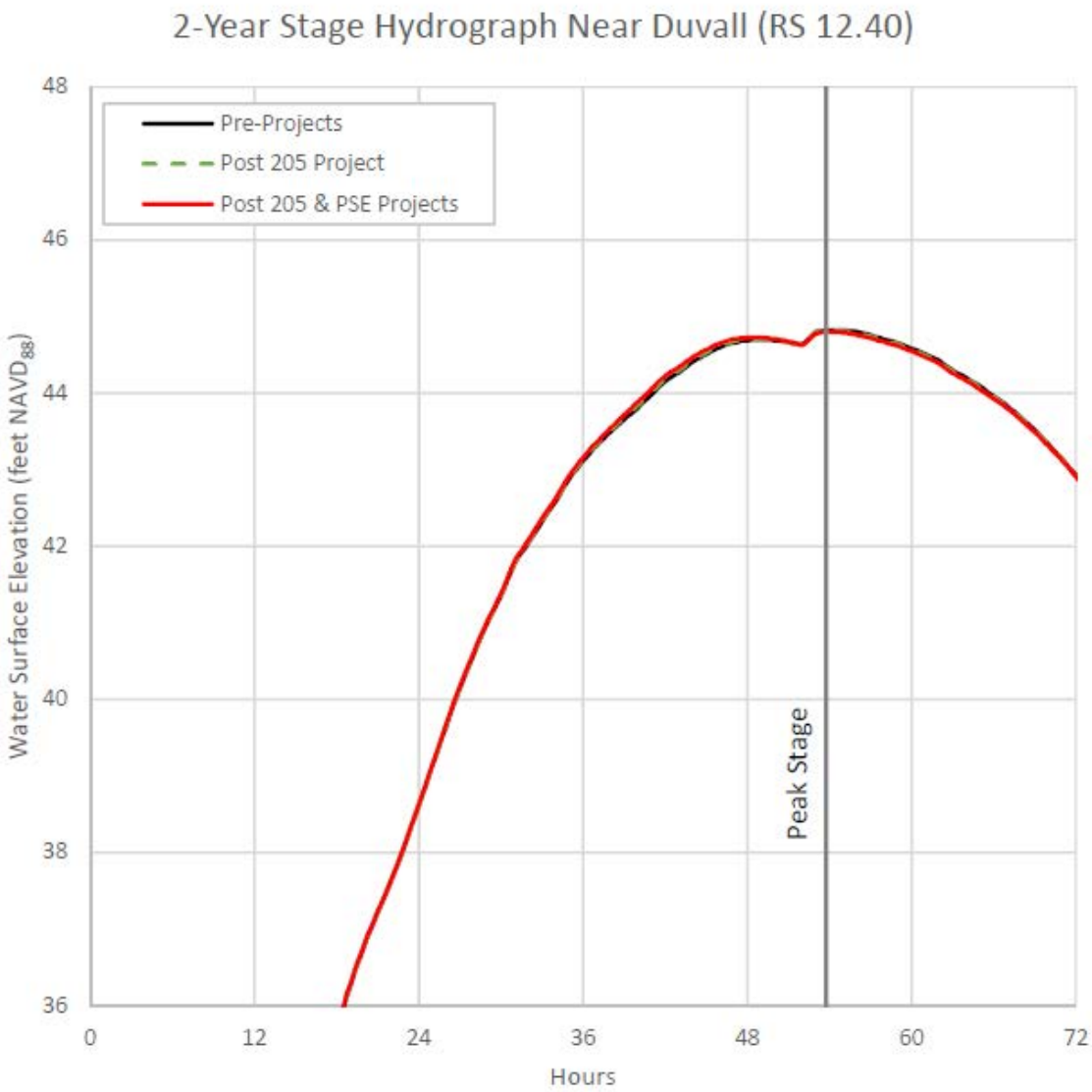


Figure 29. Comparison of 2-Year Flood Discharge Hydrographs and Modeled Differences Caused by the Projects near Duvall.



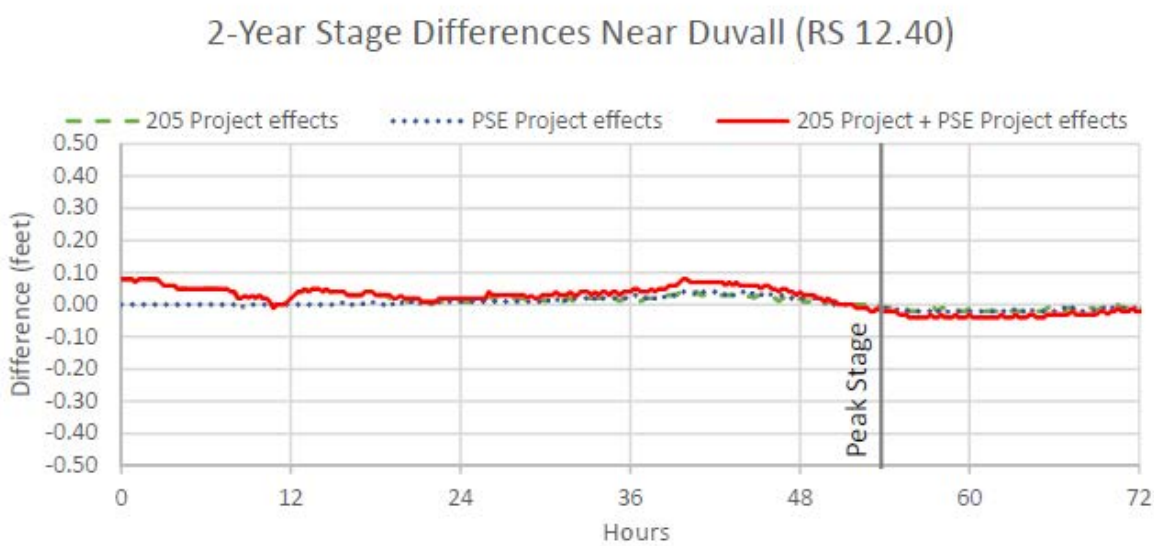


Figure 30. Comparison of 2-Year Flood Stages and Modeled Differences Caused by the Projects near Duvall.

9. SUMMARY AND CONCLUSIONS

Hydraulic modeling shows that the combined effects of the 205 and PSE projects result in a reduction in flood storage and flow attenuation upstream of Snoqualmie Falls and a corresponding increase in peak flows and peak flood stages downstream of the Falls. Hydraulic effects of the two projects were evaluated for events ranging from a 2-year flood to a 500-year flood. Detailed results of the simulations are provided in the form of stage and flow hydrographs and difference plots for locations upstream and downstream of the Falls in Appendix E. On the rising limb of the flood (i.e., when discharge is increasing), the projects reduce flood water levels upstream of the Falls and generally increase downstream flows and water levels. Key findings from the modeling conducted for this study include the following.

- The magnitude of the simulated flow increases during the rising limb of flood event hydrographs downstream of the Falls is dependent on the return period of the flood and the location of interest, but the effects are generally greatest closer to the Falls (i.e., at Fall City) and are most pronounced in a 10-year flood (0.4 foot increase in water level and 2,000 cfs increase in flow).

- With the projects' removal of flow constrictions, simulated upstream water levels in the city of Snoqualmie at Riverview Park are reduced by about 1.4 feet in the 100-year flood event. The corresponding reduction in 100-year flood storage upstream of the Falls is about 2,500 acre-feet, resulting in increased downstream flows. The largest simulated increase in flow in the 100-year flood — about 2,600 cfs — occurs slightly prior to the peak of the flood. At the peak of the 100-year flood event, the simulated increase in discharge compared to the Pre-Projects condition is about 1,260 cfs at the Falls. On the receding limb of the flood, simulated flows are generally lower than in the Pre-Projects condition.

- In the 2-year flood, simulated water levels upstream of the Falls at Riverview Park in Snoqualmie are reduced by about 2.2 feet. The larger change in water levels for the 2-year event (compared to the 100-year event) occurs because the 2-year flows remain primarily within the channel, where stage changes occur more quickly in response to flow changes compared to a floodplain. A much larger change in flow is required across a broad floodplain to cause the same change in stage seen in the channel. For the 2-year event, the greatest simulated increase in downstream flow resulting from the Snoqualmie Falls projects is about 1,000 cfs on the rising limb of the flood hydrograph upstream of Fall City, while the increase in flow at the peak of the flood event is only about 200 cfs at this location.

- The increase in flows passing the Falls due to the projects results in increases in simulated flood stages (water levels) downstream of the Falls. The highest downstream stage increase simulated at any of the evaluation points in the 100-year flood event (0.35 foot) occurs upstream of Fall City (between the Snoqualmie Falls and Twin Rivers golf courses) on the rising limb of the flood hydrograph (see Figure 16). At the peak of the flood, however, the increase is much smaller. The maximum predicted increase in peak 100-year flood stage at any of the downstream locations evaluated is

0.10 foot (see Table 8); below the Tolt River confluence the increase in the simulated maximum 100-year water level is 0.08 foot (i.e., less than 1 inch).

- The projects also result in slightly earlier arrival times (by 15 to 30 minutes) for peak flows and flood stages downstream of the Falls.

The focus of the analyses reported herein is the hydraulic effects of the USACE 205 Project and the PSE Project near Snoqualmie Falls. Although some hydrologic analyses were conducted, as described in Chapter 4, this project did not seek to evaluate changes in flood regime in the Snoqualmie watershed based on analysis of historical gage data and other similar information. A subsequent phase of work, currently underway, will assess whether flood hydrology in the Snoqualmie watershed has changed over time, and, if so, the nature and extent of change as well as the level of confidence in the findings. This second phase of investigation will consider trends in peak annual flows and also changes in hydrology for smaller but still damaging events (e.g., springtime out-of-bank flows). The work will evaluate key factors that may be influencing changes in Snoqualmie River flood hydrology, including climate change, forest practices, land development patterns, and stormwater standards.

10. REFERENCES

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APPENDIX A

Available Data

205 Project Background Information

Of the more than 800 files available from the USACE for the 205 Project, the following were found to be the most useful:

- Reports with descriptions of the design elements included in the 205 Project
- Design drawings for the left bank and right bank excavations
- Demolition plans for the abandoned railroad trestle
- Survey data (described below)

PSE Project Background Information

The following data related to the PSE Project were provided by PSE:

- A report summarizing an alternatives analysis for weir configurations dated May 2007, including results for the scenario constructed by PSE (Alternative 8B)
- Wording from the Federal Energy Regulatory Commission (FERC) 2007 License Amendment describing the proposed project and its benefits
- Combination drawings in pdf format of conditions before the PSE Project, design drawings, and as-built drawings
- Survey data (described below)

Survey Data

Survey data were provided by USACE, PSE, and the County. The surveys span the time period from before the projects began to after the projects were constructed.

Channel Survey

- 2000 USACE Survey – This survey starts about 170 feet upstream from the weir, where the old footbridge used to be. It extends about 1,150 feet upstream, to just downstream from the SR 202 bridge.
- 2005 PSE Survey – Sometimes cited in documents as "The Forebay Survey," this survey starts at the weir and extends about 370 feet upstream.
- PSE cross-section surveys – Sixteen locations upstream from the weir were surveyed over several years (2005, 2007, 2008, 2009, and 2014).

Overbank Survey

- Some USACE drawings include topographic contour lines for the channel banks and surrounding areas. The source of the survey data was not cited, but most show the banks prior to the 205 Project. The maps include 2-foot contour lines.
- King County provided LiDAR data in GIS raster format for the reach upstream from the Falls. LiDAR data were provided for 2002, 2011, and 2013.

Existing Models

Three existing HEC-RAS hydraulic models were used as the basis for this project: one model for downstream of Snoqualmie Falls and two models for upstream of the Falls.

The model for downstream of Snoqualmie Falls was provided by NHC. The upstream extent of the model is approximately the USGS Snoqualmie River at Snoqualmie gage and the model extends downstream to the confluence with the Skykomish River and a short reach of the Snohomish River. This model is an unsteady flow model developed for the 2006 FEMA FIS, then altered to include farm pads that have been permitted and constructed along the Snoqualmie River over the past few years. The model includes farm pads and other projects involving grading in the lower valley floodplain constructed prior to the beginning of 2014. For this project, the downstream end of the model was truncated between Carnation and Duvall near Northeast 138th Street.

The USACE provided a HEC-RAS model of the Snoqualmie River that has a downstream boundary about 180 feet upstream from the PSE weir and an upstream boundary at the confluence of the Middle Fork and South Fork Snoqualmie River. This model represents the Pre-205 Project condition and utilizes the 2000 USACE survey data for the channel described above for the 205 Project. The railroad trestle removed in the 205 Project is included in this model.

The second upstream model was provided by NHC. It is an updated version of the HEC-RAS model provided by the USACE. It was developed for the 2009 Letter of Map Revision request from the City of Snoqualmie to FEMA (Snoqualmie, 2009), and incorporates changes to the channel resulting from the 205 Project. The model geometry was the same as the 2009 LOMR model, except that the cross-sections between the SR 202 bridge and Meadowbrook bridge had been updated with 2011 LiDAR and 2012 in-channel survey data, and the cross-sections between the weir and SR 202 bridge were updated with the 2005 forebay survey and PSE 2005 cross-section survey. The model has the same upstream extent as the USACE model, but extends downstream to include the PSE weir and three cross-sections between Snoqualmie Falls and the weir.

Calibration Data

Two sets of high water marks were available to help calibrate the model upstream from Snoqualmie Falls. The USGS has HWMs published for the January 2009 flood event (Mastin et al., 2010). Also, the City of Snoqualmie had NHC collect HWMs for the January 2015 flood event. Maps showing the location of these marks are provided in Appendix D.

Adequacy of Available Data

The data summarized above were collected and reviewed by WSE. At the time this study was conducted, the available data were deemed adequate by WSE for the purposes of the study.

APPENDIX B

Sum of the Forks

The project area for this study is located just downstream of the confluence of the South Fork and Middle Fork of the Snoqualmie River. The North Fork of the Snoqualmie River joins the Middle Fork less than 1 mile upstream of this point. To evaluate the hydrology of the Snoqualmie River and the potential effects of the 205 and PSE projects, estimated flows upstream of the project reach were needed. Gage data for the mainstem of the Snoqualmie River just downstream of the confluences of the three forks are not available. However, there are USGS gages located on each of the upstream forks. Data from these gages were used to develop a "Sum of the Forks" record, which was then used to investigate trends in the historical flows as described below. A similar Sum of the Forks calculation is used in the King County flood warning system to set flood phases and provide advanced warning of certain flooding to downstream residents.

Peak flow frequency results for gages on the North, Middle, and South Forks of the Snoqualmie River are reported in Table B-1. Frequency analysis is most often conducted on instantaneous peak flows. However, because peak flows on the three forks do not always occur at the same time or even on the same day, the combined peak flow from the three forks cannot be estimated by summing flow frequency results for the different gages; the summed concurrent flow is needed. Prior to 1987, concurrent instantaneous or short-interval (hourly) data are not readily available for the three forks. Therefore, in order to complete an analysis for the entire period of record, the summed flow for the three gages was developed by adding the concurrent mean daily flows from each gage. The annual maximum mean daily discharges were then determined and subject to frequency analysis (in the same manner as peak flows are typically analyzed) to evaluate hydrologic trends. The Sum of the Forks flow used herein is different from that used in the flood warning system, which considers real-time data and historical data only since 1987 and, therefore, uses 15-minute data (which are first available starting in 1987) instead of mean daily flows.

Table B-1. Flow Frequency Results for North Fork, Middle Fork, and South Fork Snoqualmie River Derived from USGS Gages on Each Fork.

Return Flow Frequency	Flow (cfs)					
	North Fork (USGS 12142000)		Middle Fork (USGS 12141300)		South Fork (USGS 12143400)	
	1931–2003	1931–2013	1962–2003	1962–2013	1961–2003	1961–2013
10-year	12,300	12,600	26,100	26,700	6,900	7,000
50-year	16,300	16,900	33,100	34,200	9,600	9,900
100-year	17,900	18,700	35,700	37,100	10,700	11,000
500-year	21,500	22,700	41,000	43,100	13,200	13,700

The concurrent mean daily flows used in this analysis are smaller than flows that will be seen in the project reach. Mean daily flows are by definition smaller than peak discharges. Also, the basin area at each gage is smaller than the basin area at the confluence, as shown in Table B-2.

Table B-2. Drainage Areas for Three Forks.		
Gage	**Drainage Area at Gage (mi^2)**	**Drainage Area at Mouth of Fork (mi^2)**
North Fork Snoqualmie River (USGS 12142000)	41.6	103.69
Middle Fork Snoqualmie River (USGS 12141300)	154	171.2
South Fork Snoqualmie River (USGS 12143400)	64	86.12

Approach

The three gages representing each upstream fork of the Snoqualmie River were aggregated into a single composite record to represent flow upstream of Snoqualmie Falls. This record is termed the "Sum of the Forks." To calculate the Sum of the Forks, the mean daily flows at each gage were summed and the maximum mean daily flow for each year was extracted from the dataset. A flood frequency analysis was then conducted on the annual maximum mean daily flows for the Sum of the Forks.

Analysis

The USACE Hydrologic Engineering Centers Statistical Software Package was used to conduct a frequency analysis of the maximum mean daily flows. A log-Pearson Type III distribution was fit to the data using the methods of Bulletin 17B (USGS 1982). Analyses were conducted for two periods of record: WY 1962 to 2003 and WY 1962 to 2013.

Results

The results of this analysis are summarized in Table B-3. Detailed outputs from the frequency analysis are available elsewhere (WSE 2015b).

Table B-3. Return Period Flows for the Annual Maximum Mean Daily Flow for Sum of the Forks.			
Return Period	**Flows (cfs)**		**Increase with Inclusion of Flows from 2003–2013**
	1962–2003	**1962–2013**	
10-year	30,300	31,800	4.9%
50-year	40,200	43,300	7.9%
100-year	44,100	48,100	9.2%
500-year	52,800	59,300	12.2%

APPENDIX C

Flood Frequency Supporting Data

Snoqualmie River Near Carnation (USGS Gage 12149000) – 1930–2003

<< Low Outlier Test >>

Based on 74 events, 10 percent outlier test deviate K(N) = 2.912
Computed low outlier test value = 8,038.95
0 low outlier(s) identified below test value of 8,038.95

<< High Outlier Test >>

Based on 74 events, 10 percent outlier test deviate K(N) = 2.912
Computed high outlier test value = 108,247.99
0 high outlier(s) identified above test value of 108,247.99

<< Plotting Positions >>

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
14	Dec	1929	14,800.00	1	1991	65,200.00	0.94
28	Jan	1931	27,400.00	2	1996	61,600.00	2.28
27	Feb	1932	59,500.00	3	1932	59,500.00	3.63
13	Nov	1932	59,000.00	4	1933	59,000.00	4.97
3	Nov	1933	48,700.00	5	1987	57,100.00	6.32
25	Oct	1934	47,100.00	6	1951	52,200.00	7.66
16	May	1936	16,100.00	7	1976	52,100.00	9.01
15	Apr	1937	17,200.00	8	1969	50,600.00	10.35
18	Apr	1938	38,800.00	9	1960	49,400.00	11.69
8	Dec	1938	22,900.00	10	1990	48,900.00	13.04
16	Dec	1939	18,900.00	11	1934	48,700.00	14.38
29	Nov	1940	20,000.00	12	1944	48,400.00	15.73
19	Dec	1941	19,700.00	13	1975	48,300.00	17.07
24	Nov	1942	30,300.00	14	1978	47,600.00	18.41
3	Dec	1943	48,400.00	15	1935	47,100.00	19.76
8	Jan	1945	32,000.00	16	1997	45,000.00	21.1
26	Oct	1945	23,500.00	17	1968	43,400.00	22.45
11	Dec	1946	32,600.00	18	1984	42,600.00	23.79
19	Oct	1947	24,800.00	19	1995	41,800.00	25.13
24	Nov	1948	20,200.00	20	1972	41,800.00	26.48
4	Mar	1950	30,100.00	21	1965	41,600.00	27.82
10	Feb	1951	52,200.00	22	1956	40,800.00	29.17
4	Feb	1952	14,300.00	23	1982	39,300.00	30.51
1	Feb	1953	32,400.00	24	1981	39,300.00	31.85
10	Dec	1953	35,700.00	25	1938	38,800.00	33.2
8	Feb	1955	25,300.00	26	1999	37,800.00	34.54
12	Dec	1955	40,800.00	27	1963	37,800.00	35.89
10	Dec	1956	27,500.00	28	1974	37,600.00	37.23
17	Jan	1958	15,400.00	29	1980	37,400.00	38.58
24	Jan	1959	31,200.00	30	1954	35,700.00	39.92

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
23	Nov	1959	49,400.00	31	1983	35,300.00	41.26
22	Feb	1961	28,000.00	32	2000	35,100.00	42.61
7	Jan	1962	20,400.00	33	1947	32,600.00	43.95
20	Nov	1962	37,800.00	34	1953	32,400.00	45.3
2	Jan	1964	17,400.00	35	1989	32,000.00	46.64
29	Jan	1965	41,600.00	36	1945	32,000.00	47.98
7	May	1966	12,400.00	37	1959	31,200.00	49.33
14	Dec	1966	22,700.00	38	1943	30,300.00	50.67
26	Dec	1967	43,400.00	39	1950	30,100.00	52.02
5	Jan	1969	50,600.00	40	1986	28,800.00	53.36
23	Jan	1970	13,400.00	41	1961	28,000.00	54.7
20	Jan	1971	21,100.00	42	1993	27,800.00	56.05
28	Feb	1972	41,800.00	43	1957	27,500.00	57.39
26	Dec	1972	24,500.00	44	1931	27,400.00	58.74
16	Jan	1974	37,600.00	45	2002	26,000.00	60.08
18	Jan	1975	48,300.00	46	2003	25,800.00	61.42
3	Dec	1975	52,100.00	47	1955	25,300.00	62.77
18	Jan	1977	18,400.00	48	1948	24,800.00	64.11
2	Dec	1977	47,600.00	49	1973	24,500.00	65.46
7	Mar	1979	16,500.00	50	1946	23,500.00	66.8
15	Dec	1979	37,400.00	51	1939	22,900.00	68.15
26	Dec	1980	39,300.00	52	1967	22,700.00	69.49
15	Feb	1982	39,300.00	53	1971	21,100.00	70.83
4	Dec	1982	35,300.00	54	1962	20,400.00	72.18
25	Jan	1984	42,600.00	55	1949	20,200.00	73.52
8	Jun	1985	18,500.00	56	1941	20,000.00	74.87
24	Feb	1986	28,800.00	57	1942	19,700.00	76.21
24	Nov	1986	57,100.00	58	1998	19,000.00	77.55
10	Dec	1987	16,600.00	59	1940	18,900.00	78.9
17	Oct	1988	32,000.00	60	1985	18,500.00	80.24
10	Jan	1990	48,900.00	61	1977	18,400.00	81.59
24	Nov	1990	65,200.00	62	1992	18,000.00	82.93
29	Jan	1992	18,000.00	63	1964	17,400.00	84.27
26	Jan	1993	27,800.00	64	1937	17,200.00	85.62
2	Dec	1993	11,300.00	65	1988	16,600.00	86.96
20	Feb	1995	41,800.00	66	1979	16,500.00	88.31
9	Feb	1996	61,600.00	67	2001	16,300.00	89.65
20	Mar	1997	45,000.00	68	1936	16,100.00	90.99
31	Oct	1997	19,000.00	69	1958	15,400.00	92.34
30	Dec	1998	37,800.00	70	1930	14,800.00	93.68
16	Dec	1999	35,100.00	71	1952	14,300.00	95.03

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
1	Oct	2000	16,300.00	72	1970	13,400.00	96.37
15	Nov	2001	26,000.00	73	1966	12,400.00	97.72
1	Feb	2003	25,800.00	74	1994	11,300.00	99.06

<< Skew Weighting >>

Based on 74 events, mean-square error of station skew = 0.082

Mean-square error of regional skew = 0.302

<< Frequency Curve >>

Computed Curve	Expected Probability	Percent Chance Exceedance	Confidence Limits	
			0.05	0.095
FLOW (cfs)			FLOW (cfs)	
98122.1	102474.3	0.2	121092.5	83200.7
87333	90304.8	0.5	106025.3	74945.8
79211.8	81353.5	1	94880.9	68643.6
71088.9	72554.9	2	83922.8	62251.4
60255.4	61055.6	5	69642.3	53558.9
51862.3	52301.3	10	58888.6	46658
43078	43272	20	47993.9	39222.5
29838.4	29838.4	50	32531	27380.4
20336	20234.3	80	22327.1	18264.1
16533.8	16372.6	90	18398.4	14536.9
13888.3	13671.8	95	15678	11957.9
9929.6	9594.5	99	11574.9	8175.1

<< Systematic Statistics >>

Log Transforms:			
Flow (cfs)		Number of Events	
Mean	4.47	Historical Events	0
Standard Dev	0.194	High Events	0
Station Skew	-0.196	Low Outliers	0
Regional Skew	0.0	Zero Events	0
Weighted Skew	-0.154	Missing Events	0
Adopted Skew	-0.154	Systematic Events	74

Snoqualmie River Near Carnation (USGS Gage 12149000) – 1930–2013

<< Low Outlier Test >>

Based on 84 events, 10 percent outlier test deviate K(N) = 2.957
Computed low outlier test value = 7,683.2
0 low outlier(s) identified below test value of 7,683.2

<< High Outlier Test >>

Based on 84 events, 10 percent outlier test deviate K(N) = 2.957
Computed high outlier test value = 118,513.19
0 high outlier(s) identified above test value of 118,513.19

<< Plotting Positions >>

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
14	Dec	1929	14,800.00	1	2009	82,900.00	0.83
28	Jan	1931	27,400.00	2	2007	71,800.00	2.01
27	Feb	1932	59,500.00	3	1991	65,200.00	3.2
13	Nov	1932	59,000.00	4	1996	61,600.00	4.38
3	Nov	1933	48,700.00	5	1932	59,500.00	5.57
25	Oct	1934	47,100.00	6	1933	59,000.00	6.75
16	May	1936	16,100.00	7	1987	57,100.00	7.94
15	Apr	1937	17,200.00	8	1951	52,200.00	9.12
18	Apr	1938	38,800.00	9	1976	52,100.00	10.31
8	Dec	1938	22,900.00	10	2011	51,600.00	11.49
16	Dec	1939	18,900.00	11	1969	50,600.00	12.68
29	Nov	1940	20,000.00	12	2005	49,400.00	13.86
19	Dec	1941	19,700.00	13	1960	49,400.00	15.05
24	Nov	1942	30,300.00	14	1990	48,900.00	16.23
3	Dec	1943	48,400.00	15	1934	48,700.00	17.42
8	Jan	1945	32,000.00	16	1944	48,400.00	18.6
26	Oct	1945	23,500.00	17	1975	48,300.00	19.79
11	Dec	1946	32,600.00	18	1978	47,600.00	20.97
19	Oct	1947	24,800.00	19	1935	47,100.00	22.16
24	Nov	1948	20,200.00	20	1997	45,000.00	23.34
4	Mar	1950	30,100.00	21	1968	43,400.00	24.53
10	Feb	1951	52,200.00	22	1984	42,600.00	25.71
4	Feb	1952	14,300.00	23	1995	41,800.00	26.9
1	Feb	1953	32,400.00	24	1972	41,800.00	28.08
10	Dec	1953	35,700.00	25	1965	41,600.00	29.27
8	Feb	1955	25,300.00	26	1956	40,800.00	30.45
12	Dec	1955	40,800.00	27	1982	39,300.00	31.64
10	Dec	1956	27,500.00	28	1981	39,300.00	32.82
17	Jan	1958	15,400.00	29	1938	38,800.00	34
24	Jan	1959	31,200.00	30	1999	37,800.00	35.19

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
23	Nov	1959	49,400.00	31	1963	37,800.00	36.37
22	Feb	1961	28,000.00	32	1974	37,600.00	37.56
7	Jan	1962	20,400.00	33	1980	37,400.00	38.74
20	Nov	1962	37,800.00	34	1954	35,700.00	39.93
2	Jan	1964	17,400.00	35	1983	35,300.00	41.11
29	Jan	1965	41,600.00	36	2000	35,100.00	42.3
7	May	1966	12,400.00	37	2004	33,800.00	43.48
14	Dec	1966	22,700.00	38	2012	33,100.00	44.67
26	Dec	1967	43,400.00	39	1947	32,600.00	45.85
5	Jan	1969	50,600.00	40	1953	32,400.00	47.04
23	Jan	1970	13,400.00	41	2008	32,100.00	48.22
20	Jan	1971	21,100.00	42	1989	32,000.00	49.41
28	Feb	1972	41,800.00	43	1945	32,000.00	50.59
26	Dec	1972	24,500.00	44	1959	31,200.00	51.78
16	Jan	1974	37,600.00	45	1943	30,300.00	52.96
18	Jan	1975	48,300.00	46	1950	30,100.00	54.15
3	Dec	1975	52,100.00	47	1986	28,800.00	55.33
18	Jan	1977	18,400.00	48	1961	28,000.00	56.52
2	Dec	1977	47,600.00	49	1993	27,800.00	57.7
7	Mar	1979	16,500.00	50	1957	27,500.00	58.89
15	Dec	1979	37,400.00	51	1931	27,400.00	60.07
26	Dec	1980	39,300.00	52	2002	26,000.00	61.26
15	Feb	1982	39,300.00	53	2003	25,800.00	62.44
4	Dec	1982	35,300.00	54	1955	25,300.00	63.63
25	Jan	1984	42,600.00	55	1948	24,800.00	64.81
8	Jun	1985	18,500.00	56	1973	24,500.00	66
24	Feb	1986	28,800.00	57	1946	23,500.00	67.18
24	Nov	1986	57,100.00	58	2006	23,400.00	68.36
10	Dec	1987	16,600.00	59	1939	22,900.00	69.55
17	Oct	1988	32,000.00	60	1967	22,700.00	70.73
10	Jan	1990	48,900.00	61	1971	21,100.00	71.92
24	Nov	1990	65,200.00	62	1962	20,400.00	73.1
29	Jan	1992	18,000.00	63	1949	20,200.00	74.29
26	Jan	1993	27,800.00	64	1941	20,000.00	75.47
2	Dec	1993	11,300.00	65	1942	19,700.00	76.66
20	Feb	1995	41,800.00	66	1998	19,000.00	77.84
9	Feb	1996	61,600.00	67	1940	18,900.00	79.03
20	Mar	1997	45,000.00	68	1985	18,500.00	80.21
31	Oct	1997	19,000.00	69	1977	18,400.00	81.4
30	Dec	1998	37,800.00	70	1992	18,000.00	82.58
16	Dec	1999	35,100.00	71	2010	17,400.00	83.77

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
1	Oct	2000	16,300.00	72	1964	17,400.00	84.95
15	Nov	2001	26,000.00	73	1937	17,200.00	86.14
1	Feb	2003	25,800.00	74	1988	16,600.00	87.32
19	Nov	2003	33,800.00	75	1979	16,500.00	88.51
19	Jan	2005	49,400.00	76	2001	16,300.00	89.69
11	Jan	2006	23,400.00	77	1936	16,100.00	90.88
7	Nov	2006	71,800.00	78	1958	15,400.00	92.06
4	Dec	2007	32,100.00	79	2013	15,100.00	93.25
8	Jan	2009	82,900.00	80	1930	14,800.00	94.43
26	Oct	2009	17,400.00	81	1952	14,300.00	95.62
17	Jan	2011	51,600.00	82	1970	13,400.00	96.8
22	Feb	2012	33,100.00	83	1966	12,400.00	97.99
20	Nov	2012	15,100.00	84	1994	11,300.00	99.17

<< Skew Weighting >>

Based on 84 events, mean-square error of station skew = 0.068

Mean-square error of regional skew = 0.302

<< Frequency Curve >>

Computed Curve	Expected Probability	Percent Chance Exceedance	Confidence Limits	
			0.05	0.095
FLOW (cfs)			FLOW (cfs)	
109,282.20	114,052.70	0.2	134,568.80	92,613.50
95,972.80	99,165.60	0.5	116,181.50	82,377.90
86,149.70	88,410.40	1	102,852.10	74,712.20
76,497.00	78,017.90	2	89,978.60	67,071.20
63,901.00	64,709.00	5	73,565.70	56,903.40
54,369.00	54,804.70	10	61,491.00	49,019.00
44,613.80	44,802.10	20	49,521.80	40,716.30
30,360.80	30,360.80	50	33,015.30	27,925.50
20,483.10	20,392.00	80	22,439.70	18,458.50
16,616.20	16,473.50	90	18,440.10	14,679.60
13,954.20	13,763.60	95	15,697.20	12,091.60
10,012.20	9,721.10	99	11,604.30	8,328.40

<< Systematic Statistics >>

Log Transforms:			
Flow (cfs)		Number of Events	
Mean	4.48	Historical Events	0
Standard Dev	0.201	High Events	0
Station Skew	-0.098	Low Outliers	0
Regional Skew	0.0	Zero Events	0
Weighted Skew	-0.08	Missing Events	0
Adopted Skew	-0.08	Systematic Events	84

Snoqualmie River Near Snoqualmie (USGS Gage 12144500) – 1959–2003

<< Low Outlier Test >>

Based on 45 events, 10 percent outlier test deviate K(N) = 2.727
Computed low outlier test value = 8,181.67
0 low outlier(s) identified below test value of 8,181.67

<< High Outlier Test >>

Based on 45 events, 10 percent outlier test deviate K(N) = 2.727
Computed high outlier test value = 106,330.91
0 high outlier(s) identified above test value of 106,330.91

<< Plotting Positions >>

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
12	Nov	1958	26,000	1	1991	78,800	1.54
23	Nov	1959	61,000	2	1960	61,000	3.74
21	Feb	1961	26,600	3	1987	58,100	5.95
7	Jan	1962	18,800	4	1978	53,800	8.15
20	Nov	1962	43,200	5	1976	51,800	10.35
1	Jan	1964	15,900	6	1996	51,700	12.56
29	Jan	1965	35,800	7	1975	48,100	14.76
6	May	1966	11,600	8	1990	44,000	16.96
13	Dec	1966	20,800	9	1963	43,200	19.16
25	Dec	1967	36,000	10	1981	42,600	21.37
5	Jan	1969	41,500	11	1969	41,500	23.57
9	Apr	1970	13,300	12	1984	40,600	25.77
19	Jan	1971	27,200	13	1989	40,400	27.97
28	Feb	1972	32,500	14	1982	39,900	30.18
26	Dec	1972	26,200	15	1980	37,600	32.38
15	Jan	1974	32,100	16	1983	37,000	34.58
18	Jan	1975	48,100	17	1968	36,000	36.78
3	Dec	1975	51,800	18	1965	35,800	38.99
18	Jan	1977	17,800	19	1995	34,200	41.19
2	Dec	1977	53,800	20	2000	33,000	43.39
7	Mar	1979	14,200	21	1986	32,500	45.59
15	Dec	1979	37,600	22	1972	32,500	47.8
26	Dec	1980	42,600	23	1974	32,100	50
24	Jan	1982	39,900	24	1999	31,600	52.2
4	Dec	1982	37,000	25	1997	30,400	54.41
25	Jan	1984	40,600	26	1971	27,200	56.61
7	Jun	1985	18,000	27	1993	27,100	58.81
24	Feb	1986	32,500	28	1961	26,600	61.01
24	Nov	1986	58,100	29	1973	26,200	63.22
10	Dec	1987	22,800	30	1959	26,000	65.42

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
16	Oct	1988	40,400	31	2003	24,400	67.62
9	Nov	1989	44,000	32	2002	23,800	69.82
24	Nov	1990	78,800	33	1998	23,600	72.03
5	Dec	1991	17,700	34	1988	22,800	74.23
25	Jan	1993	27,100	35	1967	20,800	76.43
3	Mar	1994	9,280	36	1962	18,800	78.63
19	Feb	1995	34,200	37	1985	18,000	80.84
9	Feb	1996	51,700	38	1977	17,800	83.04
19	Mar	1997	30,400	39	1992	17,700	85.24
30	Oct	1997	23,600	40	2001	16,400	87.44
30	Dec	1998	31,600	41	1964	15,900	89.65
15	Dec	1999	33,000	42	1979	14,200	91.85
1	Oct	2000	16,400	43	1970	13,300	94.05
14	Apr	2002	23,800	44	1966	11,600	96.26
31	Jan	2003	24,400	45	1994	9,280	98.46

<< Skew Weighting >>

Based on 45 events, mean-square error of station skew = 0.14

Mean-square error of regional skew = 0.302

<< Frequency Curve >>

Computed Curve	Expected Probability	Percent Chance Exceedance	Confidence Limits	
			0.05	0.095
FLOW (cfs)			FLOW (cfs)	
99,285.20	106,420.90	0.2	132,337.40	80,450.40
88,820.30	93,735.40	0.5	115,914.80	72,989.10
80,825.40	84,400.00	1	103,647.80	67,190.80
72,726.40	75,192.00	2	91,492.40	61,215.90
61,763.50	63,128.00	5	75,527.80	52,932.30
53,145.50	53,896.30	10	63,435.30	46,219.40
44,014.30	44,348.20	20	51,157.40	38,842.60
30,069.50	30,069.50	50	33,828.10	26,761.30
19,989.00	19,809.90	80	22,634.60	17,222.30
15,968.30	15,685.70	90	18,402.90	13,326.80
13,187.80	12,811.90	95	15,496.70	10,664.20
9,078.80	8,508.30	99	11,153.80	6,857.60

<< Systematic Statistics >>

Log Transforms:			
Flow (cfs)		Number of Events	
Mean	4.47	Historical Events	0
Standard Dev	0.204	High Events	0
Station Skew	-0.361	Low Outliers	0
Regional Skew	0.0	Zero Events	0
Weighted Skew	-0.247	Missing Events	0
Adopted Skew	-0.247	Systematic Events	45

Snoqualmie River Near Snoqualmie (USGS Gage 12144500) – 1959–2013

<< Low Outlier Test >>

Based on 55 events, 10 percent outlier test deviate K(N) = 2.804
Computed low outlier test value = 8,052.53
0 low outlier(s) identified below test value of 8,052.53

<< High Outlier Test >>

Based on 55 events, 10 percent outlier test deviate K(N) = 2.804
Computed high outlier test value = 108,219.2
0 high outlier(s) identified above test value of 108,219.2

<< Plotting Positions >>

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
12	Nov	1958	26,000	1	1991	78,800	1.26
23	Nov	1959	61,000	2	1960	61,000	3.07
21	Feb	1961	26,600	3	2009	60,700	4.87
7	Jan	1962	18,800	4	1987	58,100	6.68
20	Nov	1962	43,200	5	2007	55,000	8.48
1	Jan	1964	15,900	6	1978	53,800	10.29
29	Jan	1965	35,800	7	1976	51,800	12.09
6	May	1966	11,600	8	1996	51,700	13.9
13	Dec	1966	20,800	9	1975	48,100	15.7
25	Dec	1967	36,000	10	1990	44,000	17.51
5	Jan	1969	41,500	11	1963	43,200	19.31
9	Apr	1970	13,300	12	1981	42,600	21.12
19	Jan	1971	27,200	13	1969	41,500	22.92
28	Feb	1972	32,500	14	1984	40,600	24.73
26	Dec	1972	26,200	15	1989	40,400	26.53
15	Jan	1974	32,100	16	1982	39,900	28.34
18	Jan	1975	48,100	17	2011	37,900	30.14
3	Dec	1975	51,800	18	1980	37,600	31.95
18	Jan	1977	17,800	19	2005	37,100	33.75
2	Dec	1977	53,800	20	1983	37,000	35.56
7	Mar	1979	14,200	21	1968	36,000	37.36
15	Dec	1979	37,600	22	1965	35,800	39.17
26	Dec	1980	42,600	23	1995	34,200	40.97
24	Jan	1982	39,900	24	2000	33,000	42.78
4	Dec	1982	37,000	25	1986	32,500	44.58
25	Jan	1984	40,600	26	1972	32,500	46.39
7	Jun	1985	18,000	27	1974	32,100	48.19
24	Feb	1986	32,500	28	1999	31,600	50
24	Nov	1986	58,100	29	1997	30,400	51.81
10	Dec	1987	22,800	30	2004	29,200	53.61

Day	Month	Year	Peak Flow (cfs)	Rank	Water Year	Peak Flow (cfs)	Median Plotting Position
16	Oct	1988	40,400	31	2012	28,800	55.42
9	Nov	1989	44,000	32	1971	27,200	57.22
24	Nov	1990	78,800	33	1993	27,100	59.03
5	Dec	1991	17,700	34	1961	26,600	60.83
25	Jan	1993	27,100	35	1973	26,200	62.64
3	Mar	1994	9,280	36	1959	26,000	64.44
19	Feb	1995	34,200	37	2003	24,400	66.25
9	Feb	1996	51,700	38	2002	23,800	68.05
19	Mar	1997	30,400	39	1998	23,600	69.86
30	Oct	1997	23,600	40	2008	23,100	71.66
30	Dec	1998	31,600	41	1988	22,800	73.47
15	Dec	1999	33,000	42	1967	20,800	75.27
1	Oct	2000	16,400	43	2010	19,500	77.08
14	Apr	2002	23,800	44	1962	18,800	78.88
31	Jan	2003	24,400	45	2006	18,700	80.69
21	Oct	2003	29,200	46	1985	18,000	82.49
18	Jan	2005	37,100	47	1977	17,800	84.3
10	Jan	2006	18,700	48	1992	17,700	86.1
7	Nov	2006	55,000	49	2001	16,400	87.91
3	Dec	2007	23,100	50	1964	15,900	89.71
7	Jan	2009	60,700	51	2013	15,700	91.52
26	Oct	2009	19,500	52	1979	14,200	93.32
16	Jan	2011	37,900	53	1970	13,300	95.13
22	Feb	2012	28,800	54	1966	11,600	96.93
19	Nov	2012	15,700	55	1994	9,280	98.74

<< Skew Weighting >>

Based on 55 events, mean-square error of station skew = 0.112

Mean-square error of regional skew = 0.302

<< Frequency Curve >>

Computed Curve	Expected Probability	Percent Chance Exceedance	Confidence Limits	
			0.05	0.095
FLOW (cfs)			FLOW (cfs)	
100,347.30	106,399.70	0.2	129,583.30	82,750.50
89,384.50	93,524.50	0.5	113,205.90	74,710.50
81,088.30	84,078.90	1	101,065.60	68,527.50
72,753.80	74,804.00	2	89,113.00	62,215.90
61,583.10	62,706.10	5	73,526.70	53,566.00
52,889.90	53,505.30	10	61,798.60	46,642.50
43,761.30	44,033.20	20	49,948.40	39,124.10
29,970.20	29,970.20	50	33,271.10	27,017.80

Computed Curve	Expected Probability	Percent Chance Exceedance	Confidence Limits	
			0.05	0.095
FLOW (cfs)			FLOW (cfs)	
20,090.20	19,947.70	80	22,459.80	17,618.80
16,158.60	15,933.60	90	18,353.10	13,792.60
13,437.40	13,137.30	95	15,527.40	11,167.80
9,401.60	8,943.10	99	11,295.40	7,378.40

<< Systematic Statistics >>

Log Transforms:			
Flow (cfs)		Number of Events	
Mean	4.47	Historical Events	0
Standard Dev	0.201	High Events	0
Station Skew	-0.269	Low Outliers	0
Regional Skew	0	Zero Events	0
Weighted Skew	-0.196	Missing Events	0
Adopted Skew	-0.196	Systematic Events	55

Sum of the Forks – 1962–2003

<< Low Outlier Test >>

Based on 40 events, 10 percent outlier test deviate K(N) = 2.682
Computed low outlier test value = 5,398.1042
0 low outlier(s) identified below test value of 5,398.1042

<< High Outlier Test >>

Based on 40 events, 10 percent outlier test deviate K(N) = 2.682
Computed high outlier test value = 56,949.2376
0 high outlier(s) identified above test value of 56,949.2376

<< Plotting Positions >>

Day	Month	Year	Maximum Mean Daily Flow (cfs)	Rank	Water Year	Maximum Mean Daily (cfs)	Median Plotting Position
2	Jan	1962	15,160	1	1991	39,570	1.73
20	Nov	1962	17,350	2	1996	36,140	4.21
1	Jan	1964	9,870	3	1976	32,120	6.68
1	Dec	1964	18,760	4	1978	31,130	9.16
6	May	1966	9,540	5	1987	27,200	11.63
13	Dec	1966	15,360	6	1982	25,930	14.11
25	Dec	1967	22,570	7	1999	25,380	16.58
5	Jan	1969	23,650	8	1989	25,380	19.06
23	Jan	1970	8,250	9	1981	24,430	21.53
19	Jan	1971	14,710	10	1975	24,180	24.01
28	Feb	1972	20,370	11	1969	23,650	26.49
26	Dec	1972	17,190	12	1995	23,600	28.96
15	Jan	1974	20,950	13	1968	22,570	31.44
18	Jan	1975	24,180	14	1997	22,320	33.91
2	Dec	1975	32,120	15	1974	20,950	36.39
18	Jan	1977	13,430	16	1984	20,750	38.86
2	Dec	1977	31,130	17	1983	20,620	41.34
6	Mar	1979	11,690	18	1972	20,370	43.81
15	Dec	1979	19,400	19	1980	19,400	46.29
26	Dec	1980	24,430	20	2000	19,230	48.76
14	Feb	1982	25,930	21	2003	19,150	51.24
3	Dec	1982	20,620	22	1965	18,760	53.71
4	Jan	1984	20,750	23	1963	17,350	56.19
7	Jun	1985	13,660	24	1973	17,190	58.66
24	Feb	1986	13,660	25	2002	16,680	61.14
23	Nov	1986	27,200	26	1967	15,360	63.61
6	Apr	1988	10,580	27	1962	15,160	66.09
16	Oct	1988	25,380	28	1971	14,710	68.56
24	Nov	1990	39,570	29	1998	13,660	71.04
25	Jan	1993	6,890	30	1986	13,660	73.51

Day	Month	Year	Maximum Mean Daily Flow (cfs)	Rank	Water Year	Maximum Mean Daily (cfs)	Median Plotting Position
3	Mar	1994	6,740	31	1985	13,660	75.99
19	Feb	1995	23,600	32	1977	13,430	78.47
29	Nov	1995	36,140	33	1979	11,690	80.94
19	Mar	1997	22,320	34	1988	10,580	83.42
30	Oct	1997	13,660	35	1964	9,870	85.89
29	Dec	1998	25,380	36	1966	9,540	88.37
15	Dec	1999	19,230	37	2001	8,270	90.84
1	Oct	2000	8,270	38	1970	8,250	93.32
14	Nov	2001	16,680	39	1993	6,890	95.79
31	Jan	2003	19,150	40	1994	6,740	98.27

<< Skew Weighting >>

Based on 40 events, mean-square error of station skew = 0.164

Mean-square error of regional skew = 0.302

<< Frequency Curve >>

Computed Curve	Expected Probability	Percent Chance Exceedance	Confidence Limits	
			0.05	0.095
FLOW (cfs)			FLOW (cfs)	
52,829.73	56,567.53	0.2	70,049.43	43,129.73
47,924.47	50,554.02	0.5	62,298.20	39,629.85
44,101.08	46,048.01	1	56,385.04	36,857.24
40,156.05	41,523.20	2	50,412.39	33,949.09
34,691.71	35,469.97	5	42,379.75	29,825.73
30,286.55	30,723.04	10	36,136.99	26,400.22
25,502.28	25,701.24	20	29,640.44	22,540.67
17,929.15	17,929.15	50	20,157.58	15,972.68
12,212.27	12,094.91	80	13,805.22	10,525.95
9,863.09	9,674.46	90	11,346.08	8,226.67
8,211.56	7,957.60	95	9,633.09	6,628.44
5,725.26	5,330.17	99	7,028.14	4,303.47

<< Systematic Statistics >>

Log Transforms:			
Flow (cfs)		Number of Events	
Mean	4.244	Historical Events	0
Standard Dev	0.191	High Events	0
Station Skew	-0.472	Low Outliers	0
Regional Skew	0.0	Zero Events	0
Weighted Skew	-0.305	Missing Events	0
Adopted Skew	-0.305	Systematic Events	40

Sum of the Forks – 1962–2013

<< Low Outlier Test >>

Based on 50 events, 10 percent outlier test deviate K(N) = 2.768
Computed low outlier test value = 5,218.0473
0 low outlier(s) identified below test value of 5,218.0473

<< High Outlier Test >>

Based on 50 events, 10 percent outlier test deviate K(N) = 2.768
Computed high outlier test value = 62,451.8756
0 high outlier(s) identified above test value of 62,451.8756

<< Plotting Positions >>

Day	Month	Year	Maximum Mean Daily Flow (cfs)	Rank	Water Year	Maximum Mean Daily Flow (cfs)	Median Plotting Position
2	Jan	1962	15,160	1	2009	44,210	1.39
20	Nov	1962	17,350	2	1991	39,570	3.37
1	Jan	1964	9,870	3	2007	36,730	5.36
1	Dec	1964	18,760	4	1996	36,140	7.34
6	May	1966	9,540	5	1976	32,120	9.33
13	Dec	1966	15,360	6	1978	31,130	11.31
25	Dec	1967	22,570	7	2005	29,730	13.29
5	Jan	1969	23,650	8	1987	27,200	15.28
23	Jan	1970	8,250	9	2011	27,140	17.26
19	Jan	1971	14,710	10	1982	25,930	19.25
28	Feb	1972	20,370	11	1999	25,380	21.23
26	Dec	1972	17,190	12	1989	25,380	23.21
15	Jan	1974	20,950	13	1981	24,430	25.2
18	Jan	1975	24,180	14	1975	24,180	27.18
2	Dec	1975	32,120	15	1969	23,650	29.17
18	Jan	1977	13,430	16	1995	23,600	31.15
2	Dec	1977	31,130	17	1968	22,570	33.13
6	Mar	1979	11,690	18	1997	22,320	35.12
15	Dec	1979	19,400	19	1974	20,950	37.1
26	Dec	1980	24,430	20	1984	20,750	39.09
14	Feb	1982	25,930	21	1983	20,620	41.07
3	Dec	1982	20,620	22	1972	20,370	43.06
4	Jan	1984	20,750	23	2004	19,840	45.04
7	Jun	1985	13,660	24	1980	19,400	47.02
24	Feb	1986	13,660	25	2000	19,230	49.01
23	Nov	1986	27,200	26	2003	19,150	50.99
6	Apr	1988	10,580	27	1965	18,760	52.98
16	Oct	1988	25,380	28	2012	17,370	54.96
24	Nov	1990	39,570	29	1963	17,350	56.94
25	Jan	1993	6,890	30	1973	17,190	58.93

Day	Month	Year	Maximum Mean Daily Flow (cfs)	Rank	Water Year	Maximum Mean Daily Flow (cfs)	Median Plotting Position
3	Mar	1994	6,740	31	2002	16,680	60.91
19	Feb	1995	23,600	32	1967	15,360	62.9
29	Nov	1995	36,140	33	1962	15,160	64.88
19	Mar	1997	22,320	34	2008	15,070	66.87
30	Oct	1997	13,660	35	1971	14,710	68.85
29	Dec	1998	25,380	36	2006	13,850	70.83
15	Dec	1999	19,230	37	1998	13,660	72.82
1	Oct	2000	8,270	38	1986	13,660	74.8
14	Nov	2001	16,680	39	1985	13,660	76.79
31	Jan	2003	19,150	40	1977	13,430	78.77
18	Nov	2003	19,840	41	1979	11,690	80.75
18	Jan	2005	29,730	42	2013	11,210	82.74
10	Jan	2006	13,850	43	2010	11,166	84.72
6	Nov	2006	36,730	44	1988	10,580	86.71
4	Dec	2007	15,070	45	1964	9,870	88.69
7	Jan	2009	44,210	46	1966	9,540	90.67
26	Oct	2009	11,166	47	2001	8,270	92.66
16	Jan	2011	27,140	48	1970	8,250	94.64
22	Feb	2012	17,370	49	1993	6,890	96.63
30	Sep	2013	11,210	50	1994	6,740	98.61

<< Skew Weighting >>

Based on 51 events, mean-square error of station skew = 0.121

Mean-square error of regional skew = 0.302

<< Frequency Curve >>

Computed Curve	Expected Probability	Percent Chance Exceedance	Confidence Limits	
			0.05	0.095
FLOW (cfs)			FLOW (cfs)	
59,277.64	63,154.71	0.2	77,094.89	48,761.47
52,942.96	55,595.73	0.5	67,482.59	44,152.80
48,142.05	50,058.95	1	60,352.24	40,601.78
43,311.40	44,626.59	2	53,326.33	36,969.59
36,822.09	37,543.45	5	44,152.83	31,976.97
31,757.05	32,153.31	10	37,238.49	27,965.19
26,419.92	26,595.59	20	30,238.65	23,588.08
18,306.26	18,306.26	50	20,357.31	16,474.93
12,435.67	12,341.89	80	13,921.75	10,875.62
10,078.46	9,929.44	90	11,463.12	8,572.59
8,437.01	8,236.92	95	9,763.05	6,982.87
5,982.12	5,672.71	99	7,197.71	4,669.59

<< Systematic Statistics >>

Log Transforms:			
Flow (cfs)		Number of Events	
Mean	4.257	Historical Events	0
Standard Dev	0.195	High Events	0
Station Skew	-0.263	Low Outliers	0
Regional Skew	0.0	Zero Events	0
Weighted Skew	-0.188	Missing Events	0
Adopted Skew	-0.188	Systematic Events	50

APPENDIX D

Calibration High Water Marks

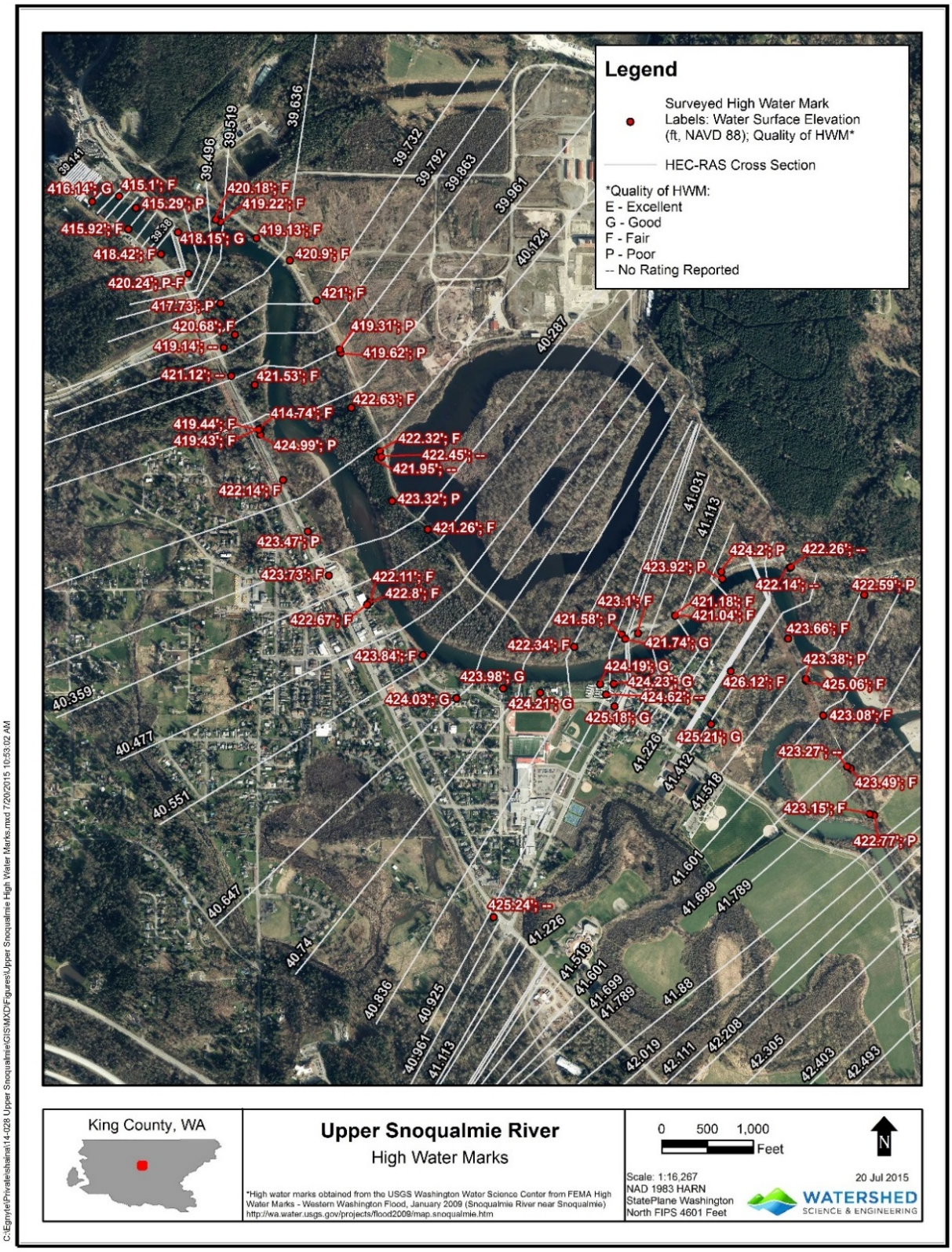


Figure D-1. January 2009 Flood High Water Marks, Collected by the USGS.

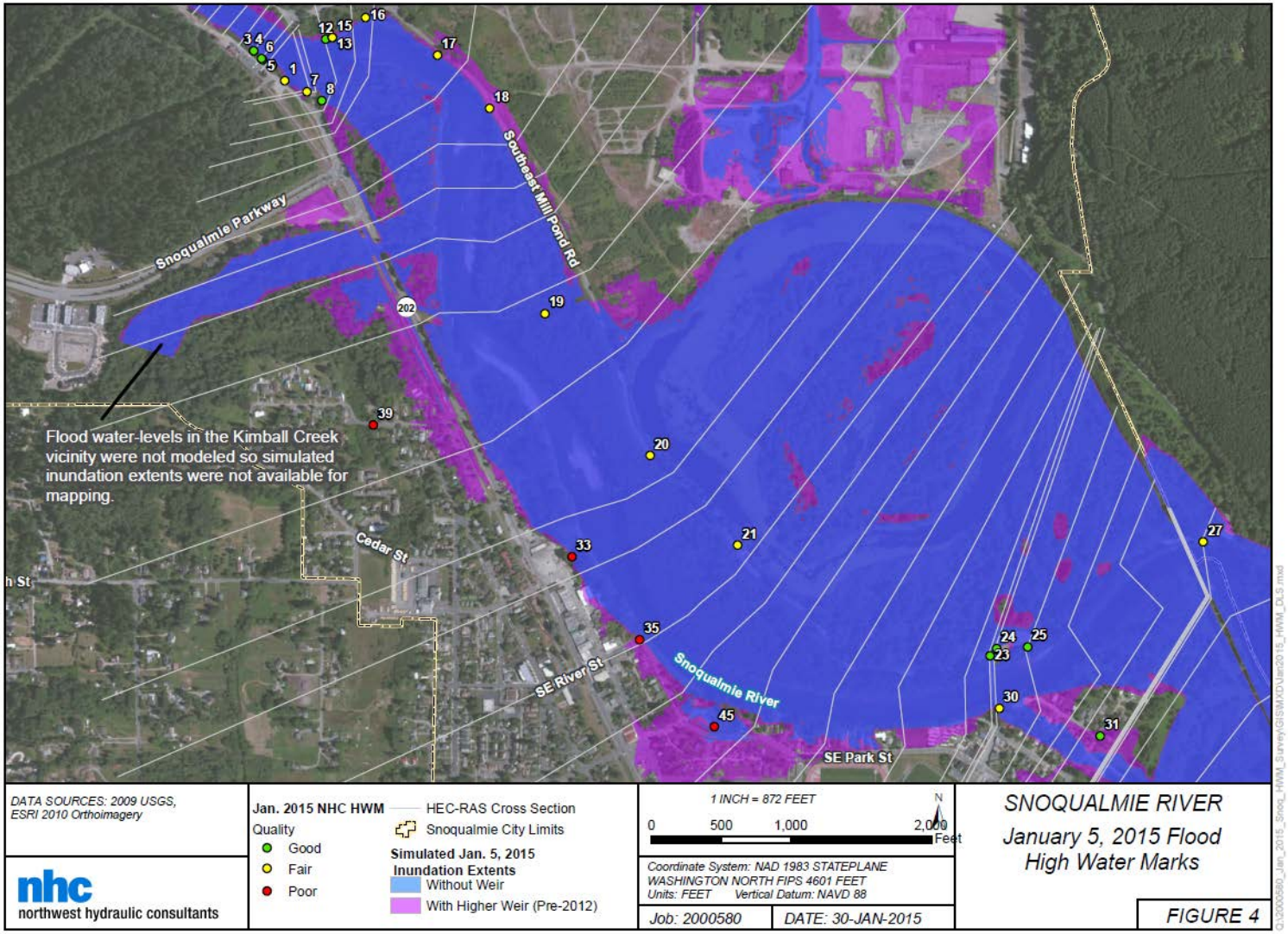
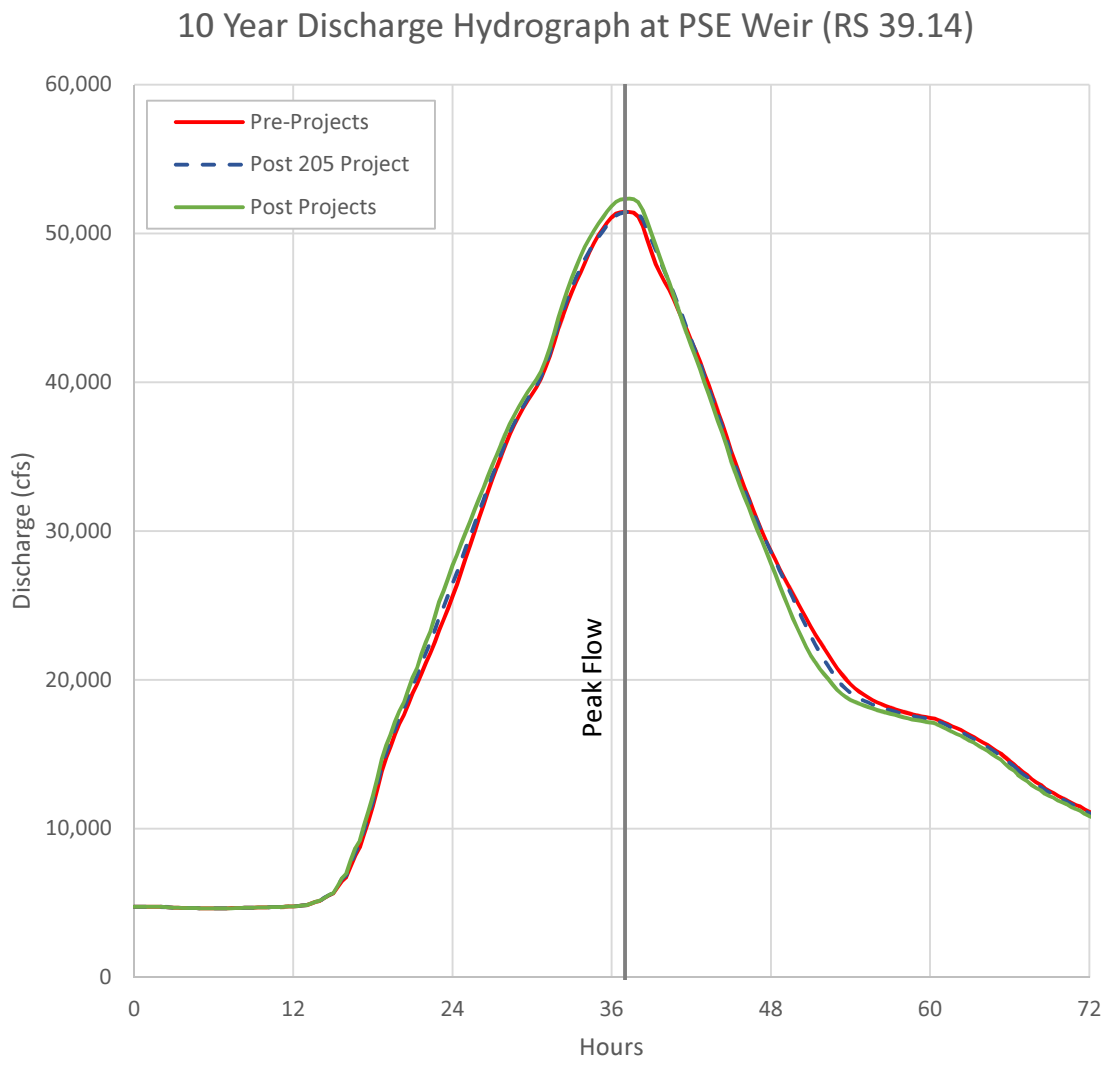
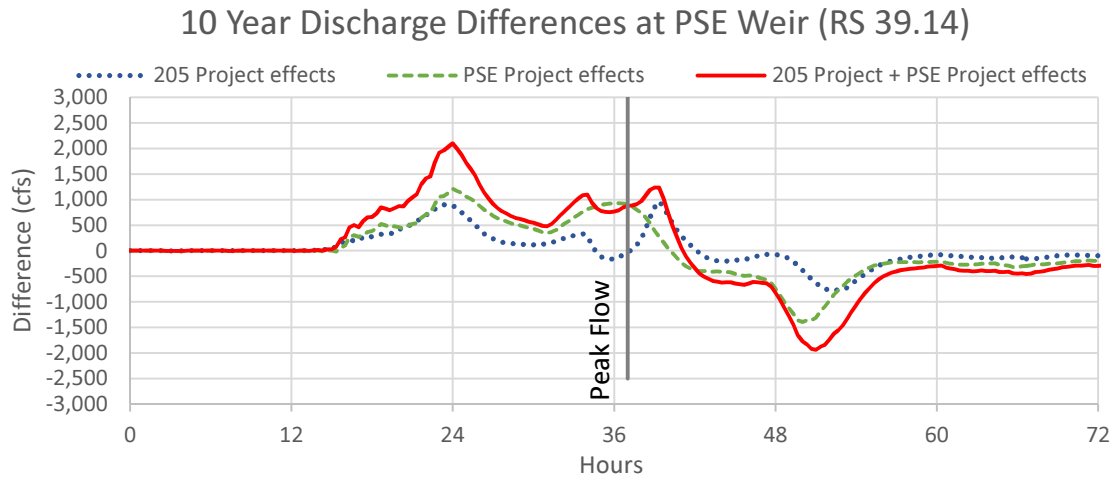


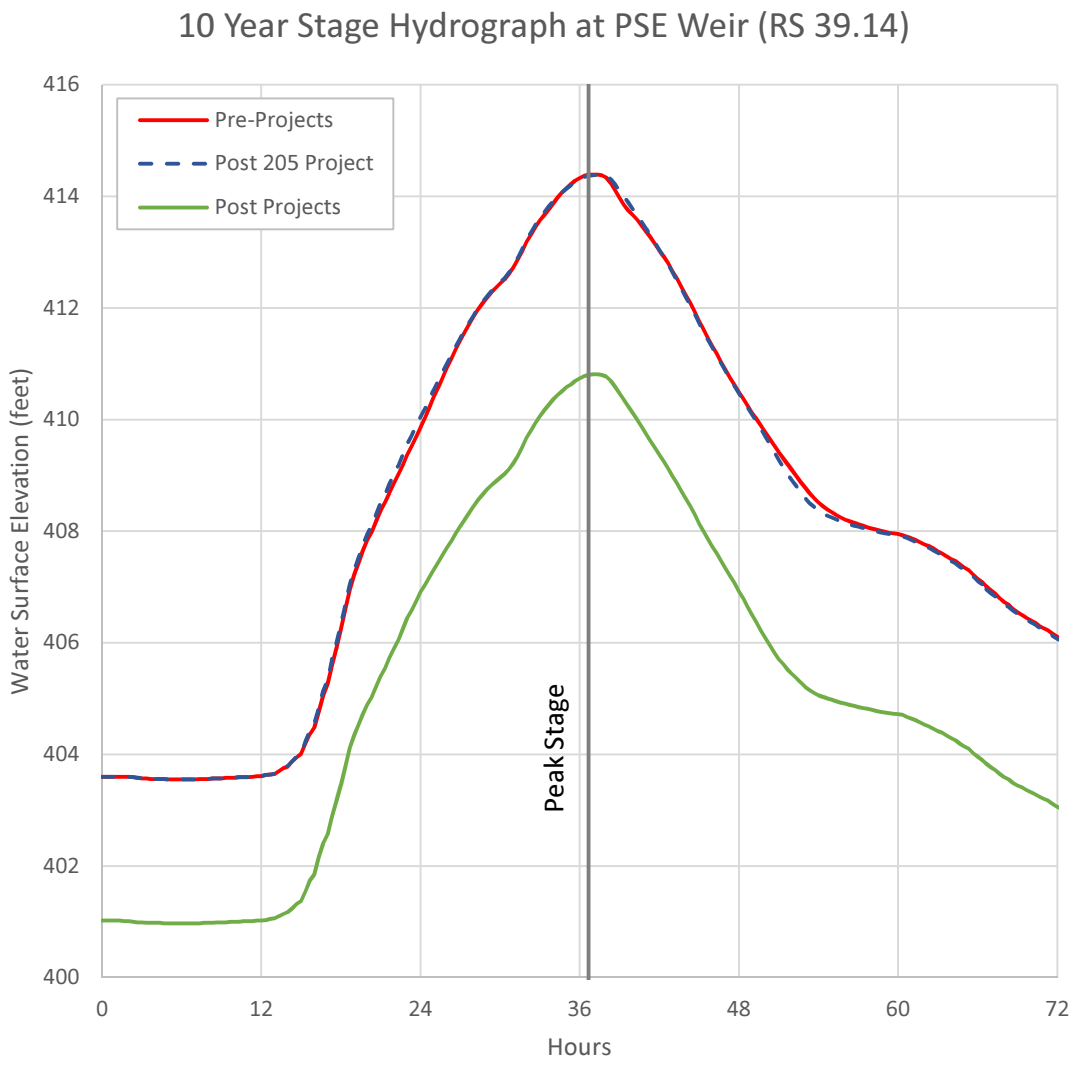
Figure D-2. Figure copied from February 6, 2015, e-mail correspondence from NHC to Nancy Davidson at City of Snoqualmie Department of Public Works with subject "Task Order 32 – January 2015 Flood High Water Mark Identification." Full document was provided to WSE with elevations for each numbered point.

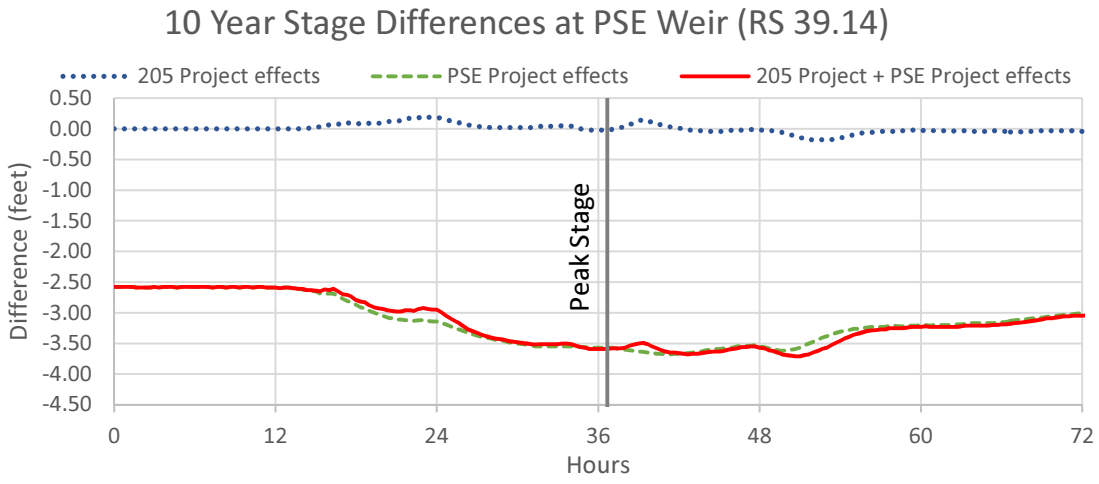
APPENDIX E

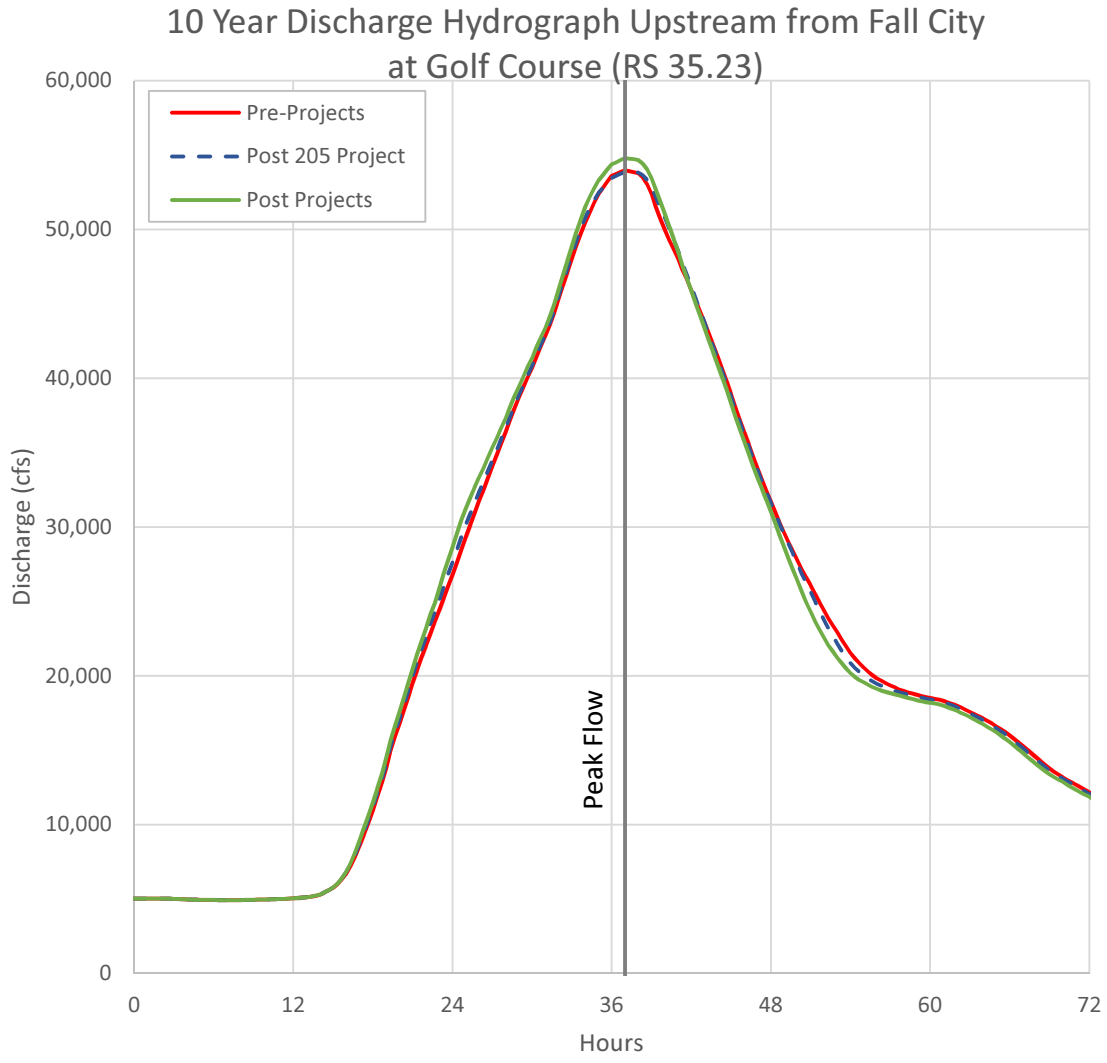
Model Results Graphics – Stage and Discharge Comparison Plots

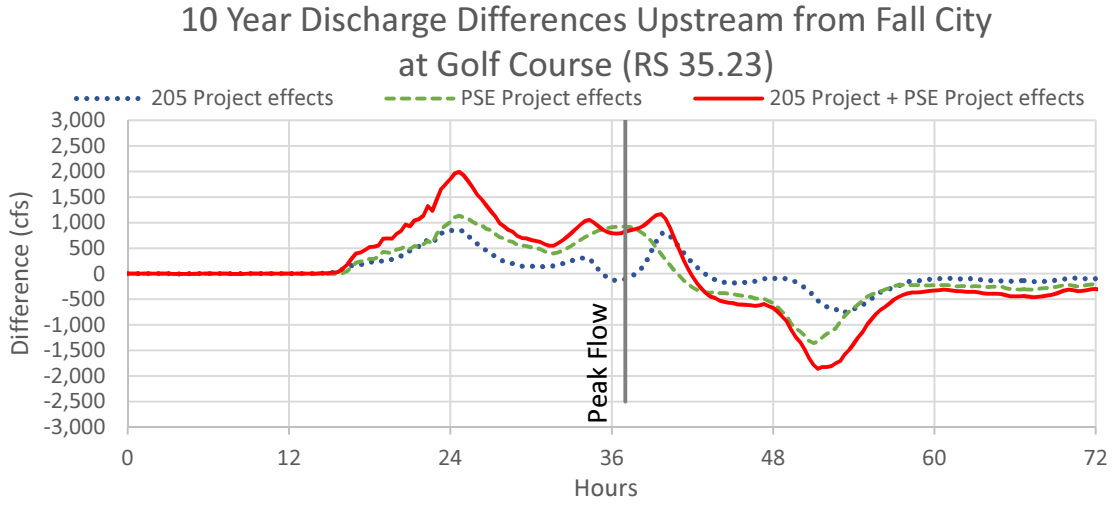

10 Year Discharge Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Peak Flow
Discharge (cfs)
60,000
50,000
40,000
30,000
20,000
10,000
0
0
12
24
36
48
60
72
Hours

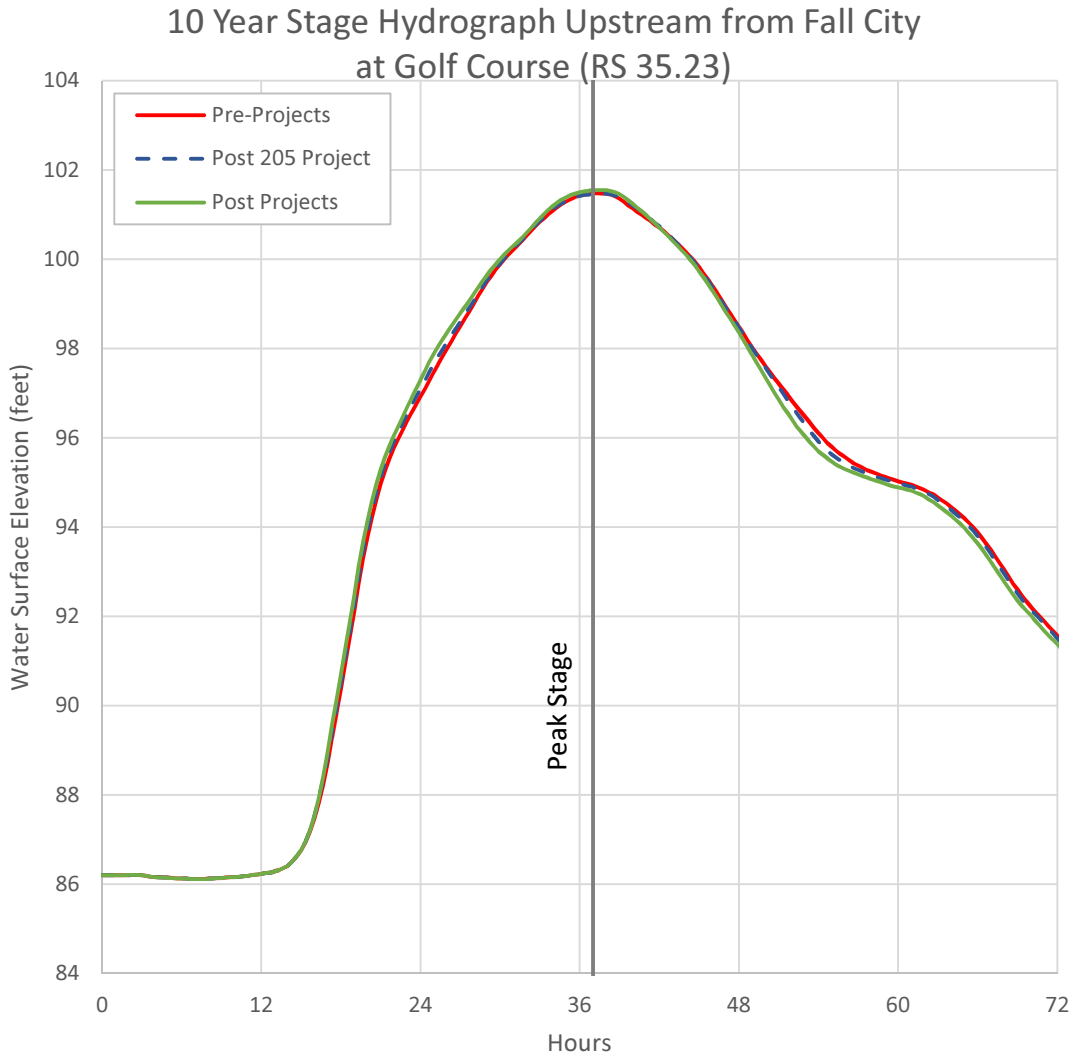

10 Year Discharge Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Flow
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
0
12
24
36
48
60
72
Hours

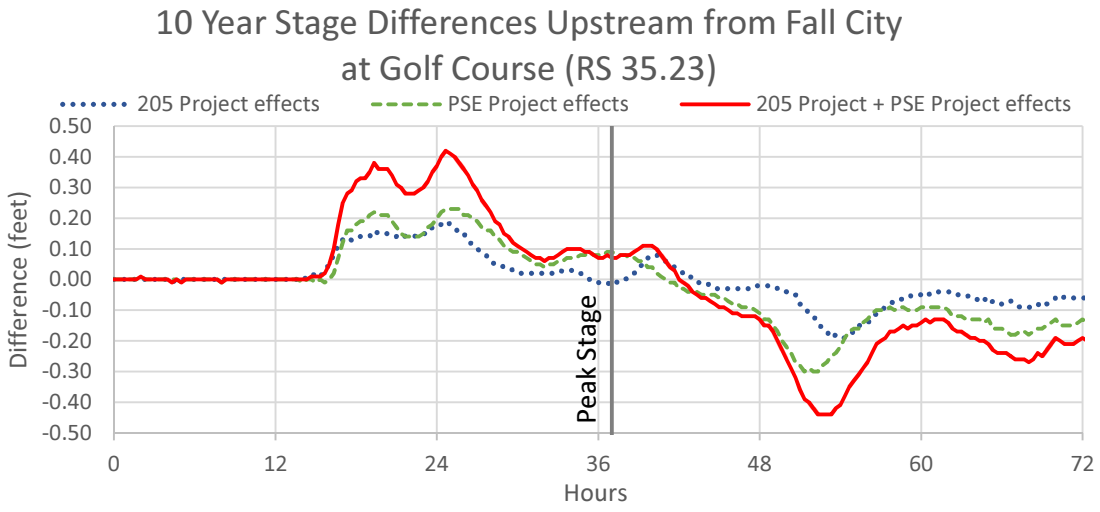

10 Year Stage Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
416
414
412
410
408
406
404
402
400
0
12
24
36
48
60
72
Hours
Peak Stage


10 Year Stage Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
-3.50
-4.00
-4.50
0
12
24
36
48
60
72
Hours
Peak Stage


10 Year Discharge Hydrograph Upstream from Fall City at Golf Course (RS 35.23)
Pre-Projects
Post 205 Project
Post Projects
Discharge (cfs)
60,000
50,000
40,000
30,000
20,000
10,000
0
Peak Flow
0
12
24
36
48
60
72
Hours


10 Year Discharge Differences Upstream from Fall City at Golf Course (RS 35.23)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
Peak Flow
0
12
24
36
48
60
72
Hours


10 Year Stage Hydrograph Upstream from Fall City
at Golf Course (RS 35.23)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
104
102
100
98
96
94
92
90
88
86
84
0
12
24
36
48
60
72
Hours
Peak Stage


10 Year Stage Differences Upstream from Fall City
at Golf Course (RS 35.23)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
12
24
36
48
60
72
Hours
Peak Stage

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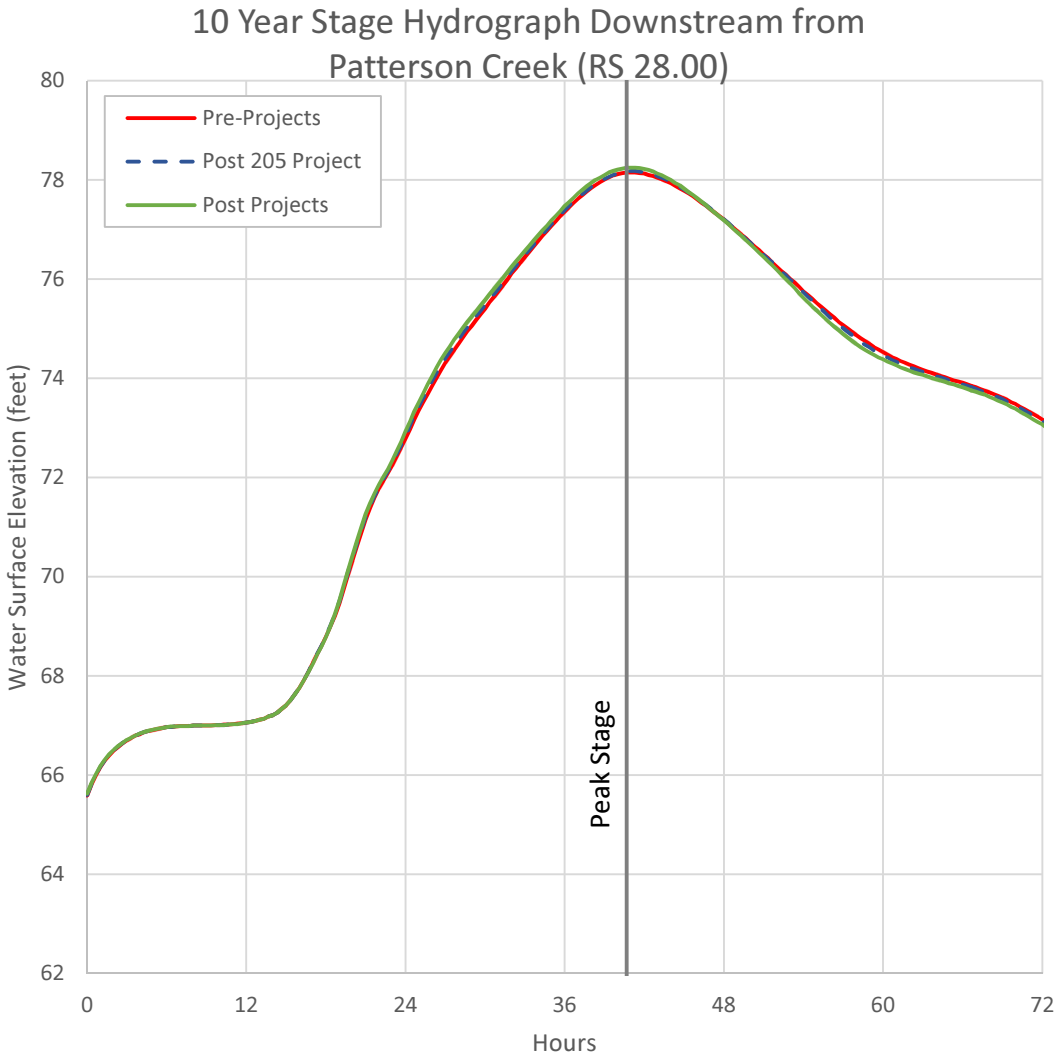
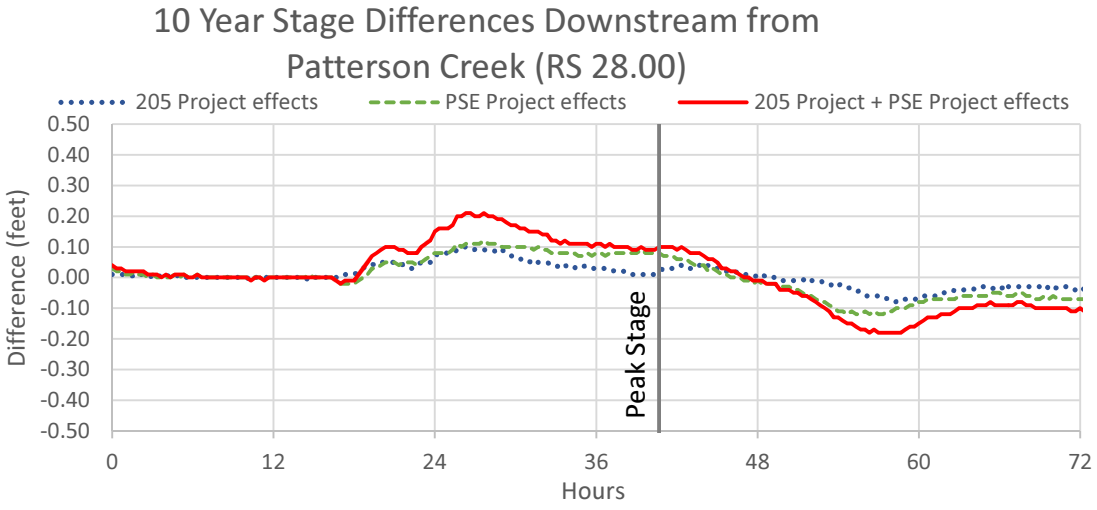
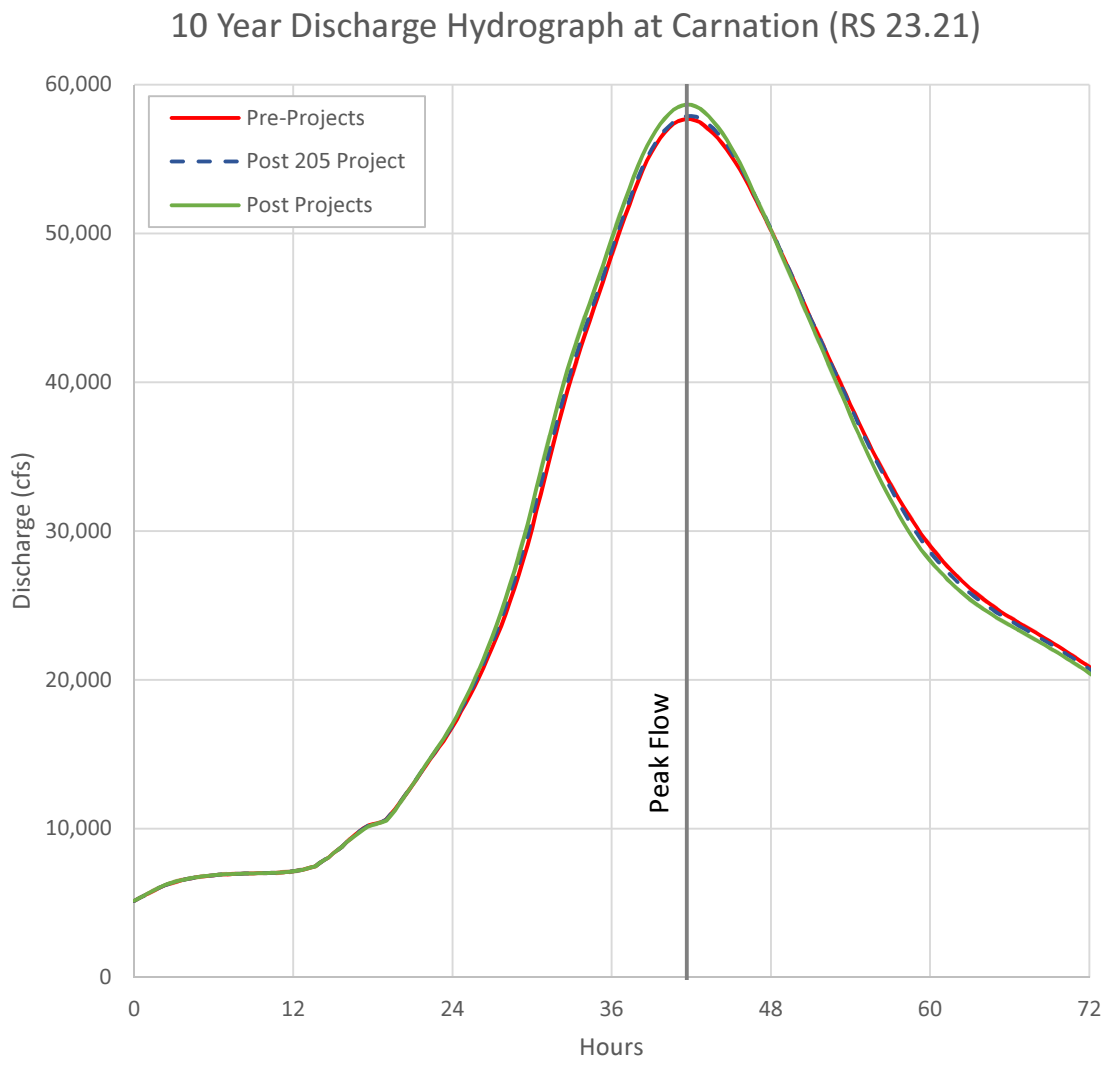
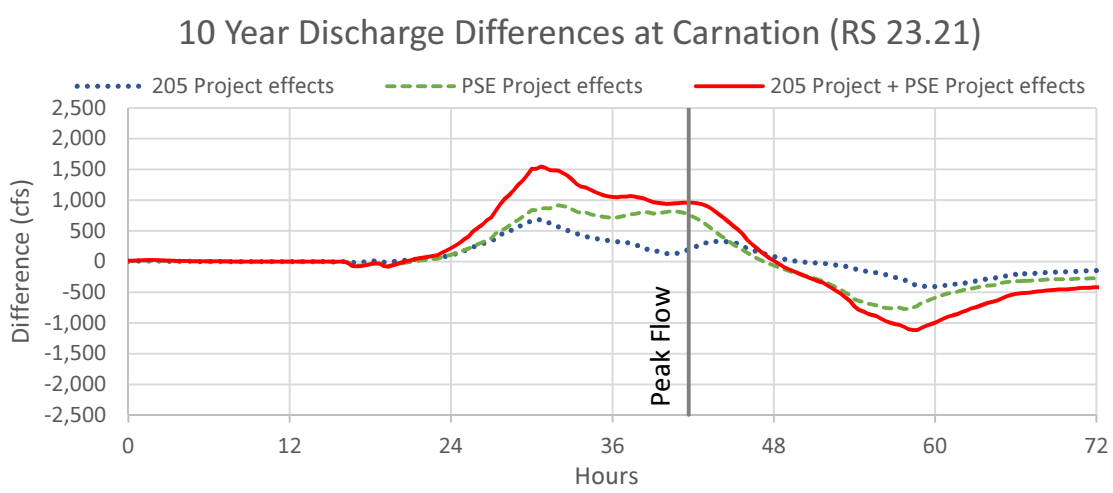
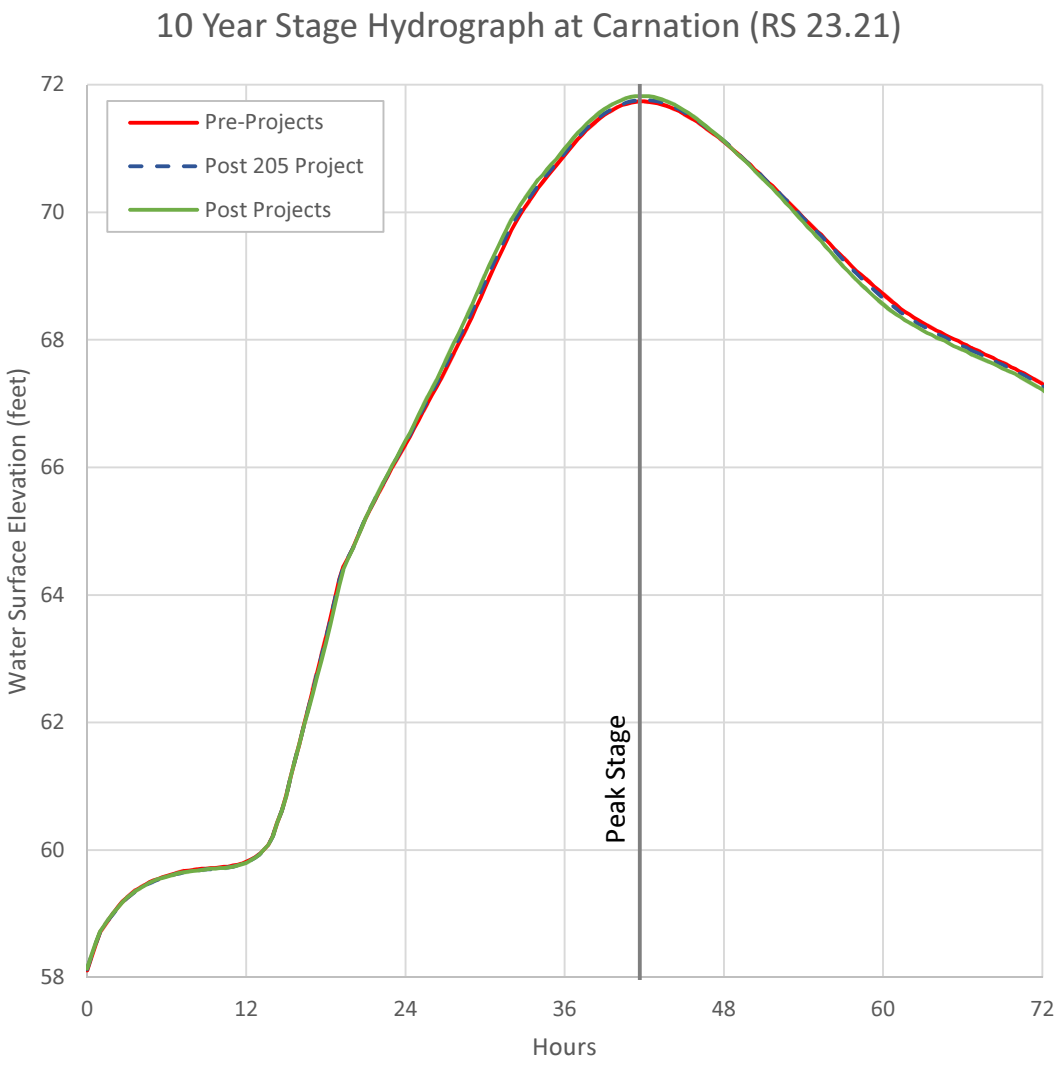
Only stage results are reported for Patterson Creek because there are multiple parallel flow paths in the HEC-RAS model at this location and determining the total flow passing this point is difficult

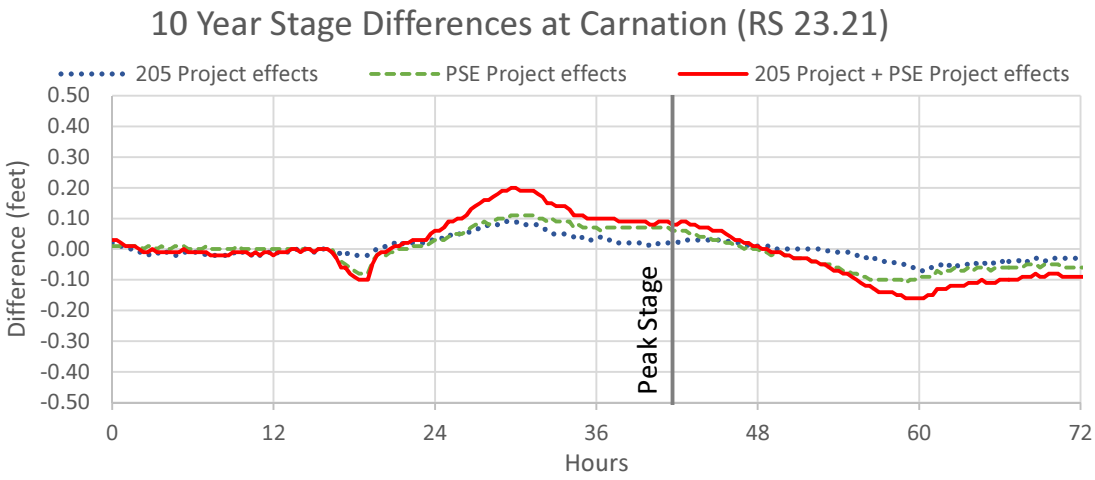

10 Year Stage Hydrograph Downstream from Patterson Creek (RS 28.00)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
80
78
76
74
72
70
68
66
64
62
Peak Stage
0
12
24
36
48
60
72
Hours

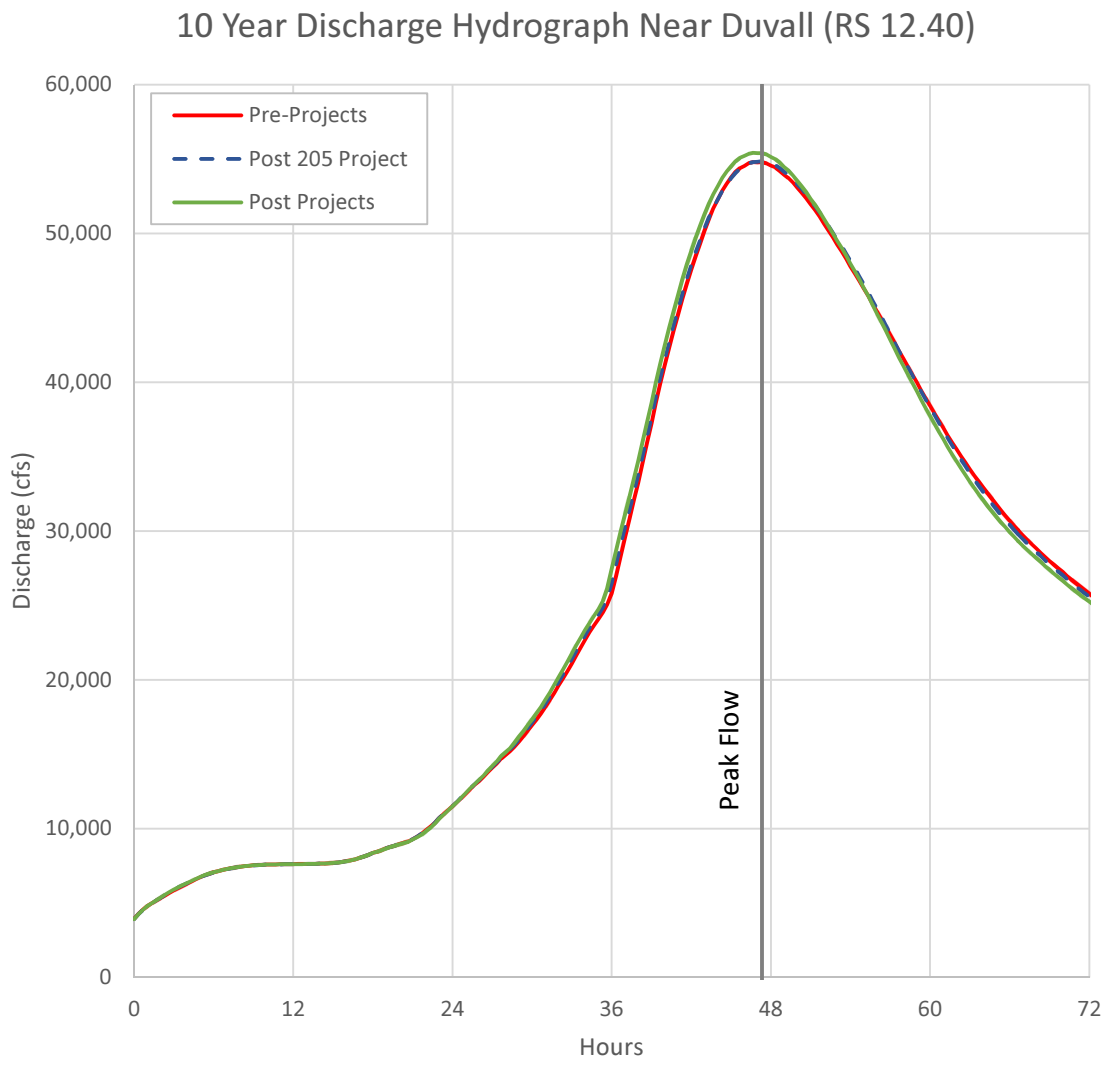

10 Year Stage Differences Downstream from Patterson Creek (RS 28.00)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
Peak Stage
0
12
24
36
48
60
72
Hours

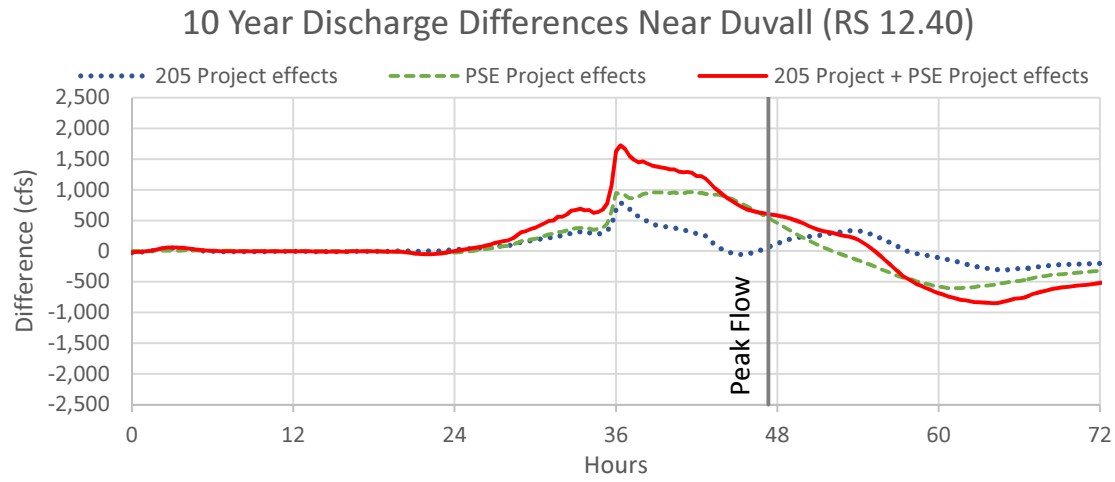

10 Year Discharge Hydrograph at Carnation (RS 23.21)
Pre-Projects
Post 205 Project
Post Projects
Discharge (cfs)
60,000
50,000
40,000
30,000
20,000
10,000
0
Peak Flow
0
12
24
36
48
60
72
Hours

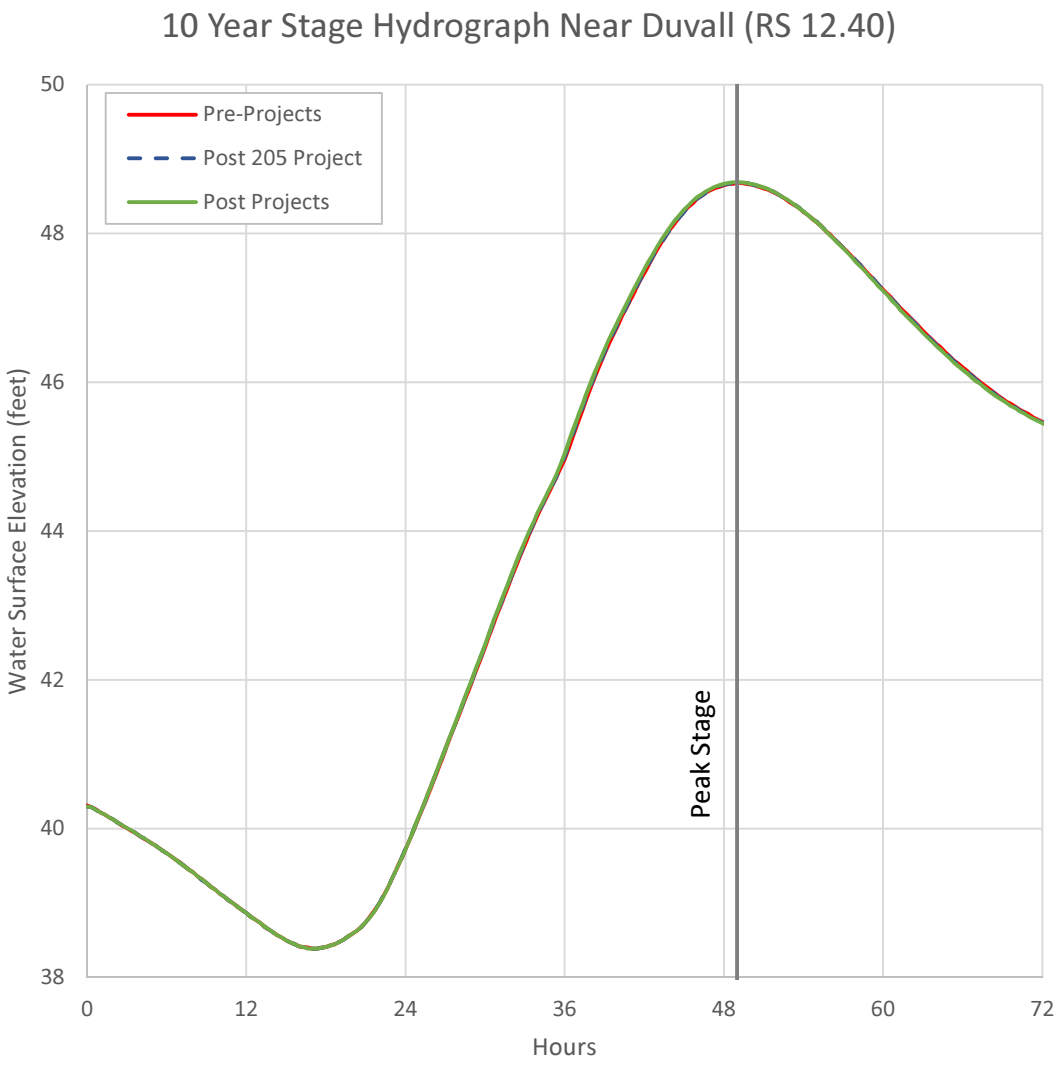

10 Year Discharge Differences at Carnation (RS 23.21)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (cfs)
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
Peak Flow
0
12
24
36
48
60
72
Hours

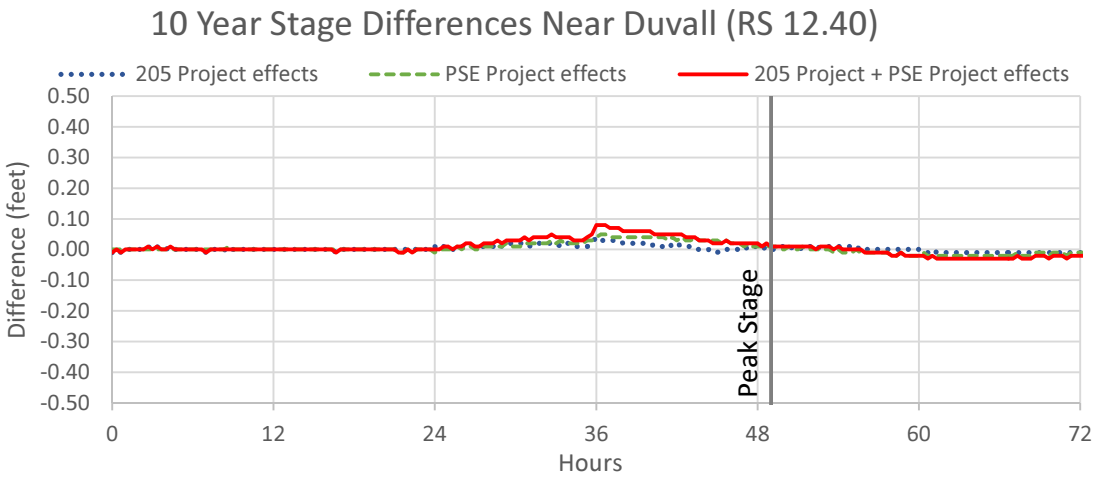

10 Year Stage Hydrograph at Carnation (RS 23.21)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
72
70
68
66
64
62
60
58
0
12
24
36
48
60
72
Hours
Peak Stage

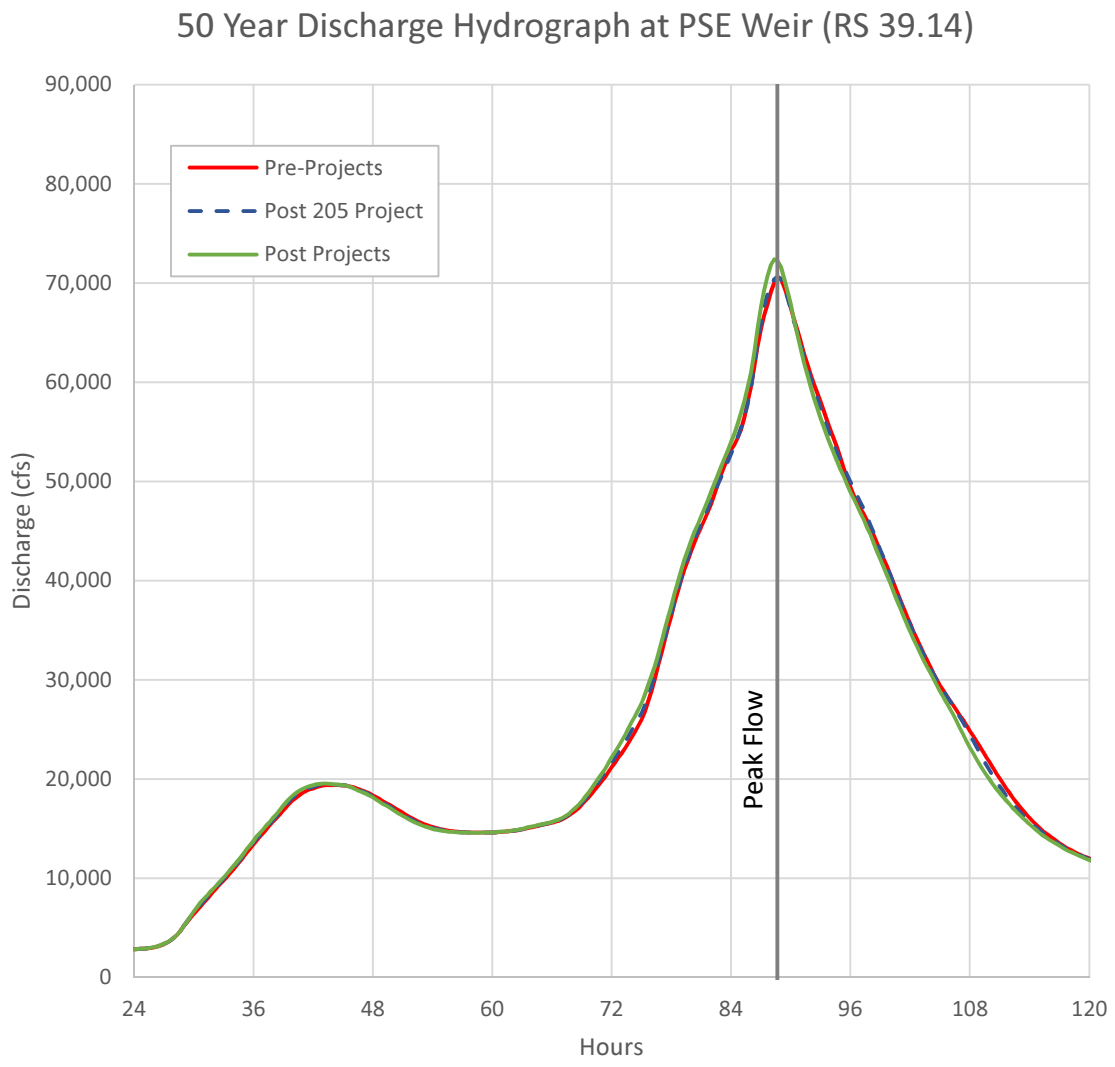

10 Year Stage Differences at Carnation (RS 23.21)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
12
24
36
48
60
72
Hours
Peak Stage

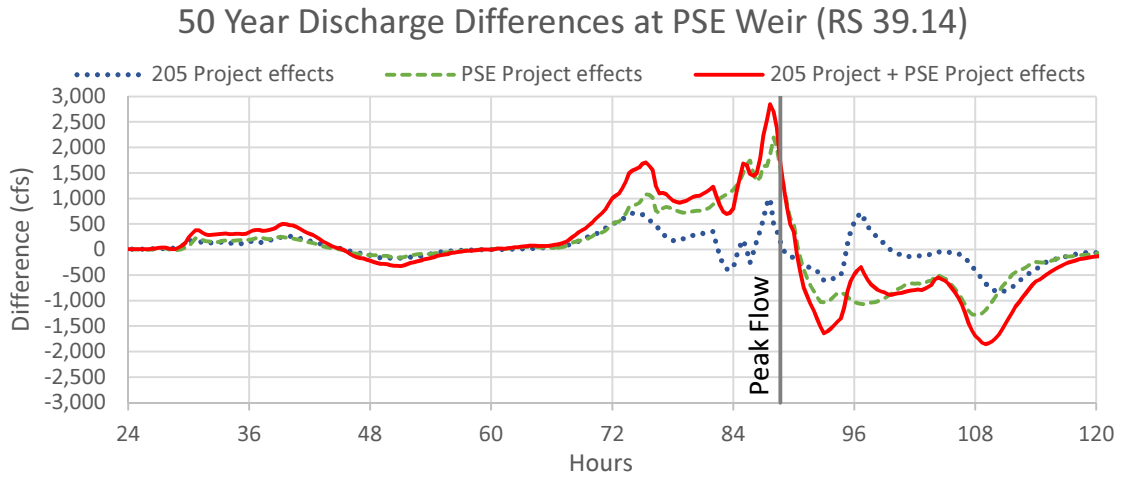

10 Year Discharge Hydrograph Near Duvall (RS 12.40)
Pre-Projects
Post 205 Project
Post Projects
Discharge (cfs)
60,000
50,000
40,000
30,000
20,000
10,000
0
Peak Flow
0
12
24
36
48
60
72
Hours

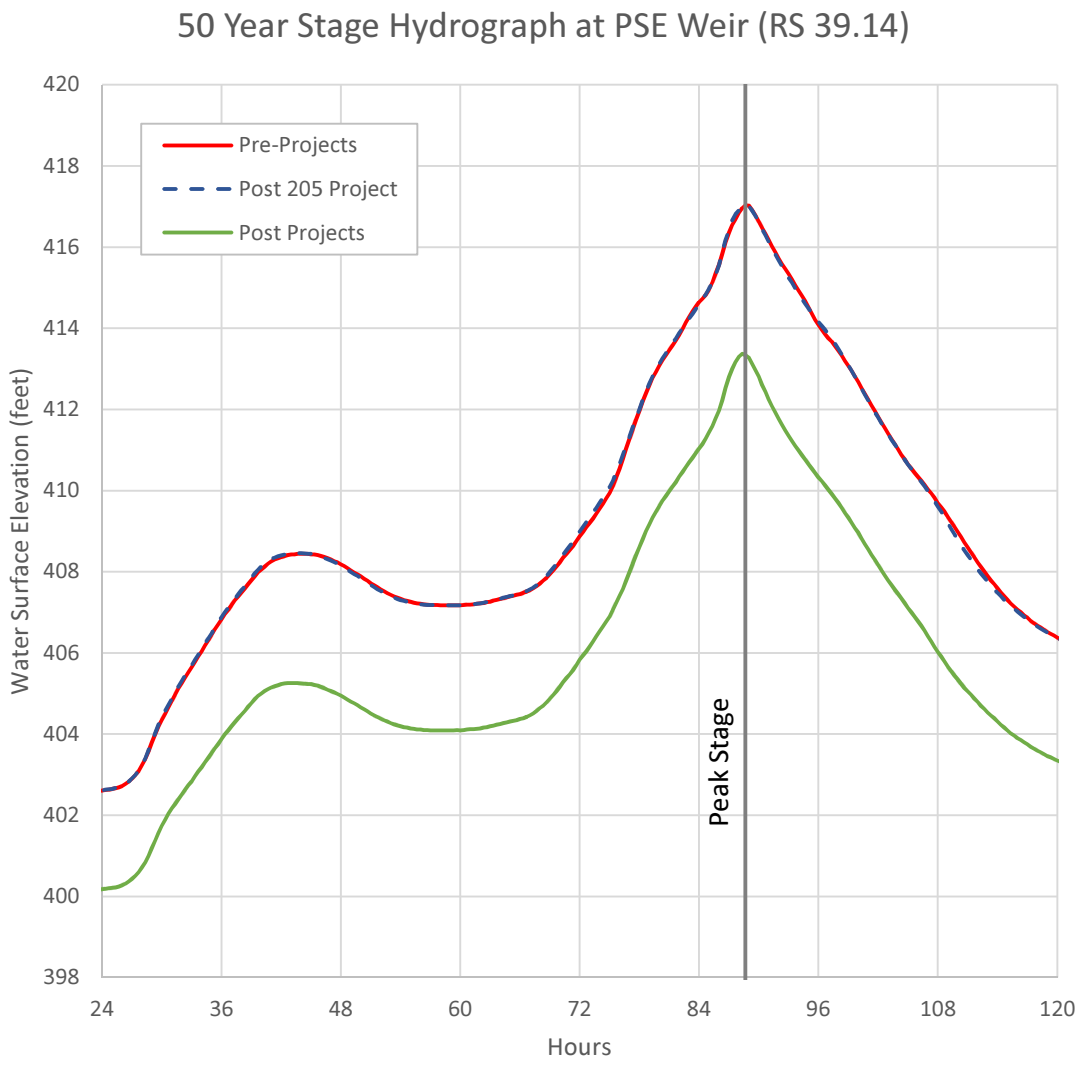

10 Year Discharge Differences Near Duvall (RS 12.40)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (cfs)
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
Peak Flow
0
12
24
36
48
60
72
Hours

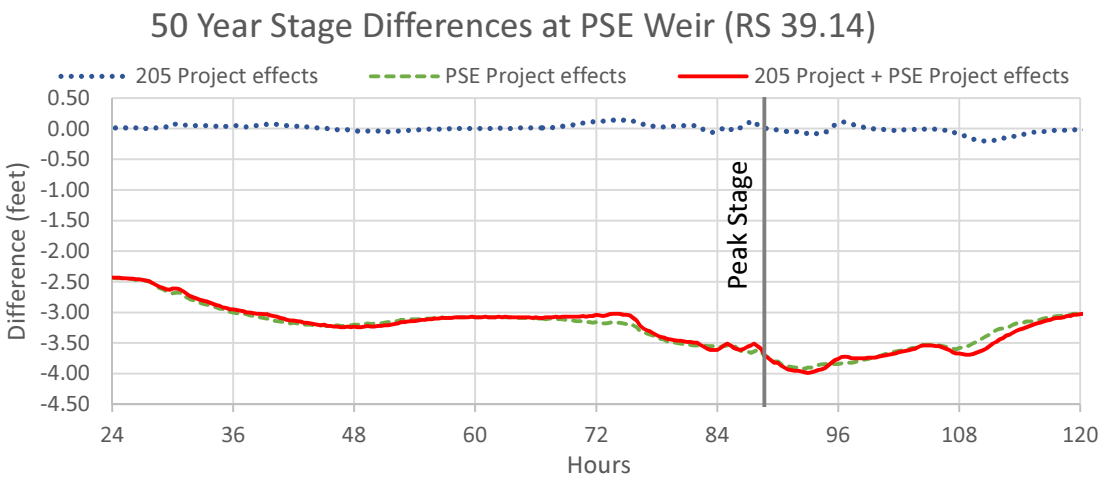

10 Year Stage Hydrograph Near Duvall (RS 12.40)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
50
48
46
44
42
40
38
0
12
24
36
48
60
72
Hours
Peak Stage

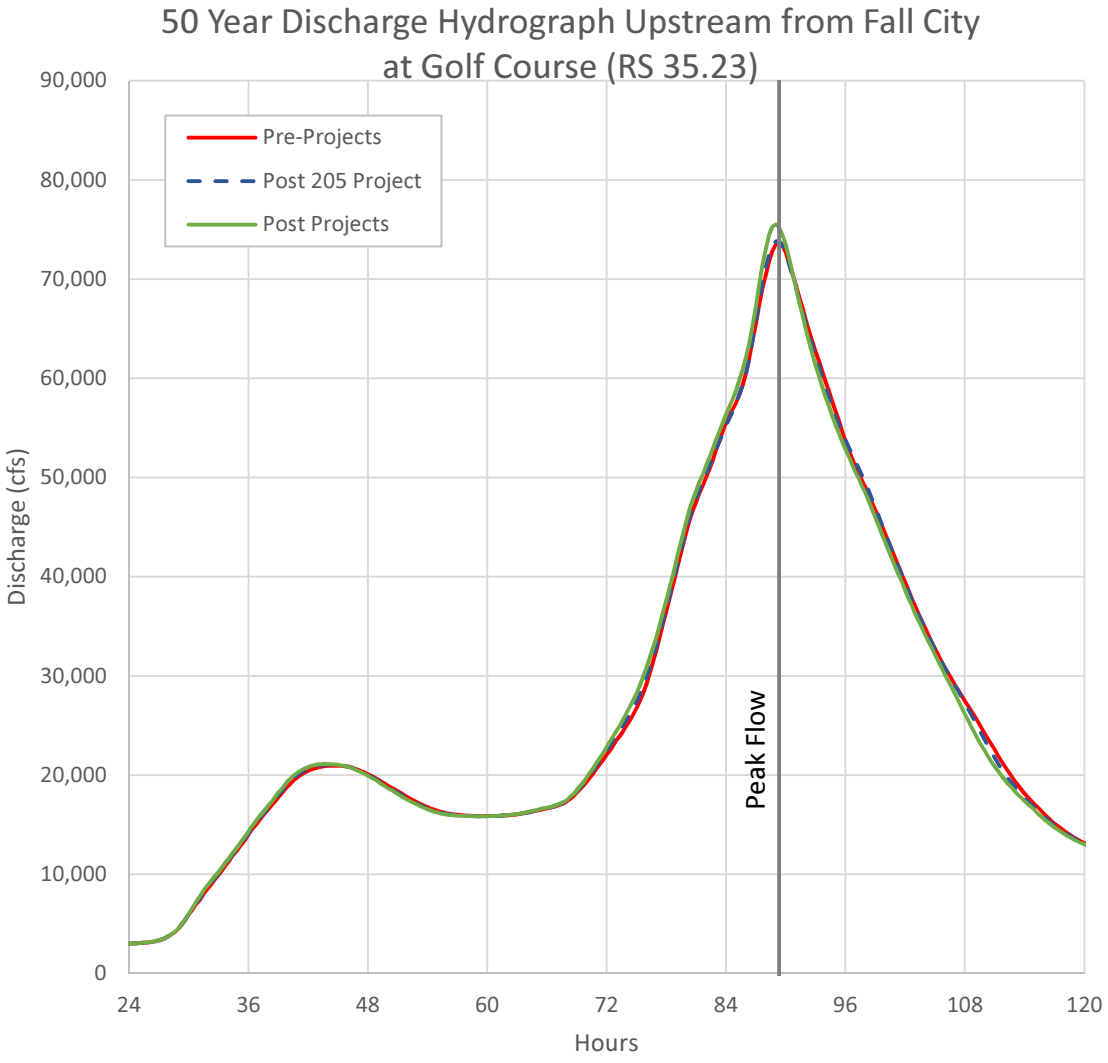

10 Year Stage Differences Near Duvall (RS 12.40)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
12
24
36
48
60
72
Hours
Peak Stage

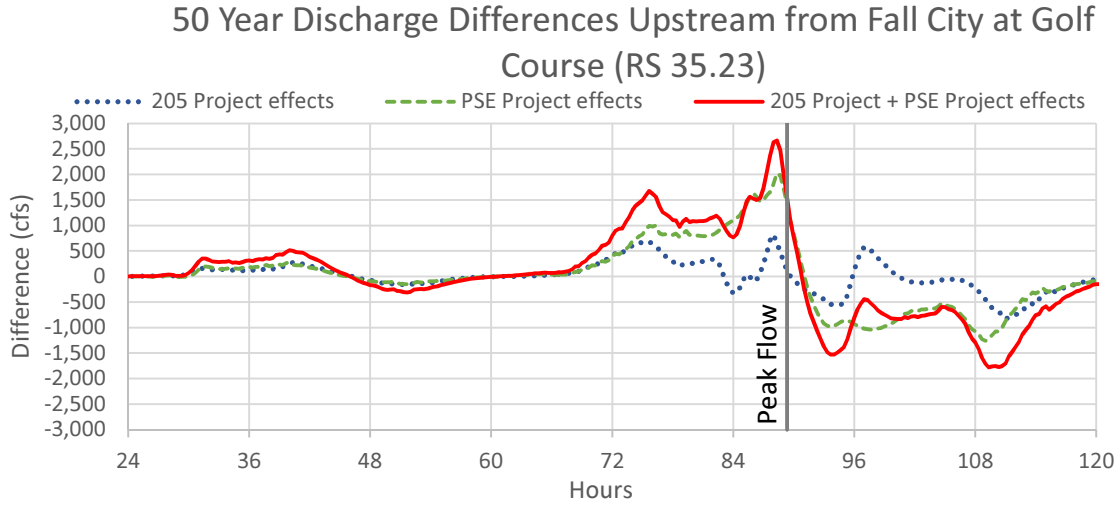

50 Year Discharge Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Peak Flow
Discharge (cfs)
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
24
36
48
60
72
84
96
108
120
Hours

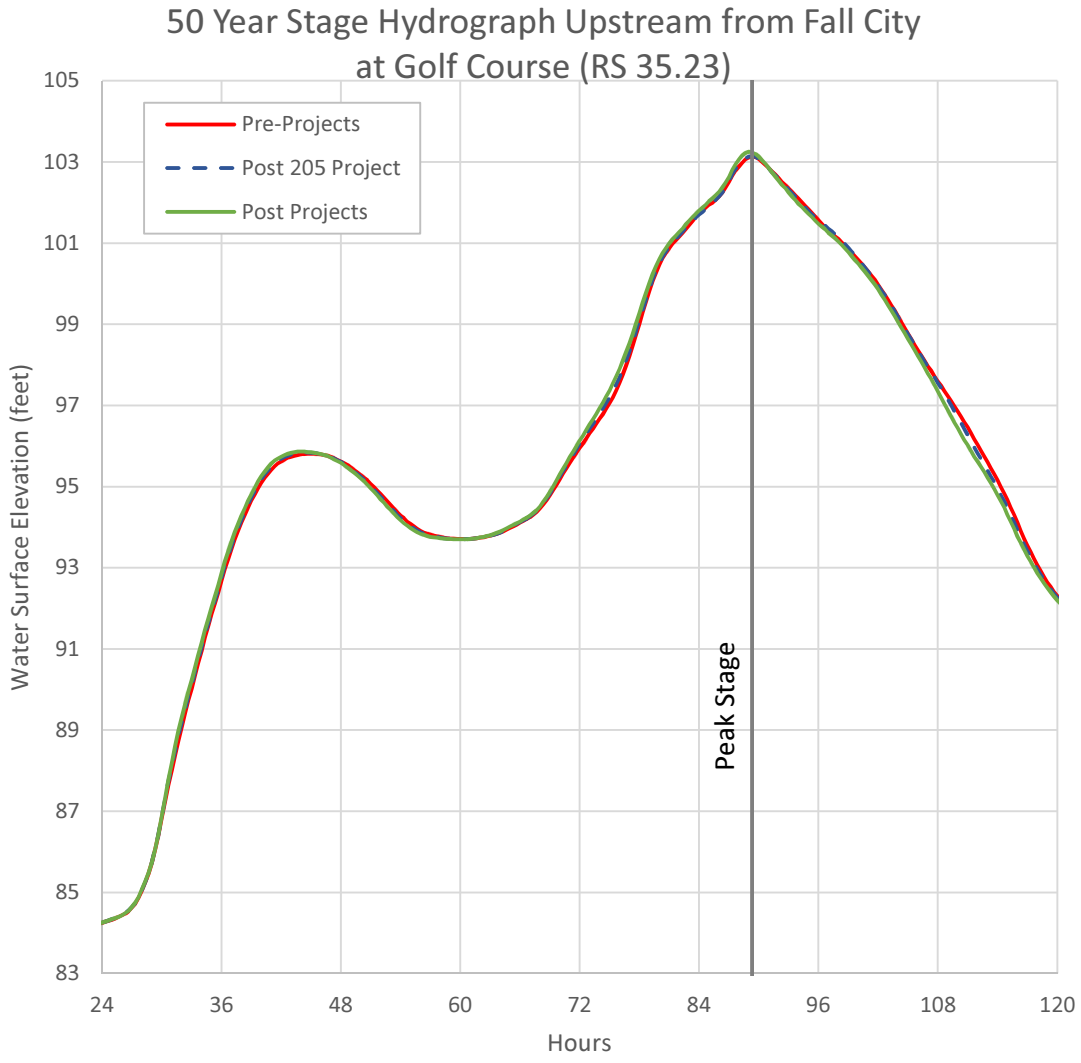

50 Year Discharge Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Flow
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
24
36
48
60
72
84
96
108
120
Hours

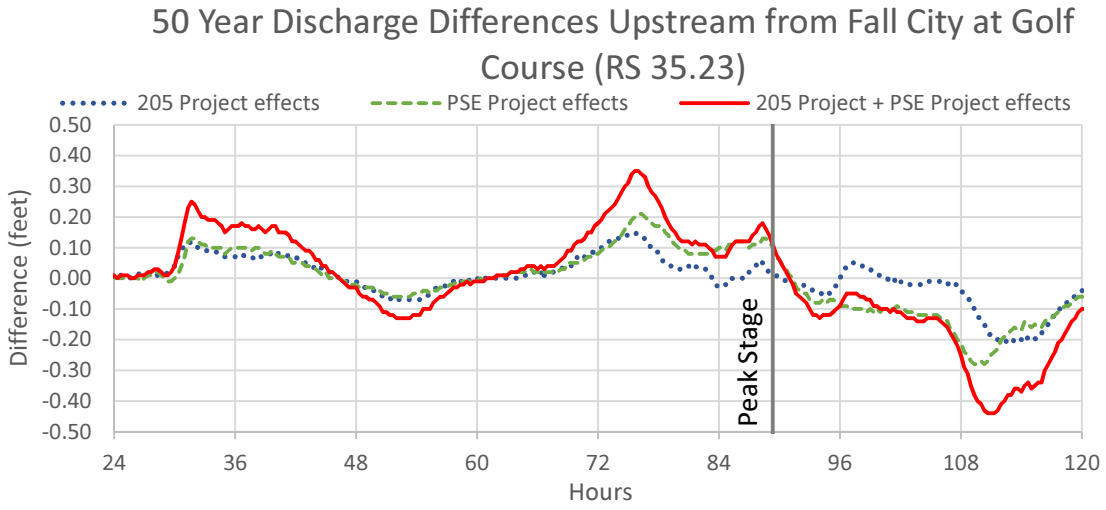

50 Year Stage Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Peak Stage
Water Surface Elevation (feet)
420
418
416
414
412
410
408
406
404
402
400
398
24
36
48
60
72
84
96
108
120
Hours


50 Year Stage Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Stage
Difference (feet)
0.50
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
-3.50
-4.00
-4.50
24
36
48
60
72
84
96
108
120
Hours


50 Year Discharge Hydrograph Upstream from Fall City at Golf Course (RS 35.23)
Pre-Projects
Post 205 Project
Post Projects
Peak Flow
Discharge (cfs)
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
24
36
48
60
72
84
96
108
120
Hours


50 Year Discharge Differences Upstream from Fall City at Golf Course (RS 35.23)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Flow
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
24
36
48
60
72
84
96
108
120
Hours


50 Year Stage Hydrograph Upstream from Fall City at Golf Course (RS 35.23)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
105
103
101
99
97
95
93
91
89
87
85
83
24
36
48
60
72
84
96
108
120
Hours
Peak Stage


50 Year Discharge Differences Upstream from Fall City at Golf Course (RS 35.23)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

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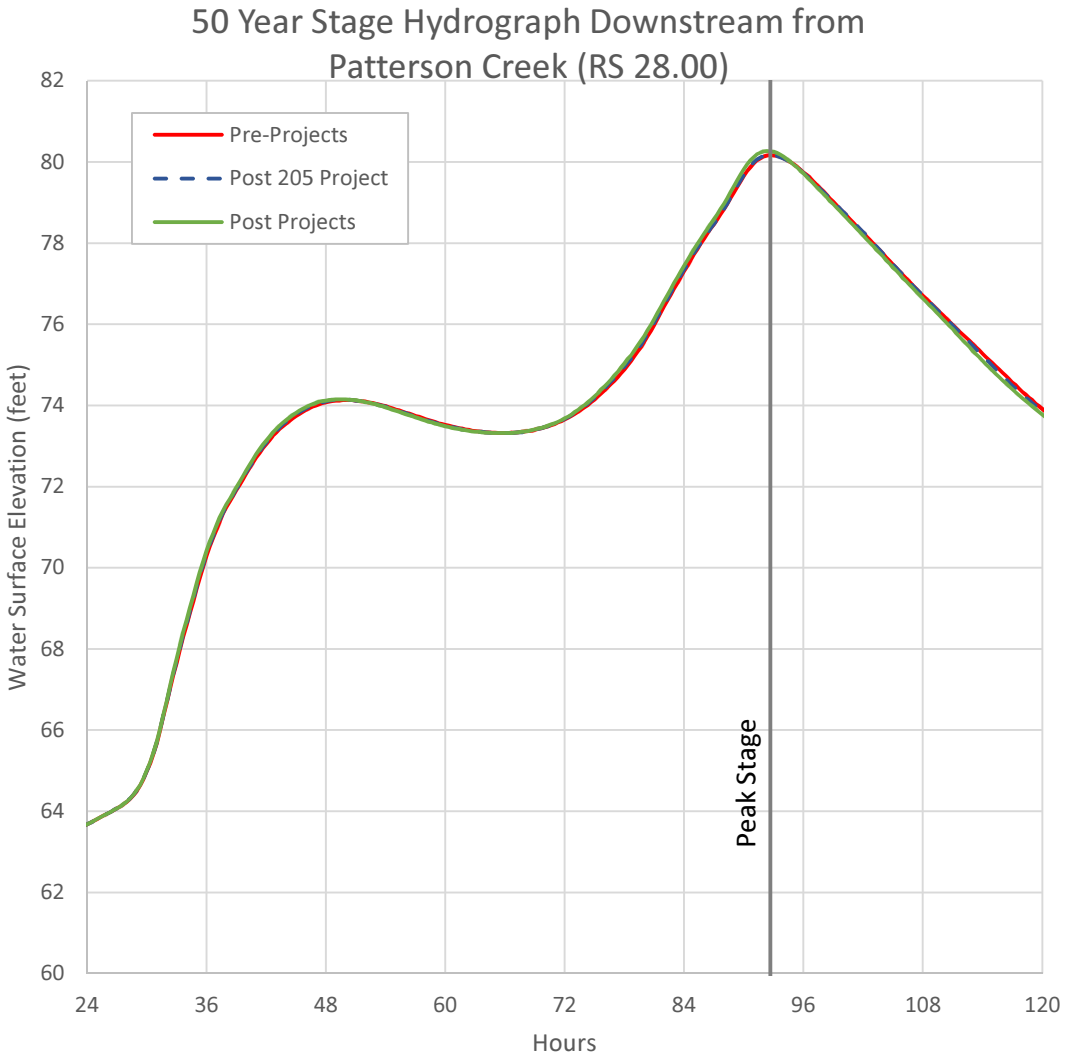
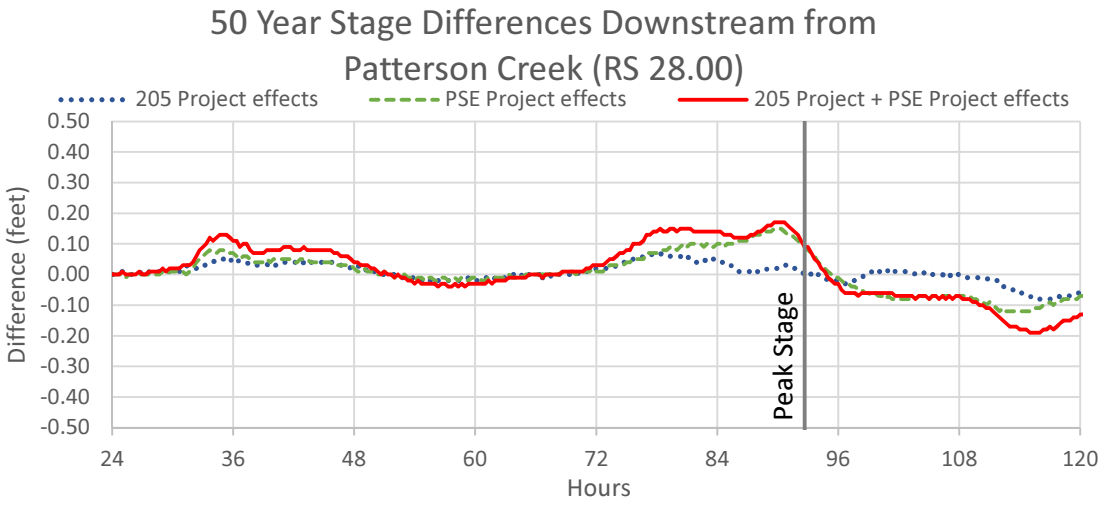
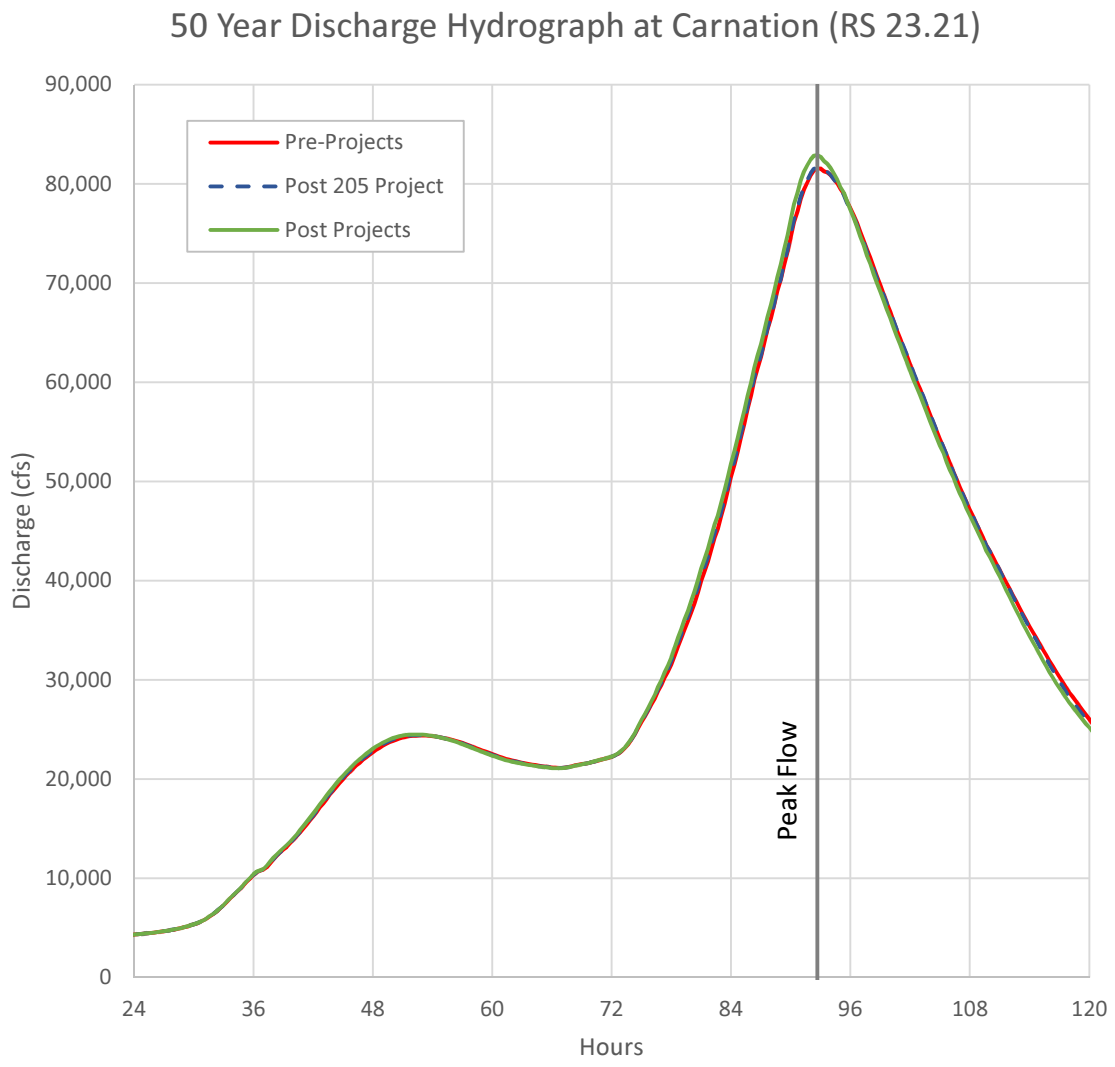
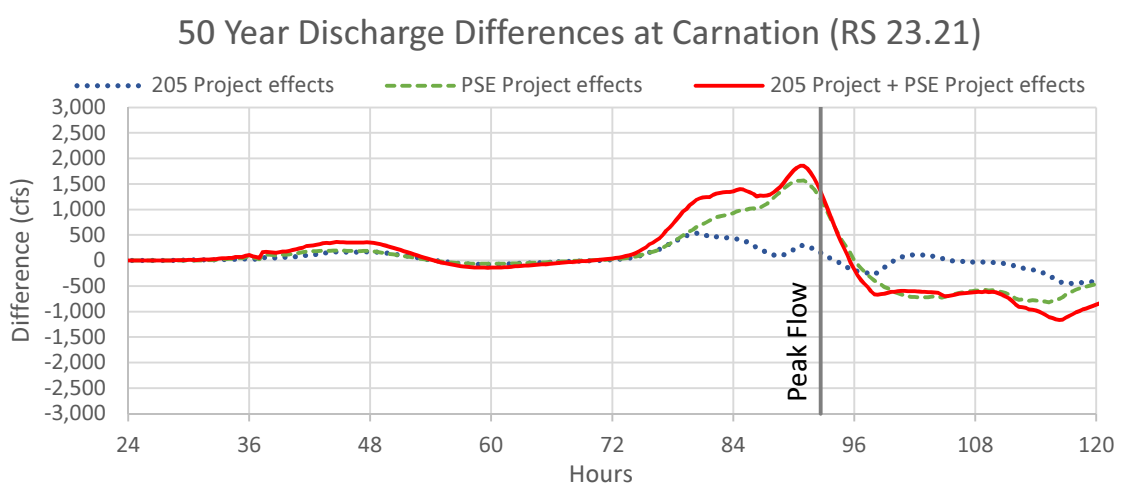
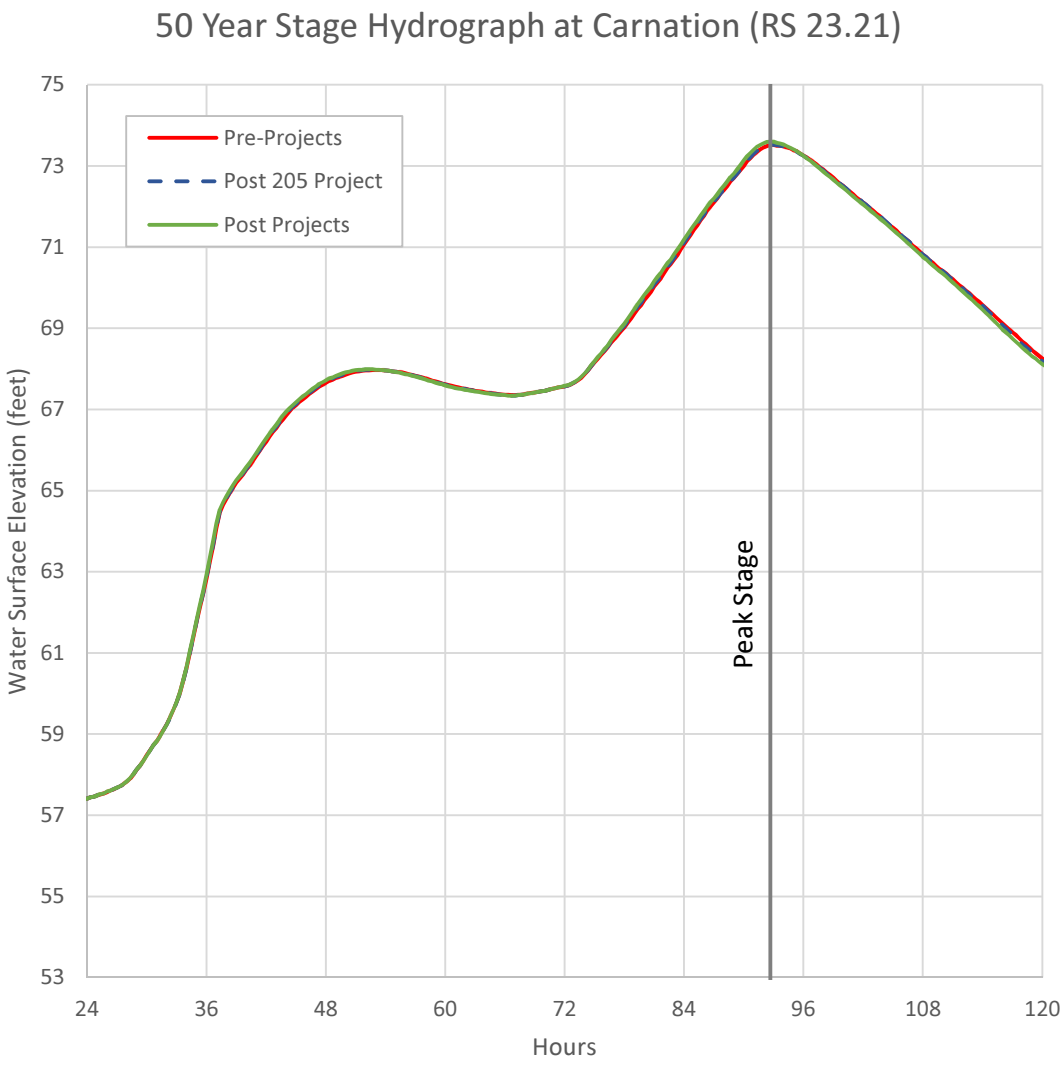
Only stage results are reported for Patterson Creek because there are multiple parallel flow paths in the HEC-RAS model at this location and determining the total flow passing this point is difficult

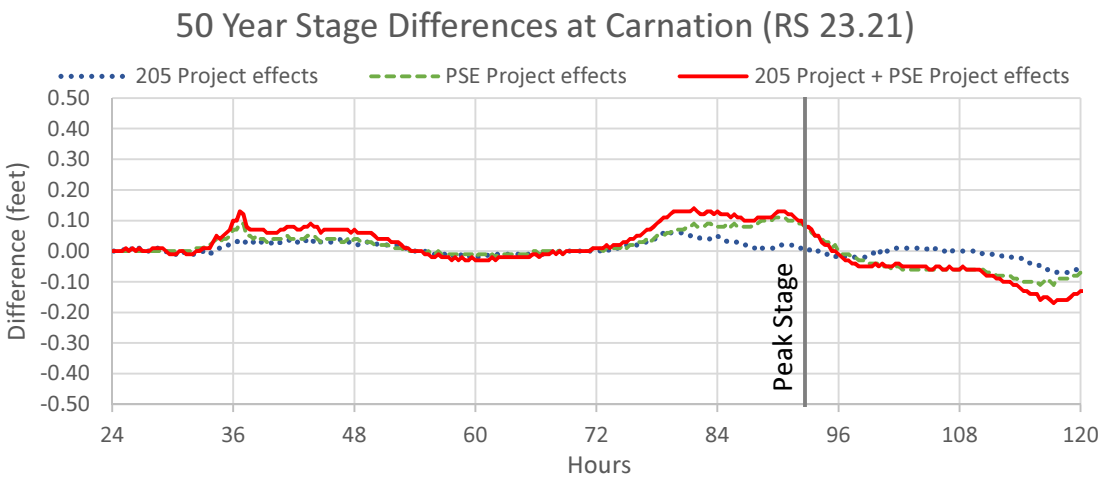

50 Year Stage Hydrograph Downstream from Patterson Creek (RS 28.00)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
82
80
78
76
74
72
70
68
66
64
62
60
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

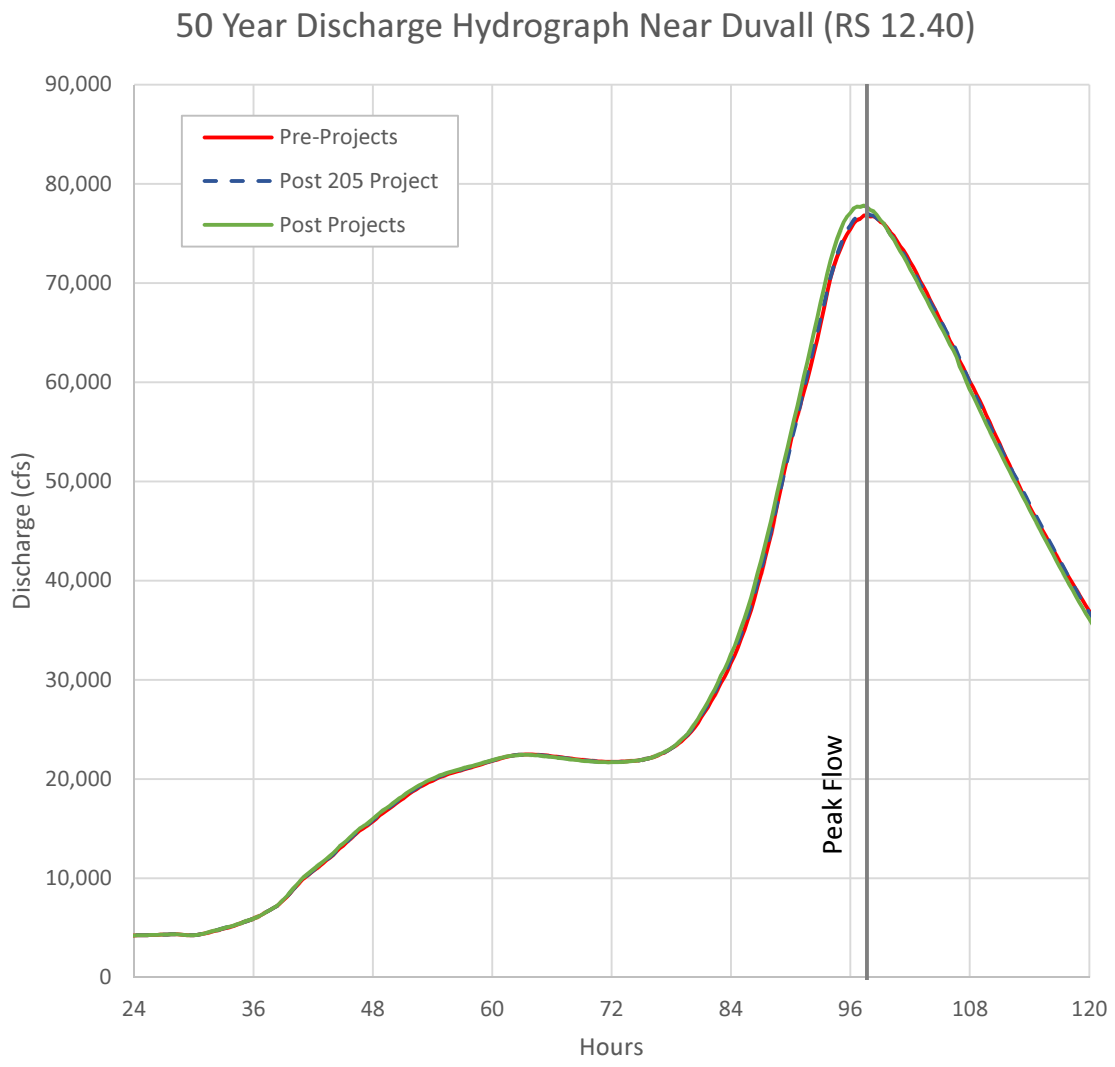

50 Year Stage Differences Downstream from Patterson Creek (RS 28.00)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

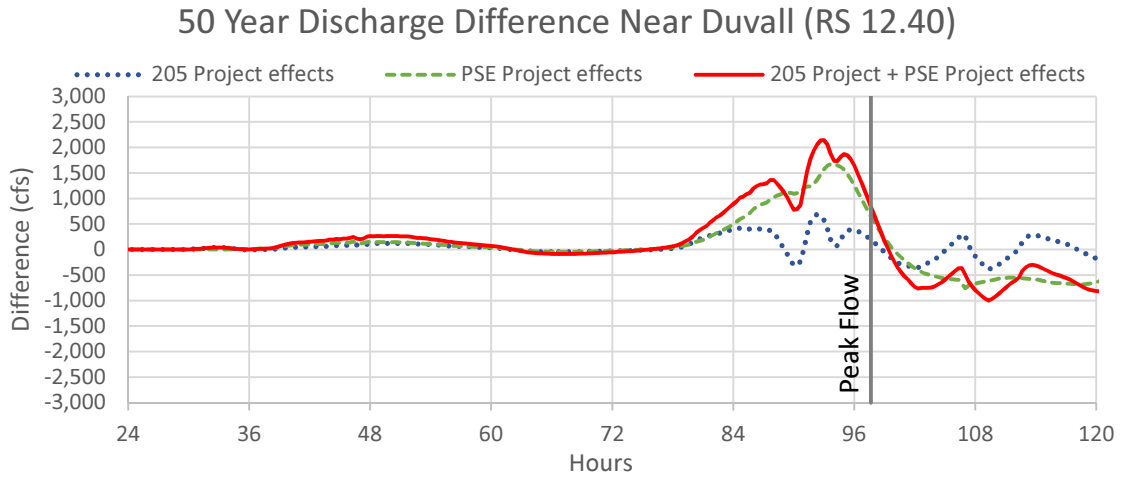

50 Year Discharge Hydrograph at Carnation (RS 23.21)
Pre-Projects
Post 205 Project
Post Projects
Discharge (cfs)
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
24
36
48
60
72
84
96
108
120
Hours
Peak Flow

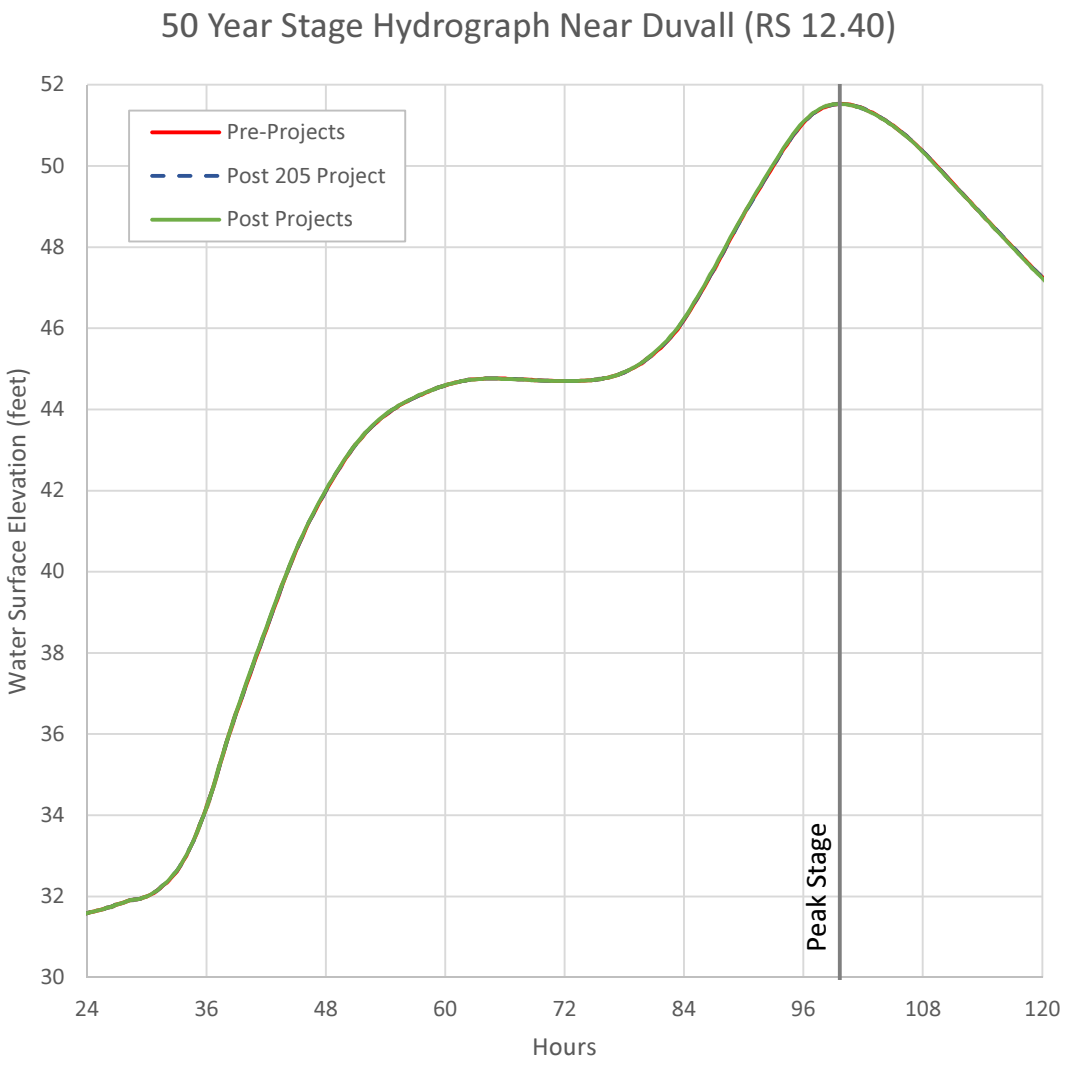

50 Year Discharge Differences at Carnation (RS 23.21)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
24
36
48
60
72
84
96
108
120
Hours
Peak Flow

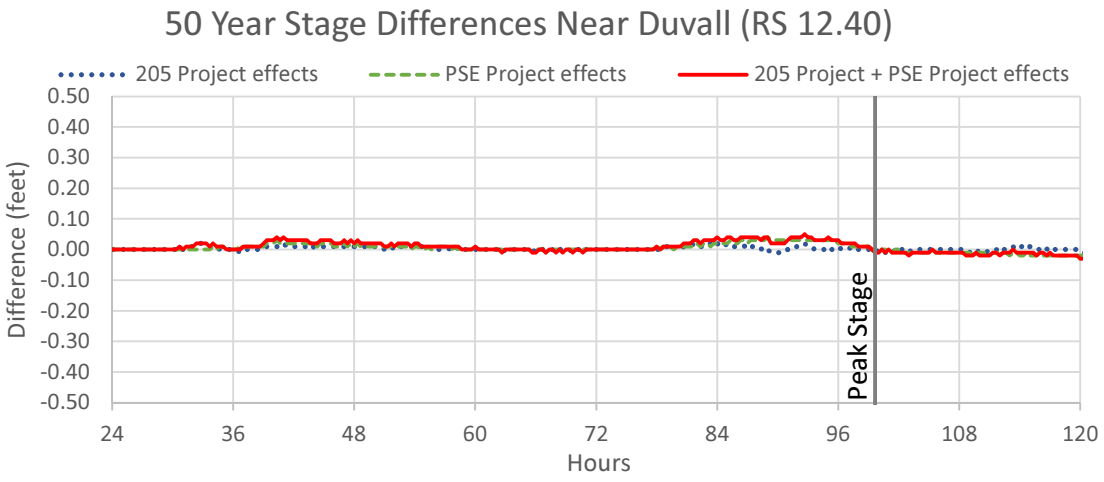

50 Year Stage Hydrograph at Carnation (RS 23.21)
Pre-Projects
Post 205 Project
Post Projects
Peak Stage
Water Surface Elevation (feet)
75
73
71
69
67
65
63
61
59
57
55
53
24
36
48
60
72
84
96
108
120
Hours

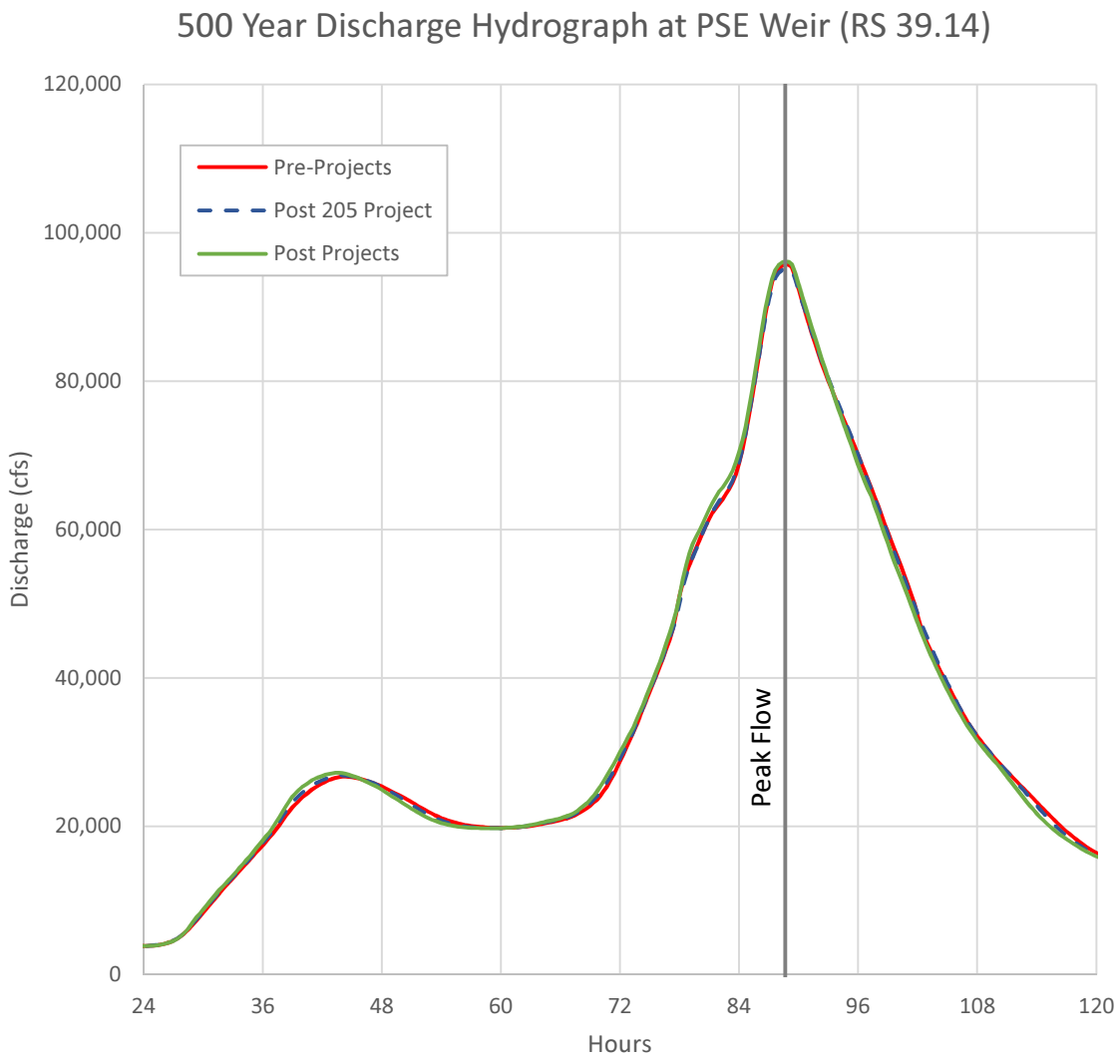

50 Year Stage Differences at Carnation (RS 23.21)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Stage
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
24
36
48
60
72
84
96
108
120
Hours

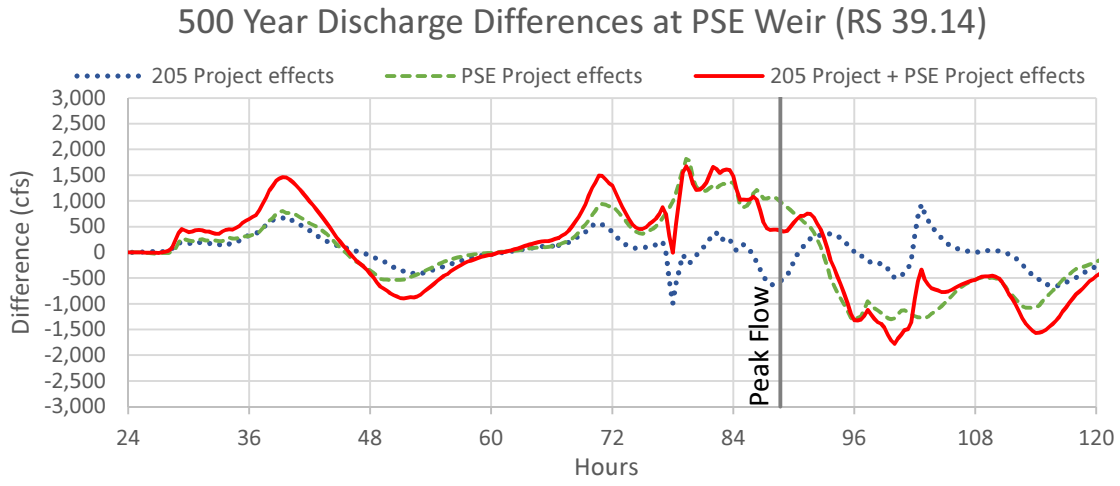

50 Year Discharge Hydrograph Near Duvall (RS 12.40)
Pre-Projects
Post 205 Project
Post Projects
Discharge (cfs)
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
24
36
48
60
72
84
96
108
120
Hours
Peak Flow

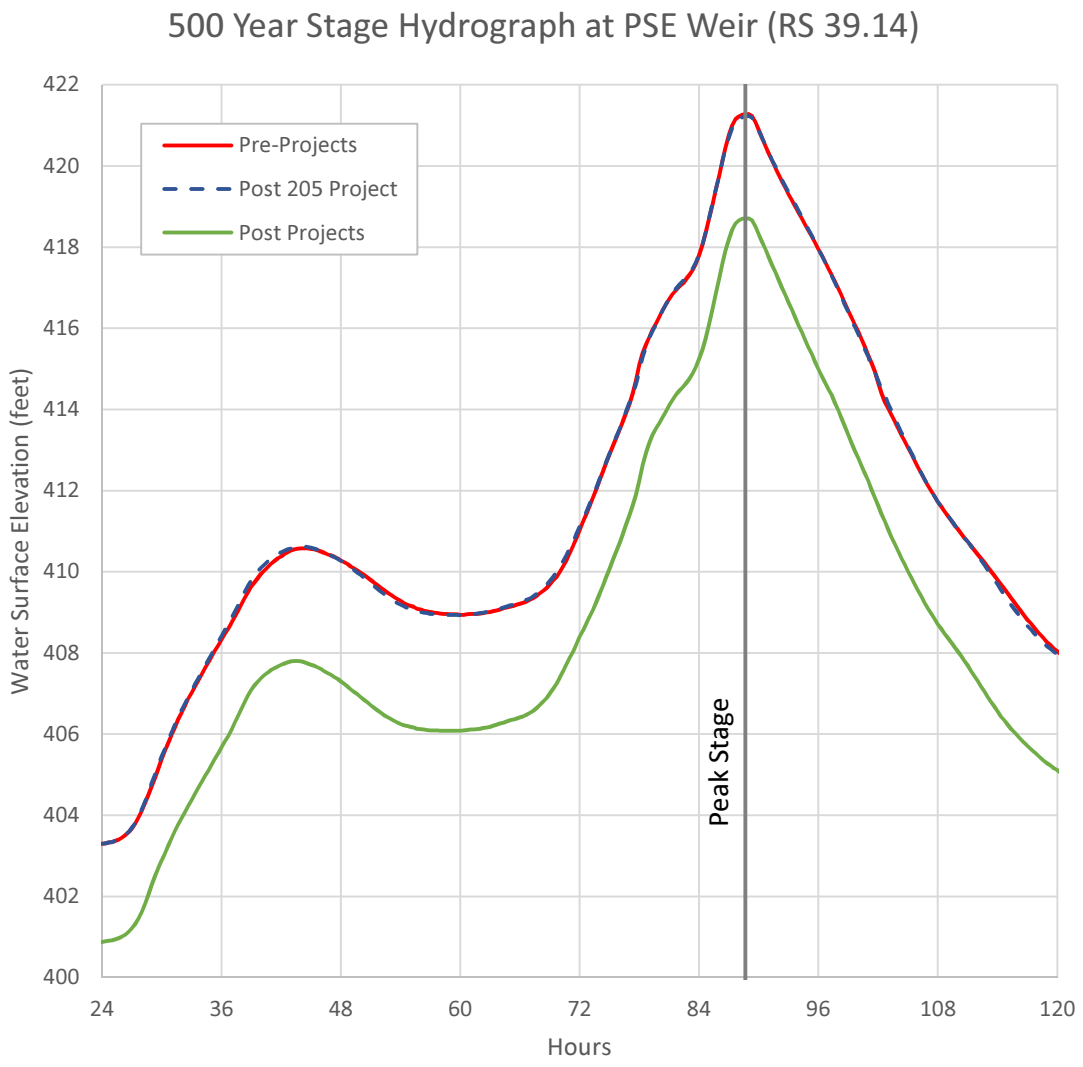

50 Year Discharge Difference Near Duvall (RS 12.40)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
24
36
48
60
72
84
96
108
120
Hours
Peak Flow

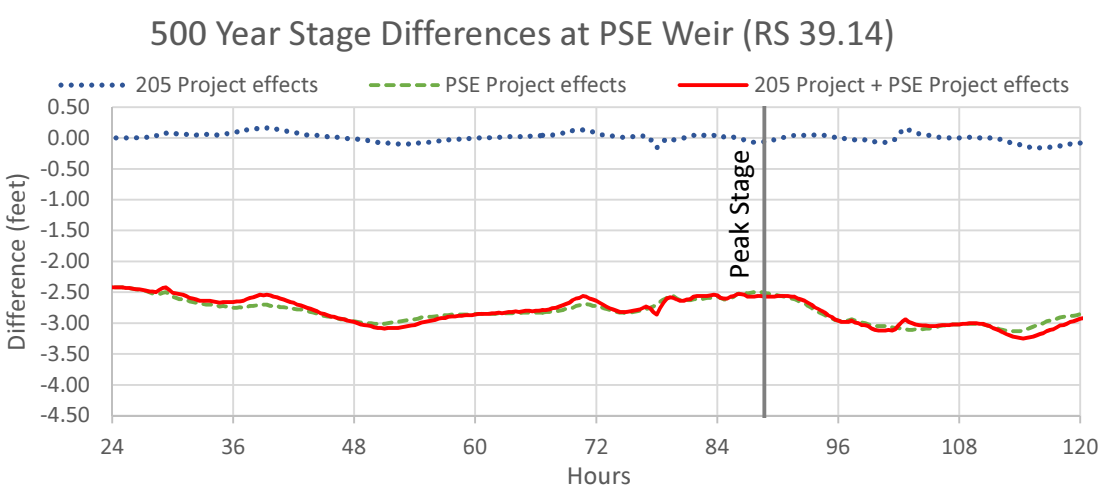

50 Year Stage Hydrograph Near Duvall (RS 12.40)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
52
50
48
46
44
42
40
38
36
34
32
30
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

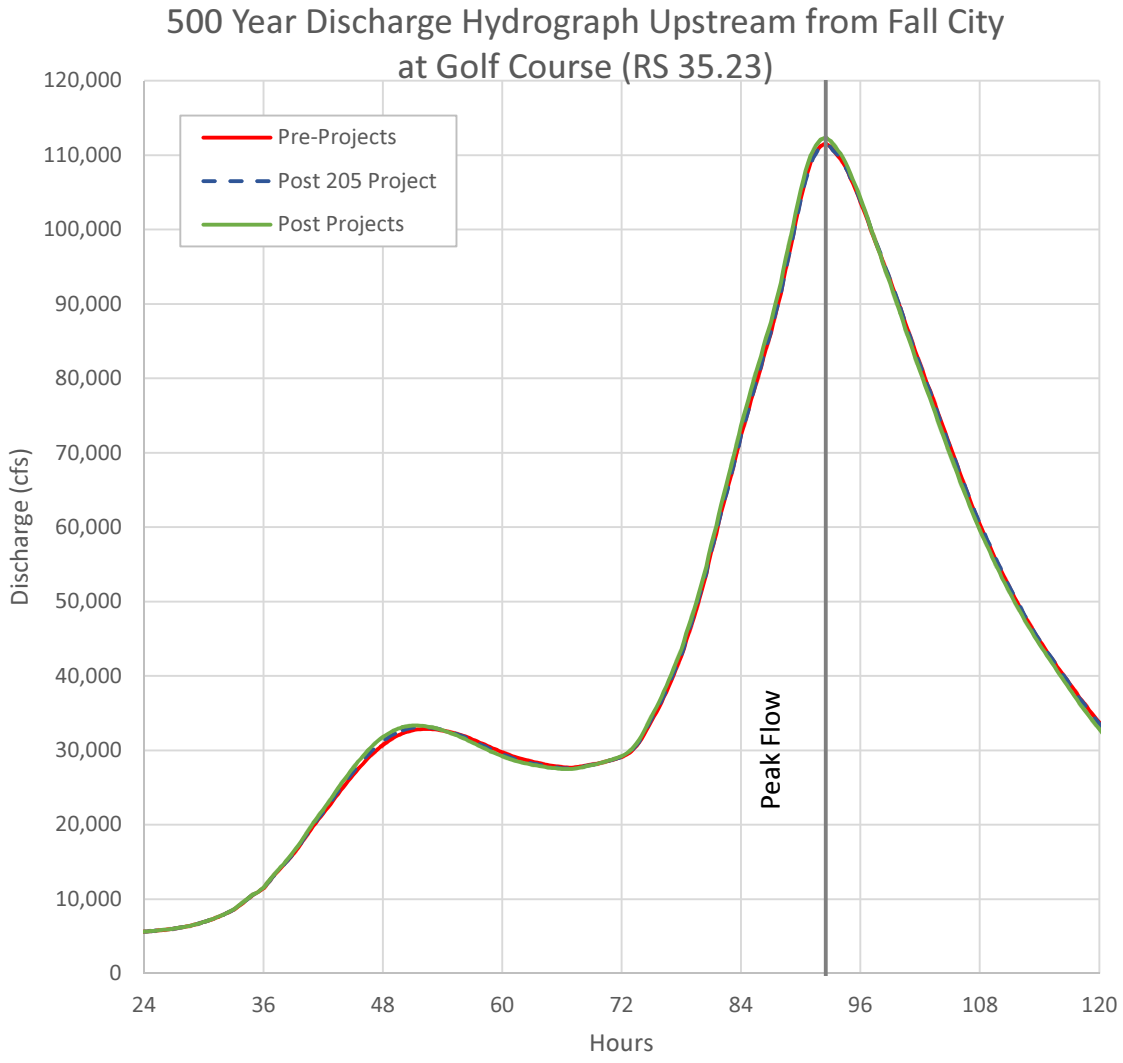

50 Year Stage Differences Near Duvall (RS 12.40)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

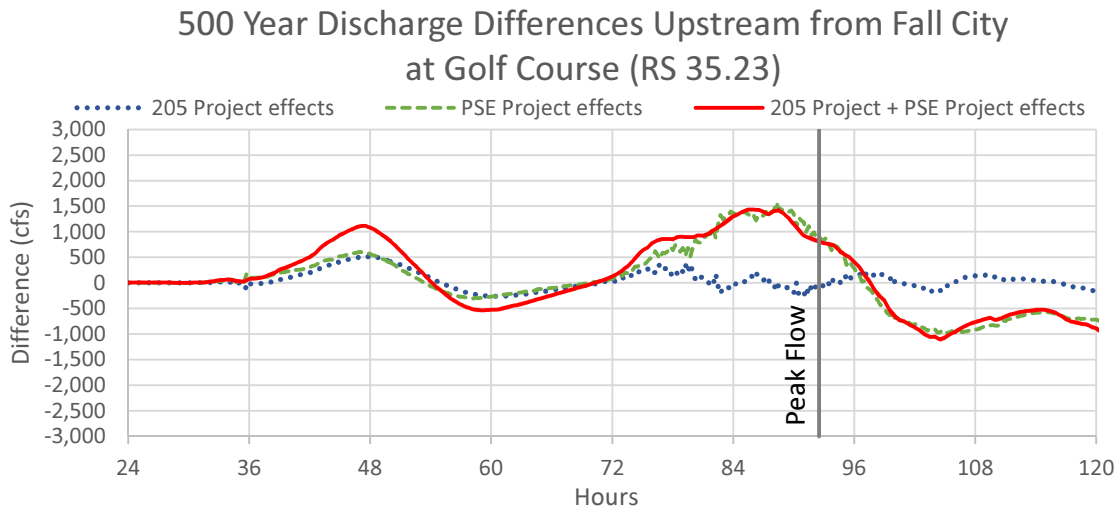

500 Year Discharge Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Peak Flow
Discharge (cfs)
120,000
100,000
80,000
60,000
40,000
20,000
0
24
36
48
60
72
84
96
108
120
Hours

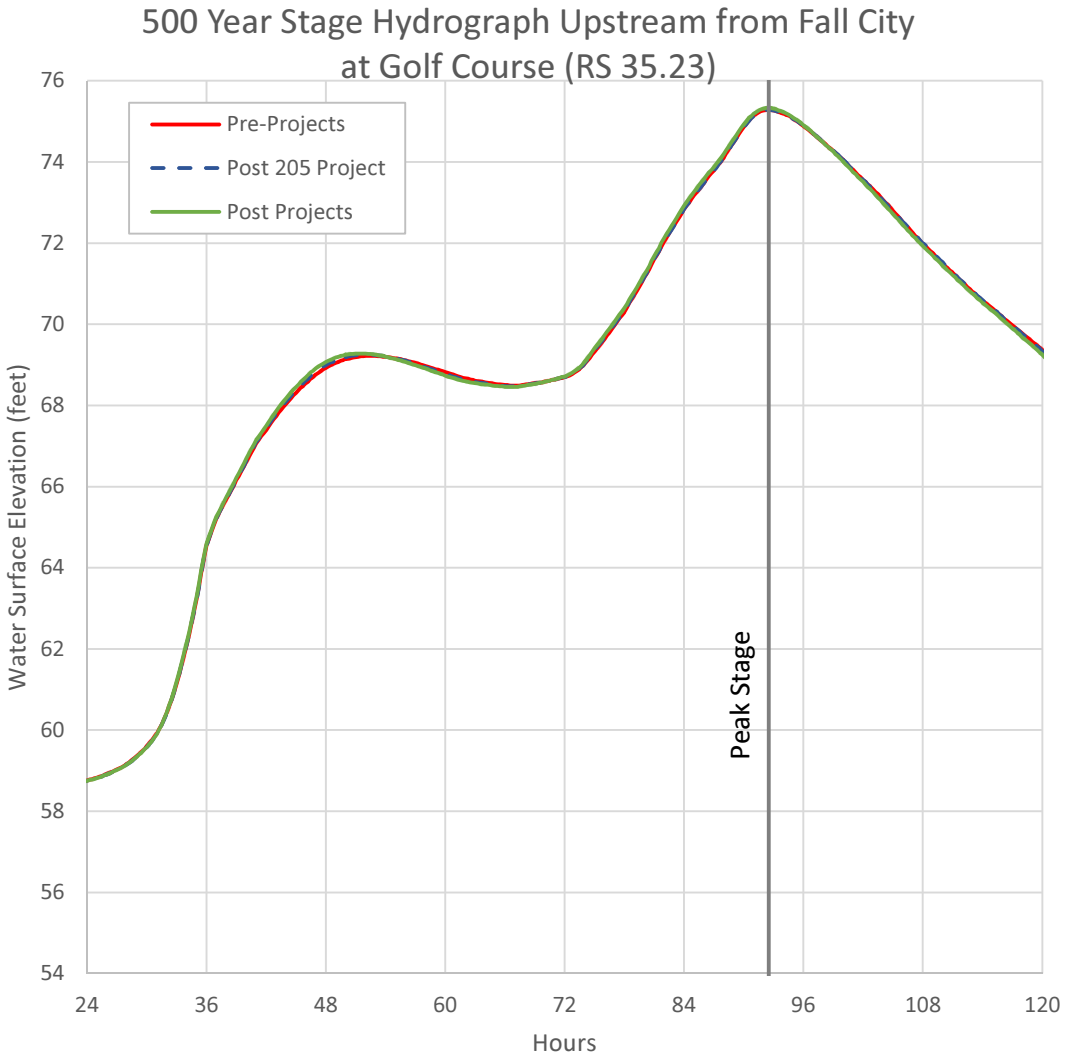

500 Year Discharge Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Flow
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
24
36
48
60
72
84
96
108
120
Hours

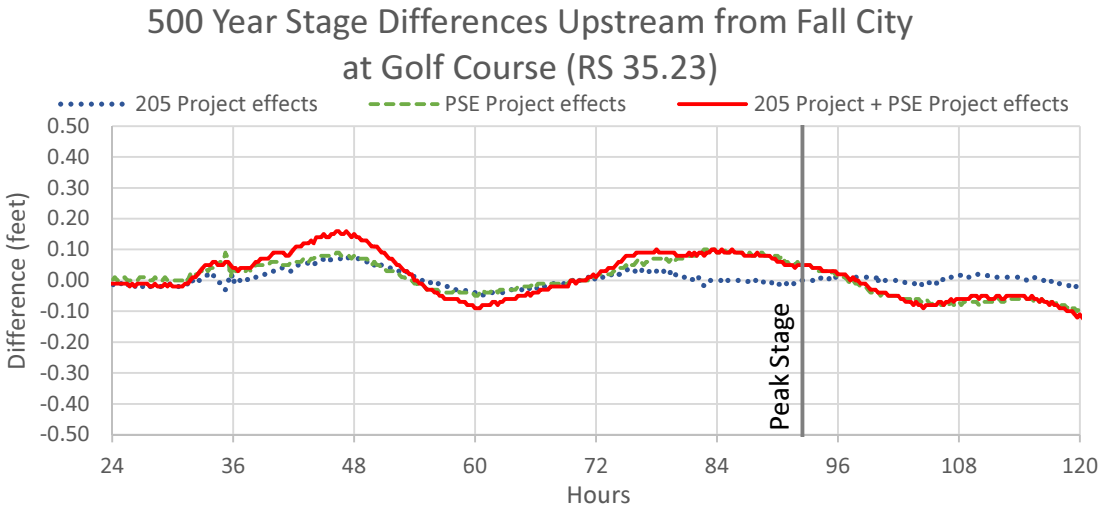

500 Year Stage Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Peak Stage
Water Surface Elevation (feet)
422
420
418
416
414
412
410
408
406
404
402
400
24
36
48
60
72
84
96
108
120
Hours


500 Year Stage Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Stage
Difference (feet)
0.50
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
-3.50
-4.00
-4.50
24
36
48
60
72
84
96
108
120
Hours


500 Year Discharge Hydrograph Upstream from Fall City at Golf Course (RS 35.23)
Pre-Projects
Post 205 Project
Post Projects
Peak Flow
Discharge (cfs)
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
24
36
48
60
72
84
96
108
120
Hours


500 Year Discharge Differences Upstream from Fall City at Golf Course (RS 35.23)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Flow
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
24
36
48
60
72
84
96
108
120
Hours


500 Year Stage Hydrograph Upstream from Fall City
at Golf Course (RS 35.23)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
76
74
72
70
68
66
64
62
60
58
56
54
24
36
48
60
72
84
96
108
120
Hours
Peak Stage


500 Year Stage Differences Upstream from Fall City
at Golf Course (RS 35.23)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

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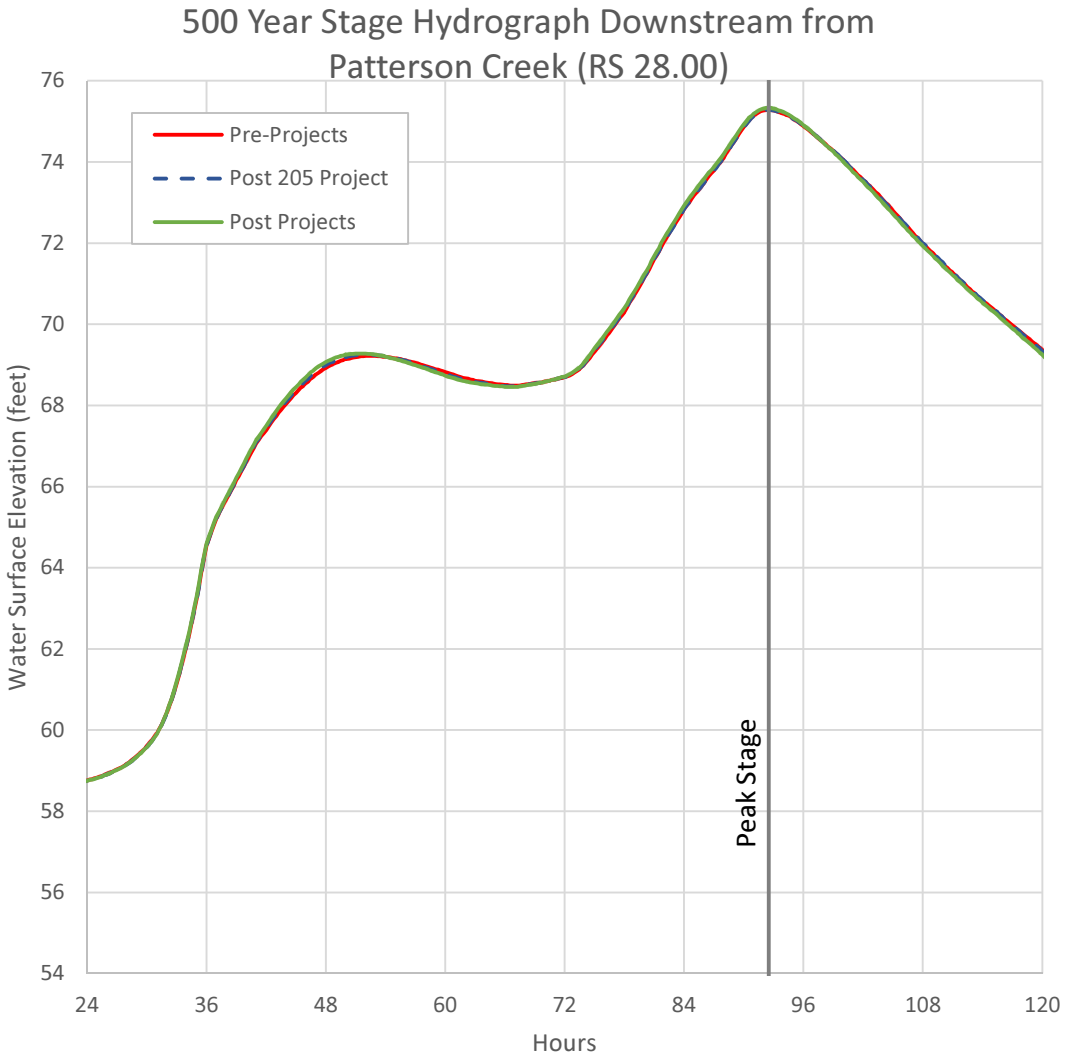
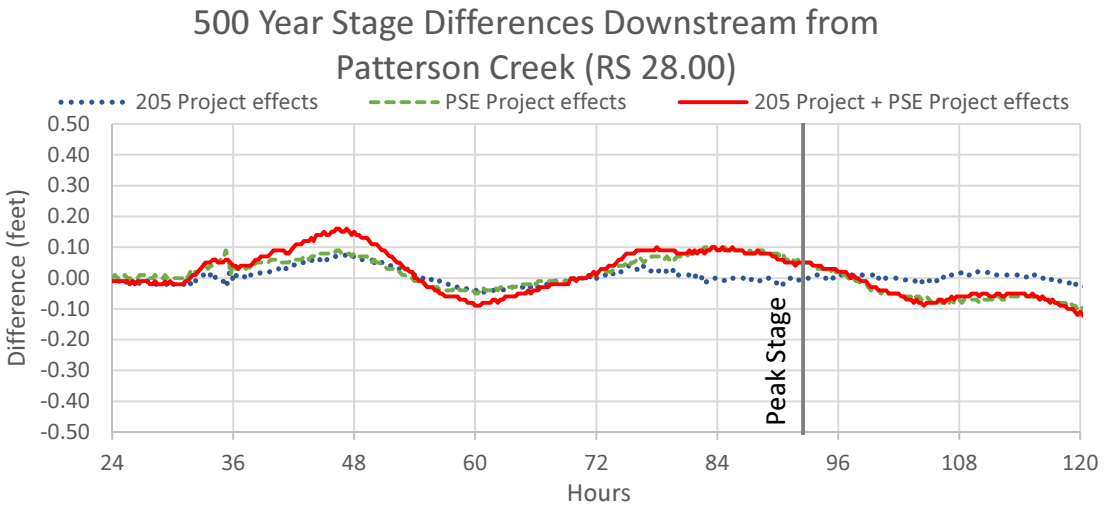
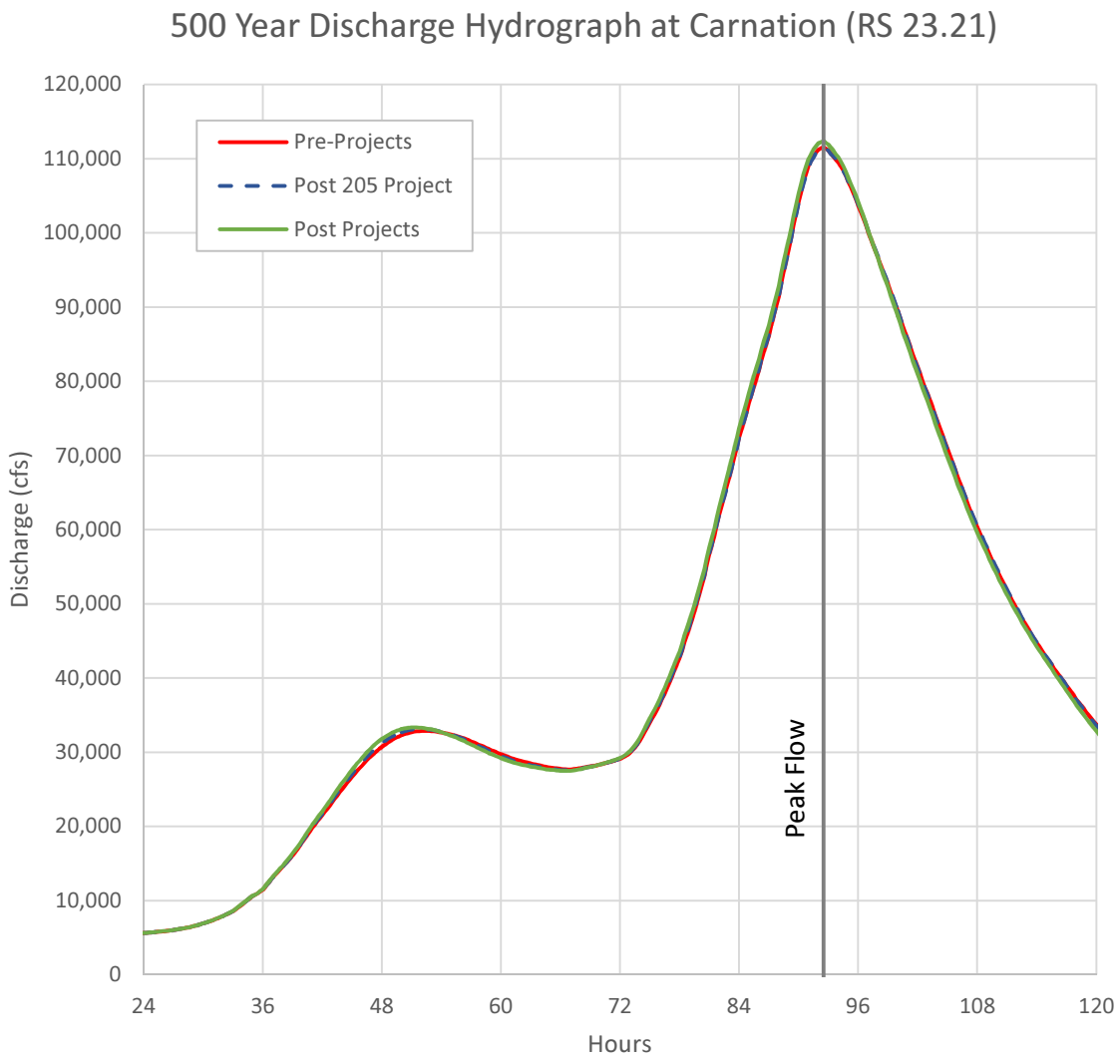
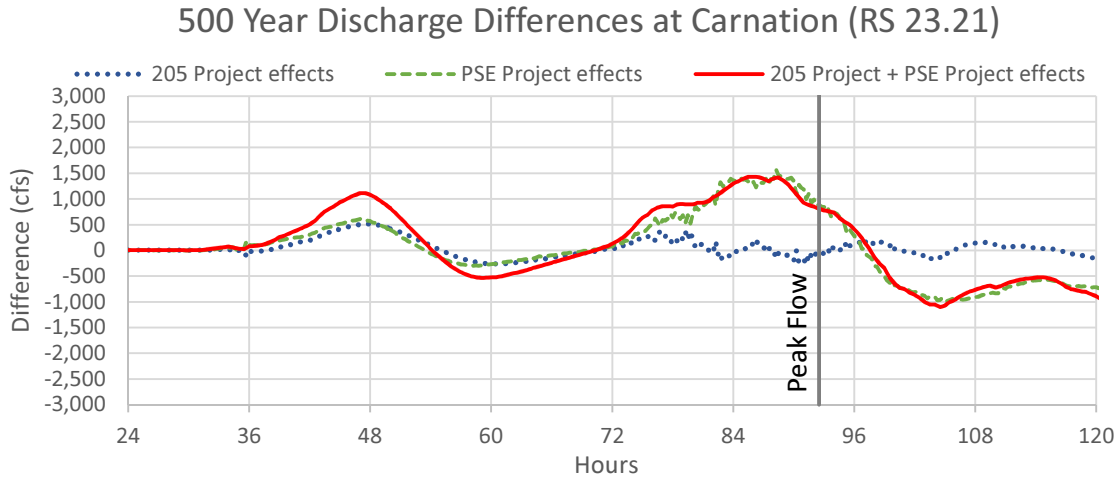
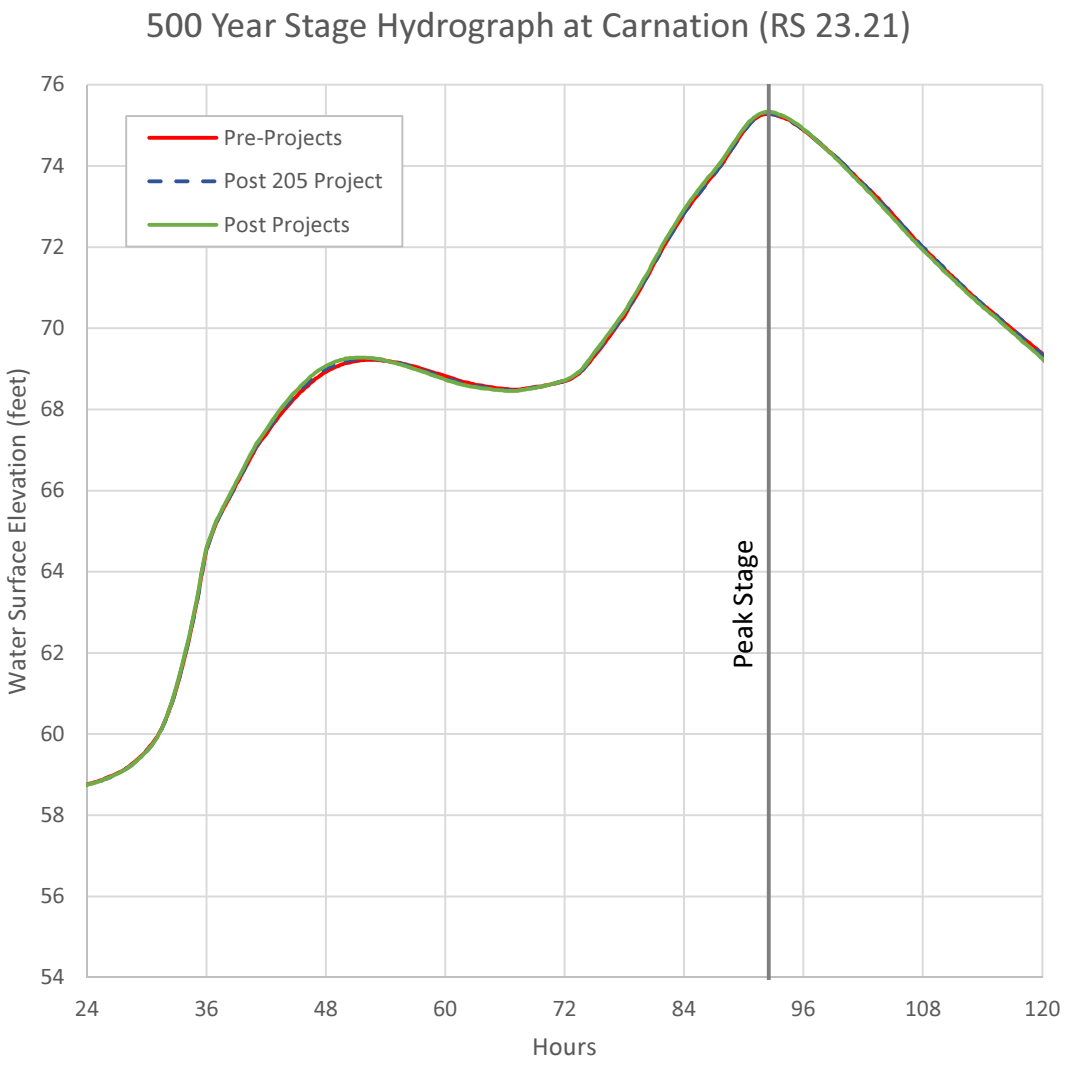
Only stage results are reported for Patterson Creek because there are multiple parallel flow paths in the HEC-RAS model at this location and determining the total flow passing this point is difficult

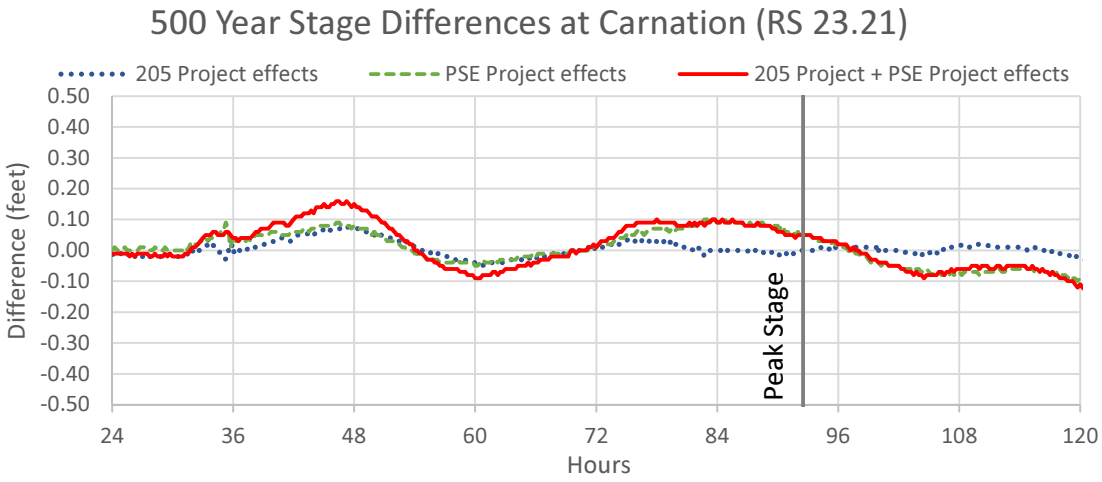

500 Year Stage Hydrograph Downstream from Patterson Creek (RS 28.00)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
76
74
72
70
68
66
64
62
60
58
56
54
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

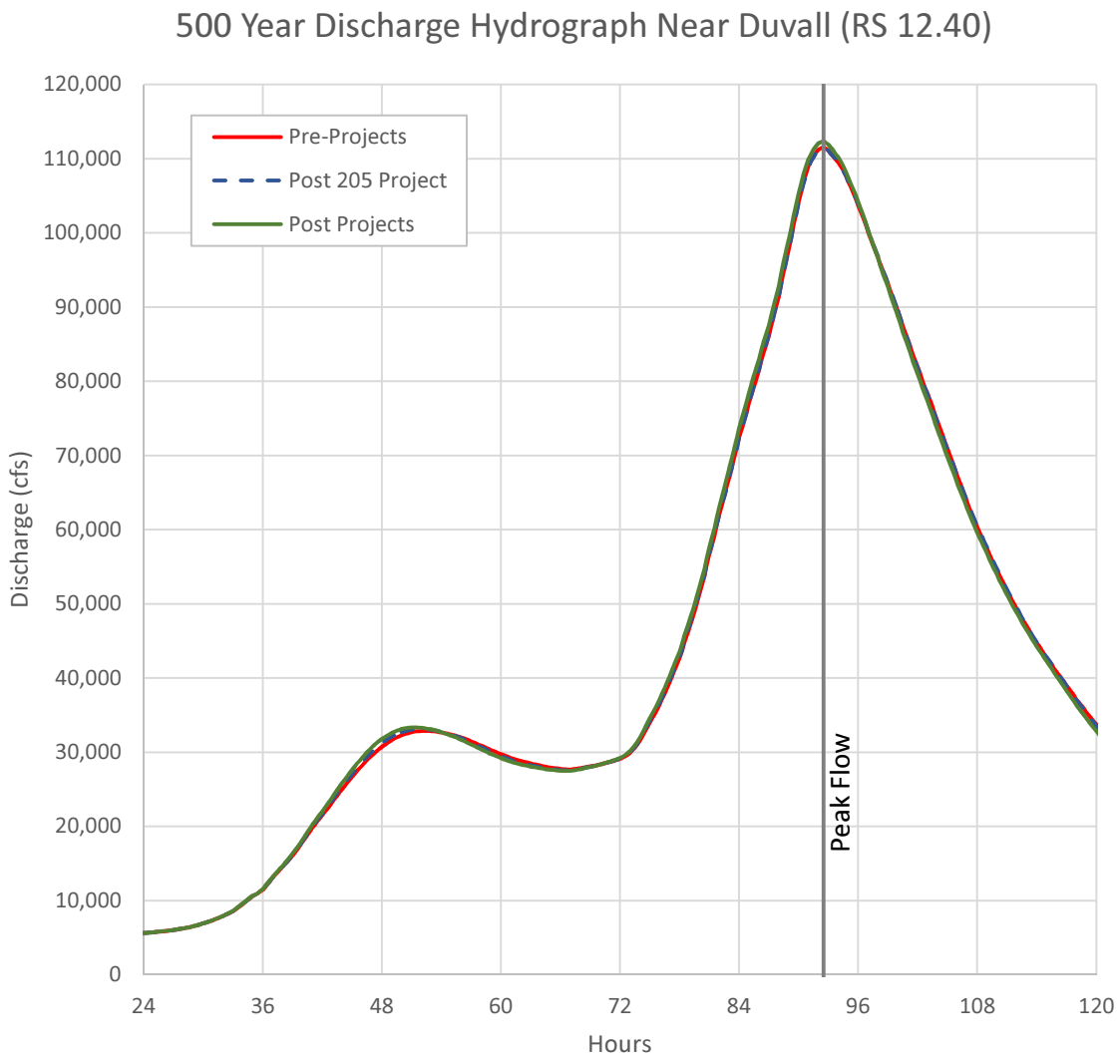

500 Year Stage Differences Downstream from Patterson Creek (RS 28.00)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

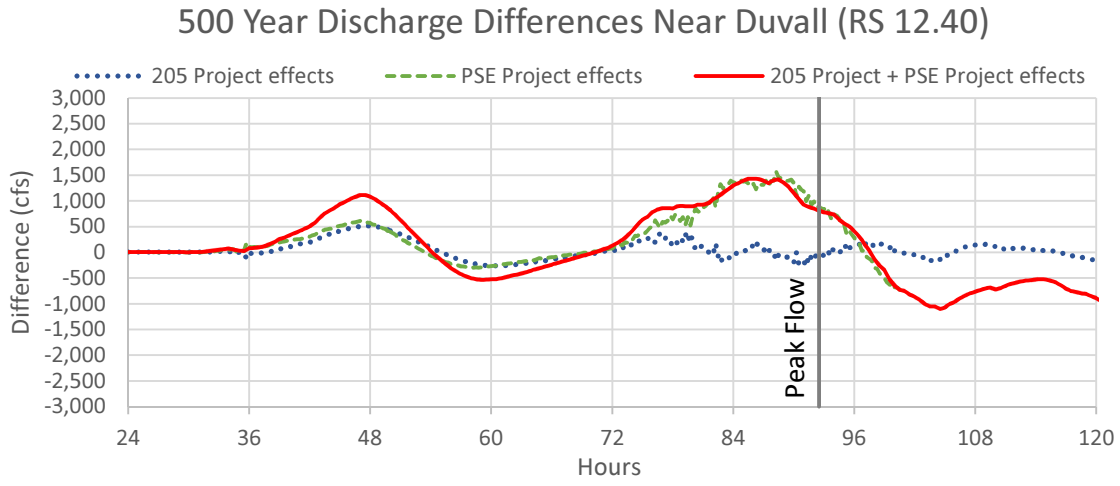

500 Year Discharge Hydrograph at Carnation (RS 23.21)
Pre-Projects
Post 205 Project
Post Projects
Discharge (cfs)
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
24
36
48
60
72
84
96
108
120
Hours
Peak Flow

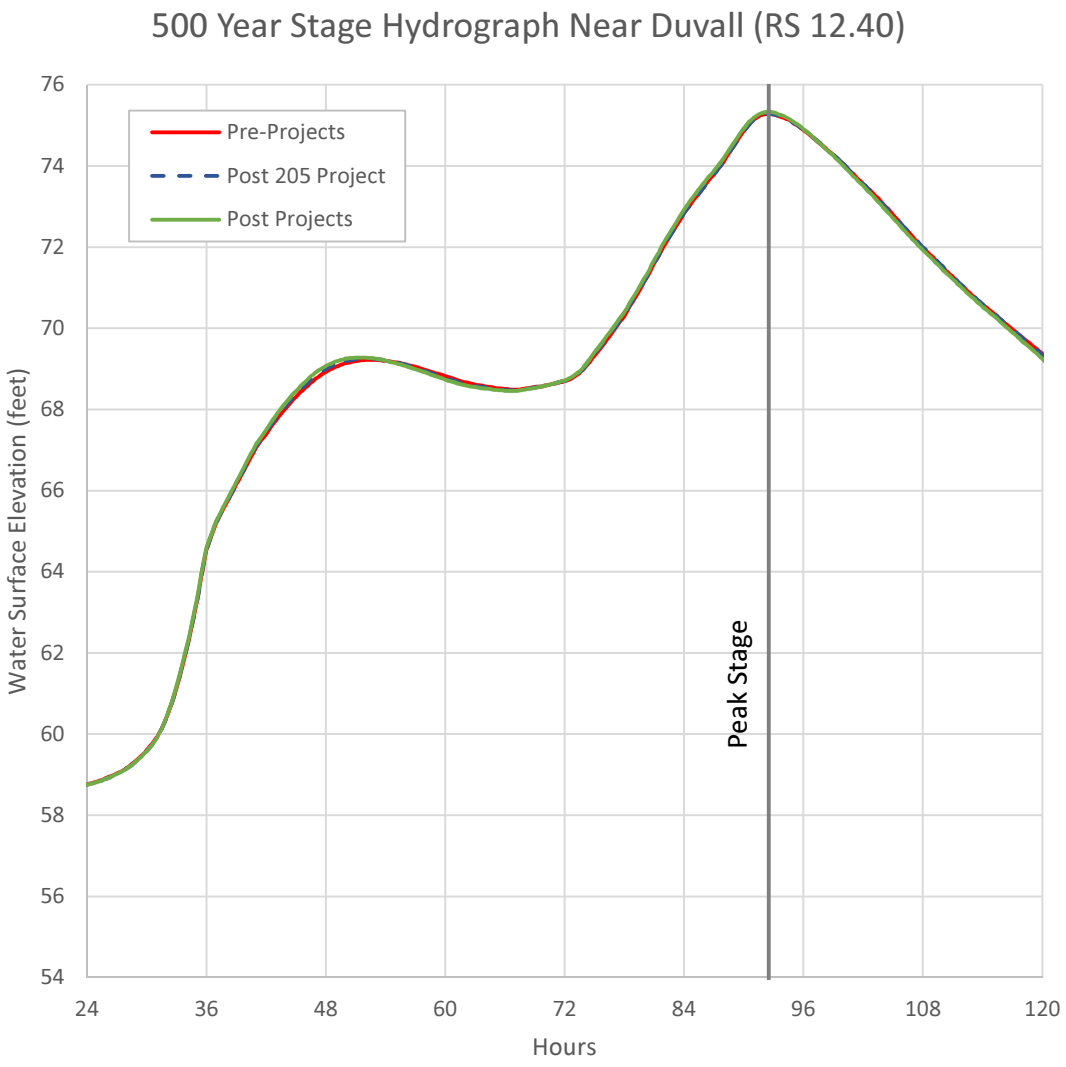

500 Year Discharge Differences at Carnation (RS 23.21)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
24
36
48
60
72
84
96
108
120
Hours
Peak Flow

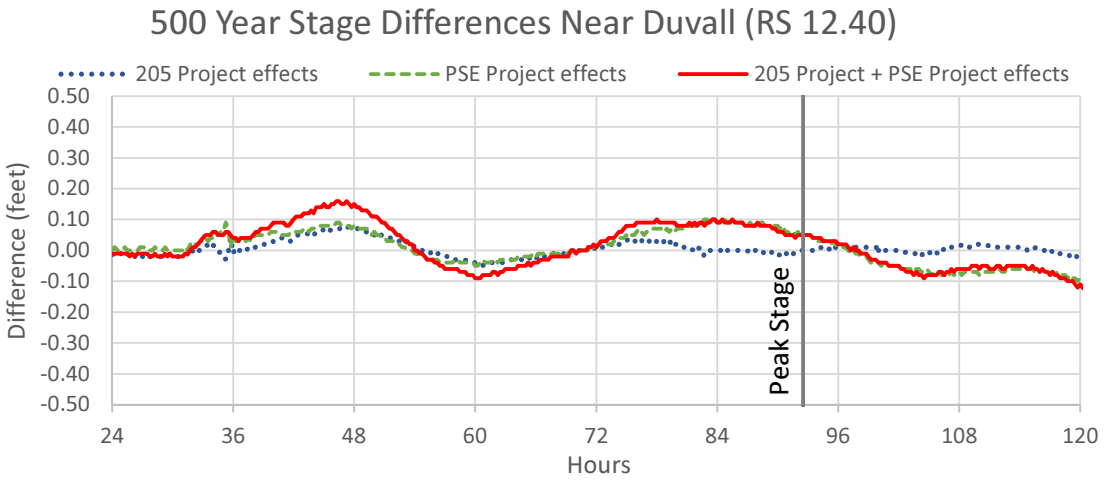

500 Year Stage Hydrograph at Carnation (RS 23.21)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
76
74
72
70
68
66
64
62
60
58
56
54
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

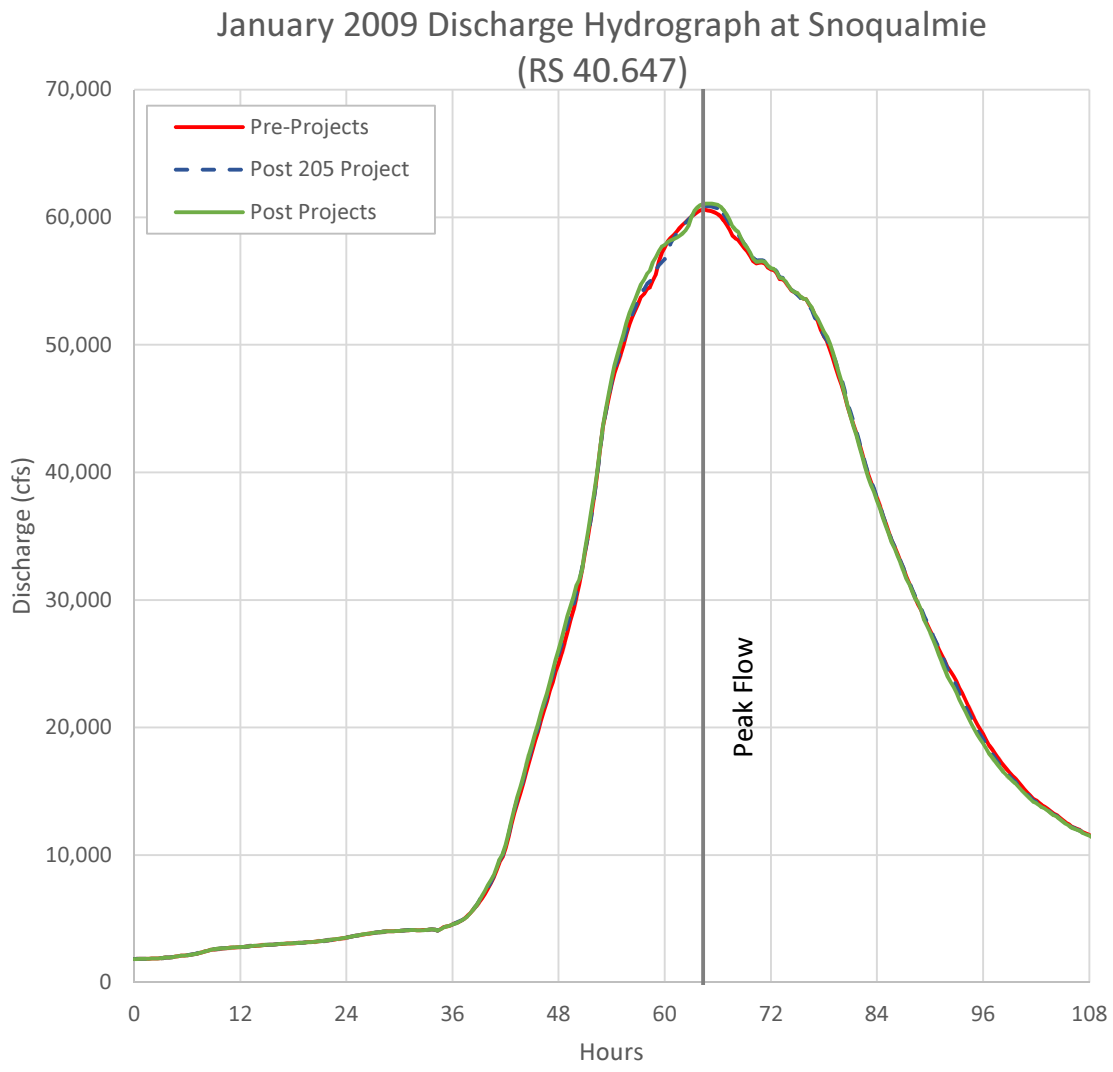

500 Year Stage Differences at Carnation (RS 23.21)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

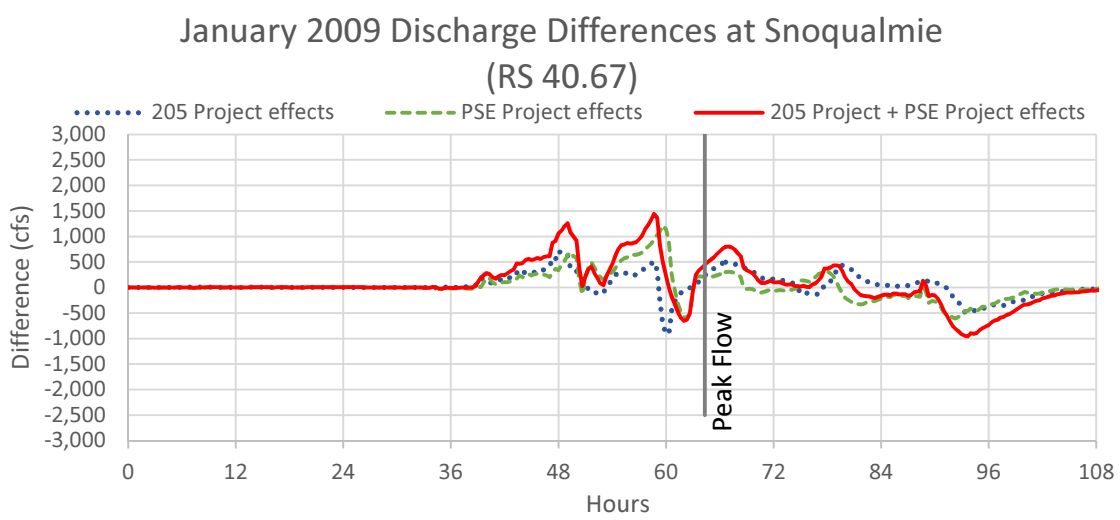

500 Year Discharge Hydrograph Near Duvall (RS 12.40)
Pre-Projects
Post 205 Project
Post Projects
Discharge (cfs)
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
24
36
48
60
72
84
96
108
120
Hours
Peak Flow

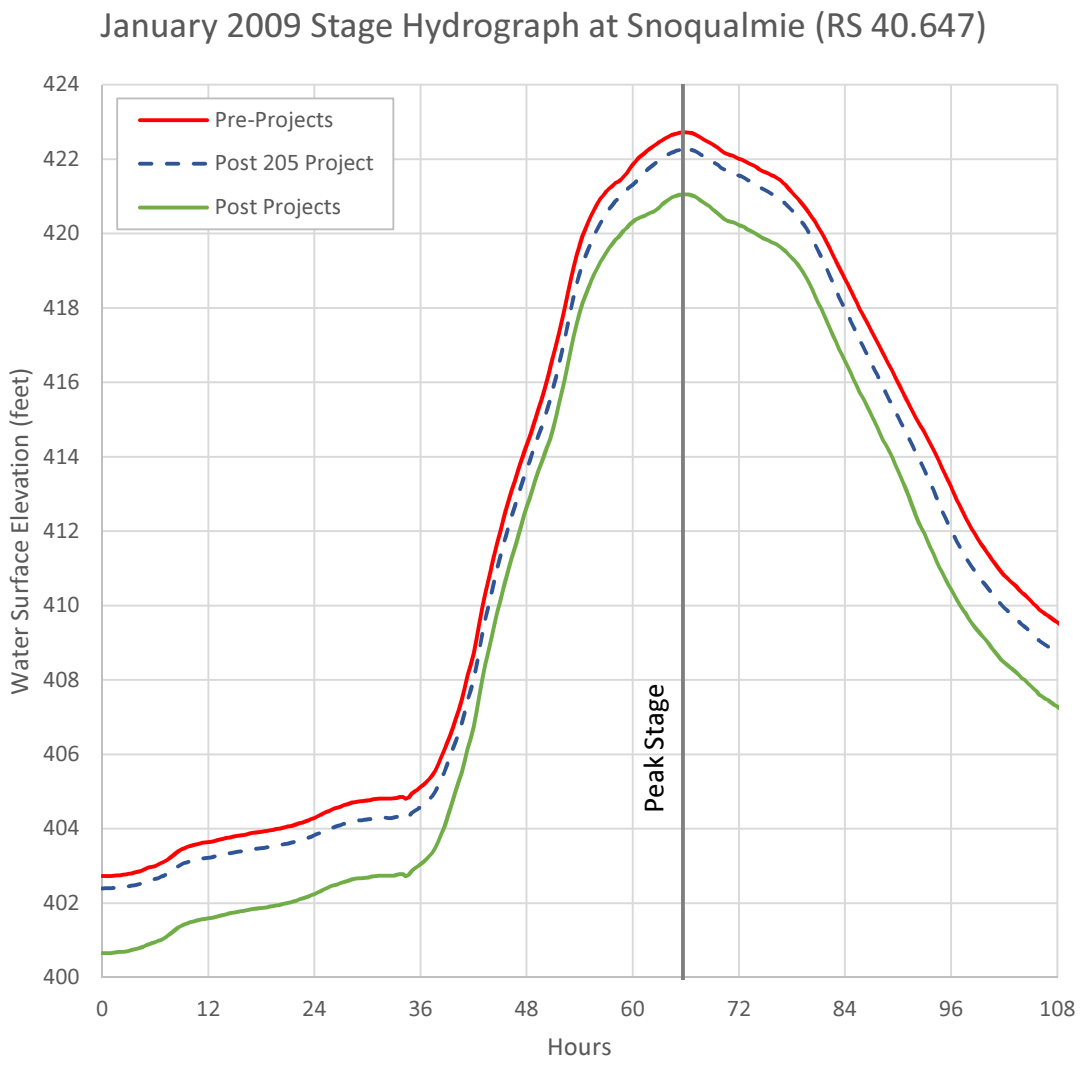

500 Year Discharge Differences Near Duvall (RS 12.40)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
24
36
48
60
72
84
96
108
120
Hours
Peak Flow

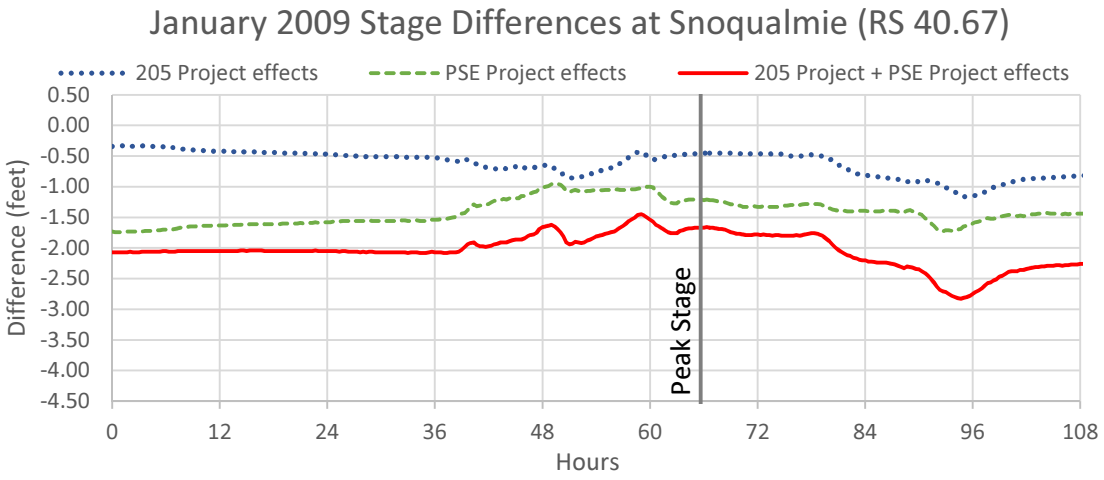

500 Year Stage Hydrograph Near Duvall (RS 12.40)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
76
74
72
70
68
66
64
62
60
58
56
54
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

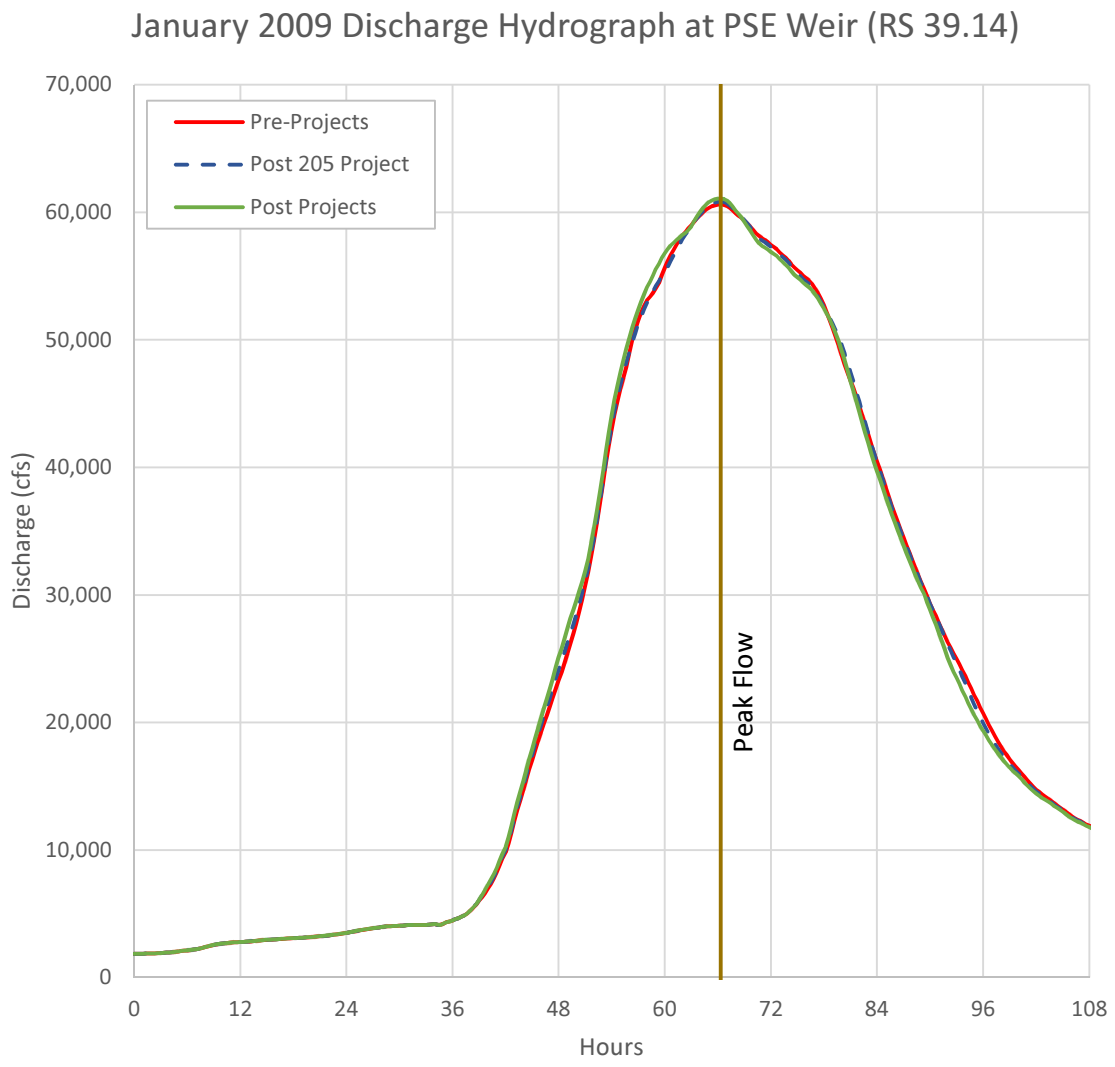

500 Year Stage Differences Near Duvall (RS 12.40)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
24
36
48
60
72
84
96
108
120
Hours
Peak Stage

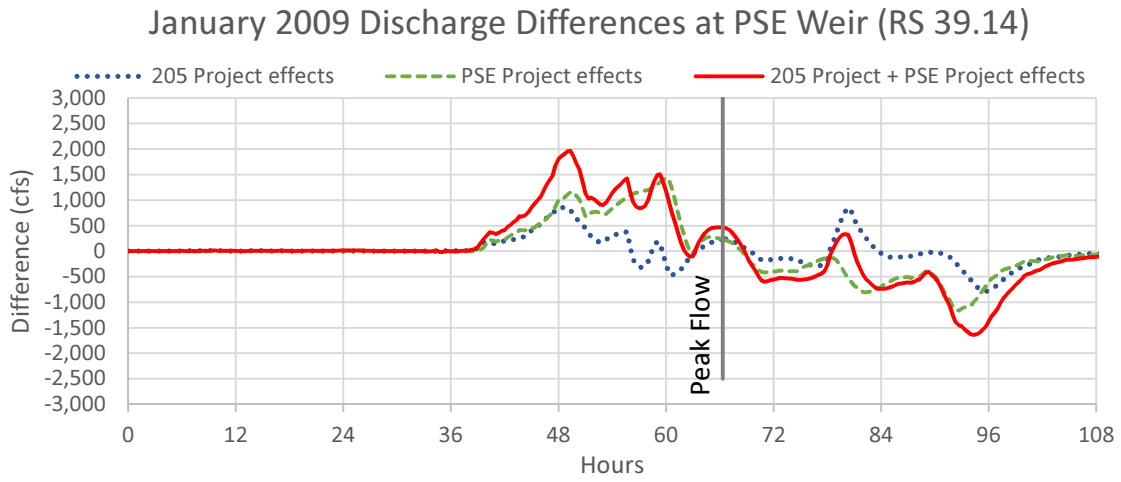

January 2009 Discharge Hydrograph at Snoqualmie
(RS 40.647)
Pre-Projects
Post 205 Project
Post Projects
Peak Flow
Discharge (cfs)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
0
12
24
36
48
60
72
84
96
108
Hours

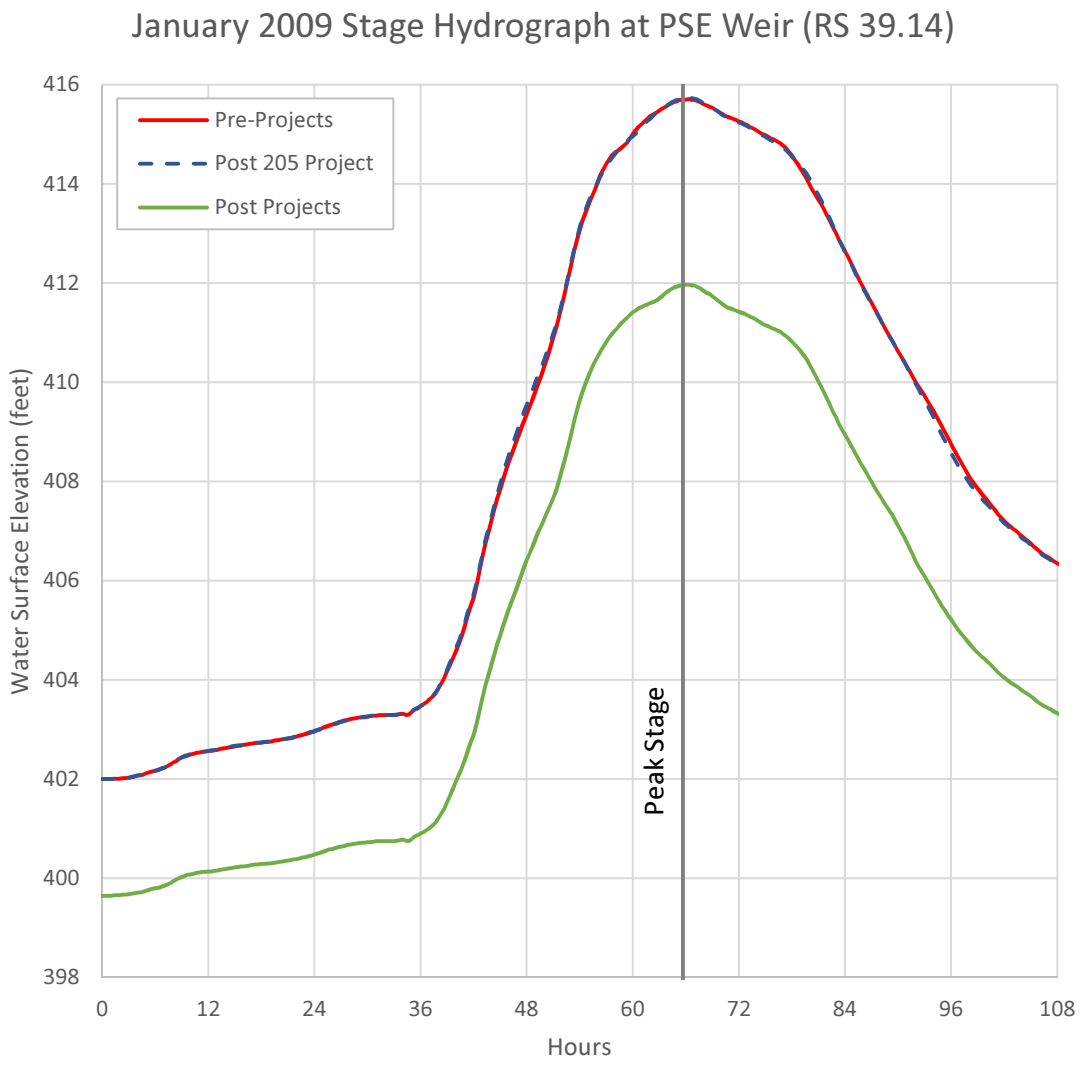

January 2009 Discharge Differences at Snoqualmie
(RS 40.67)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Flow
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
0
12
24
36
48
60
72
84
96
108
Hours

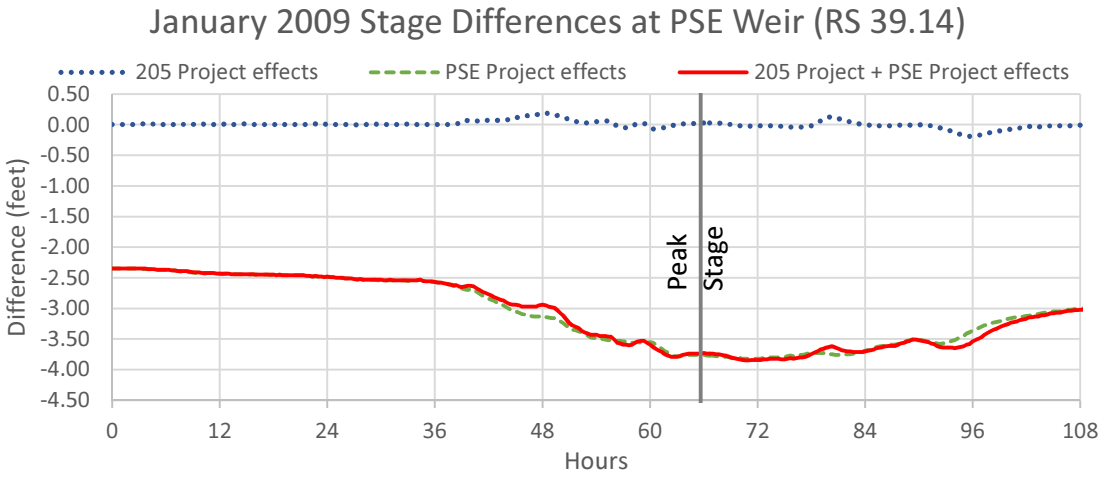

January 2009 Stage Hydrograph at Snoqualmie (RS 40.647)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
424
422
420
418
416
414
412
410
408
406
404
402
400
0
12
24
36
48
60
72
84
96
108
Hours
Peak Stage

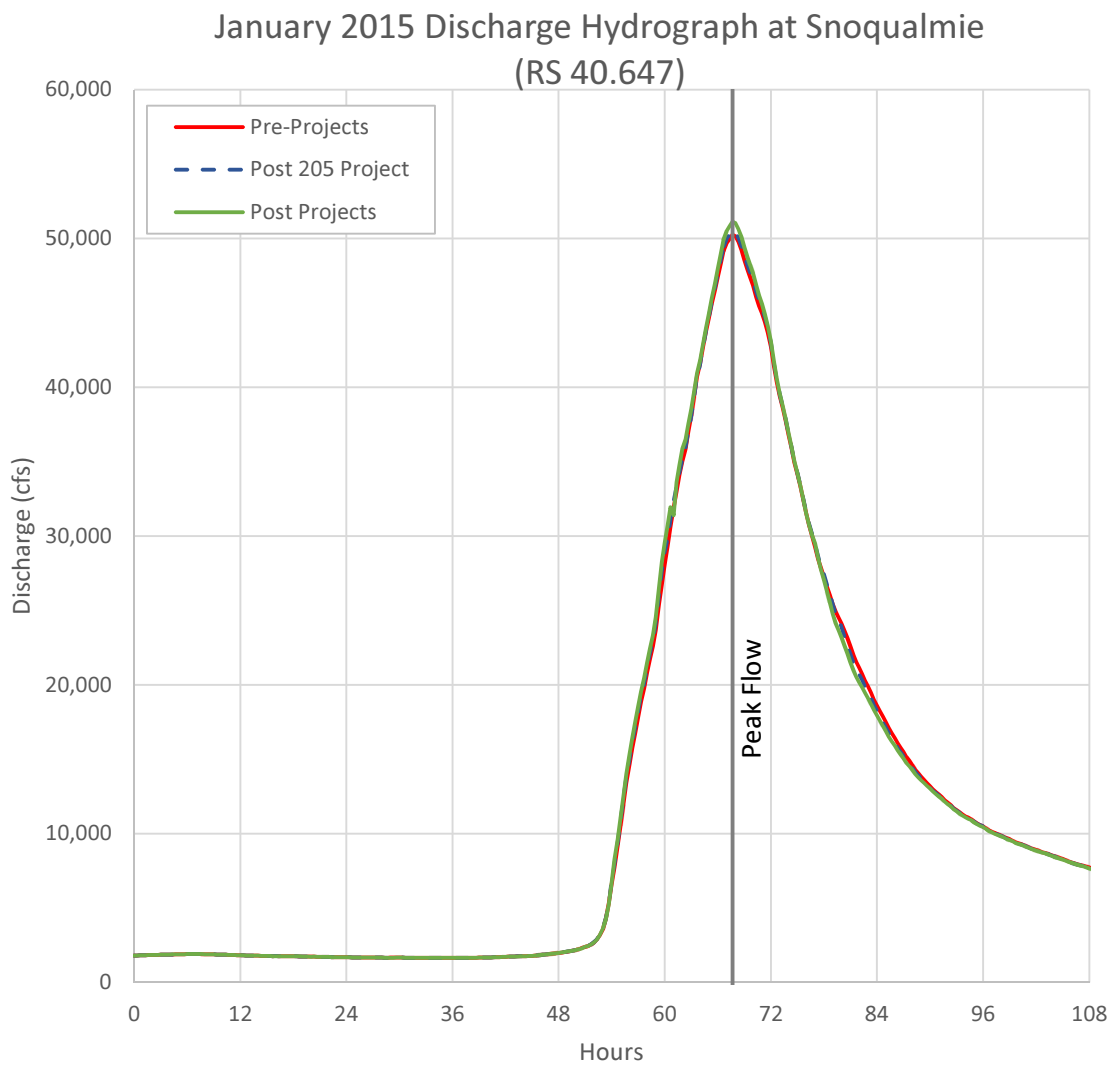

January 2009 Stage Differences at Snoqualmie (RS 40.67)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
-3.50
-4.00
-4.50
0
12
24
36
48
60
72
84
96
108
Hours
Peak Stage

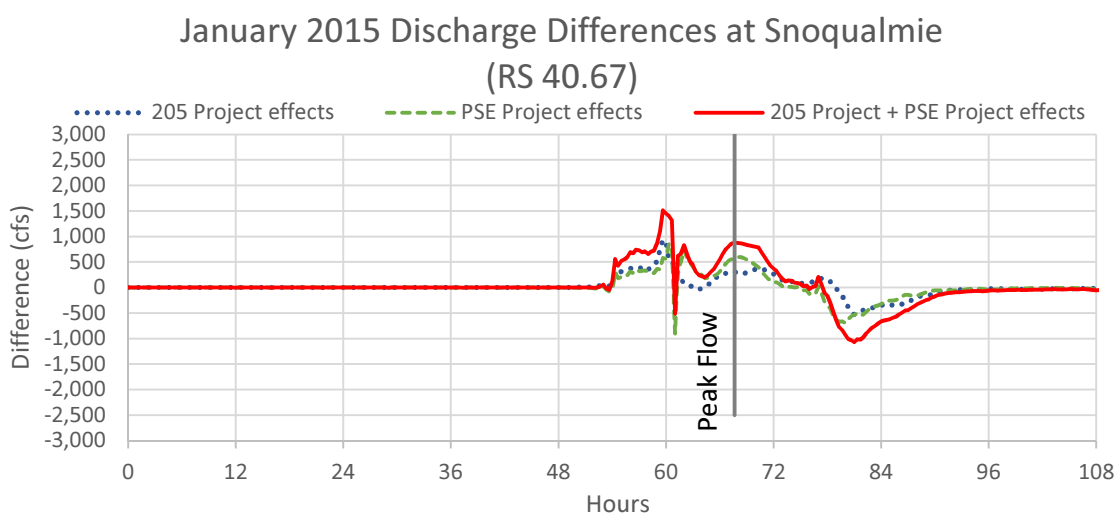

January 2009 Discharge Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Peak Flow
Discharge (cfs)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
0
12
24
36
48
60
72
84
96
108
Hours

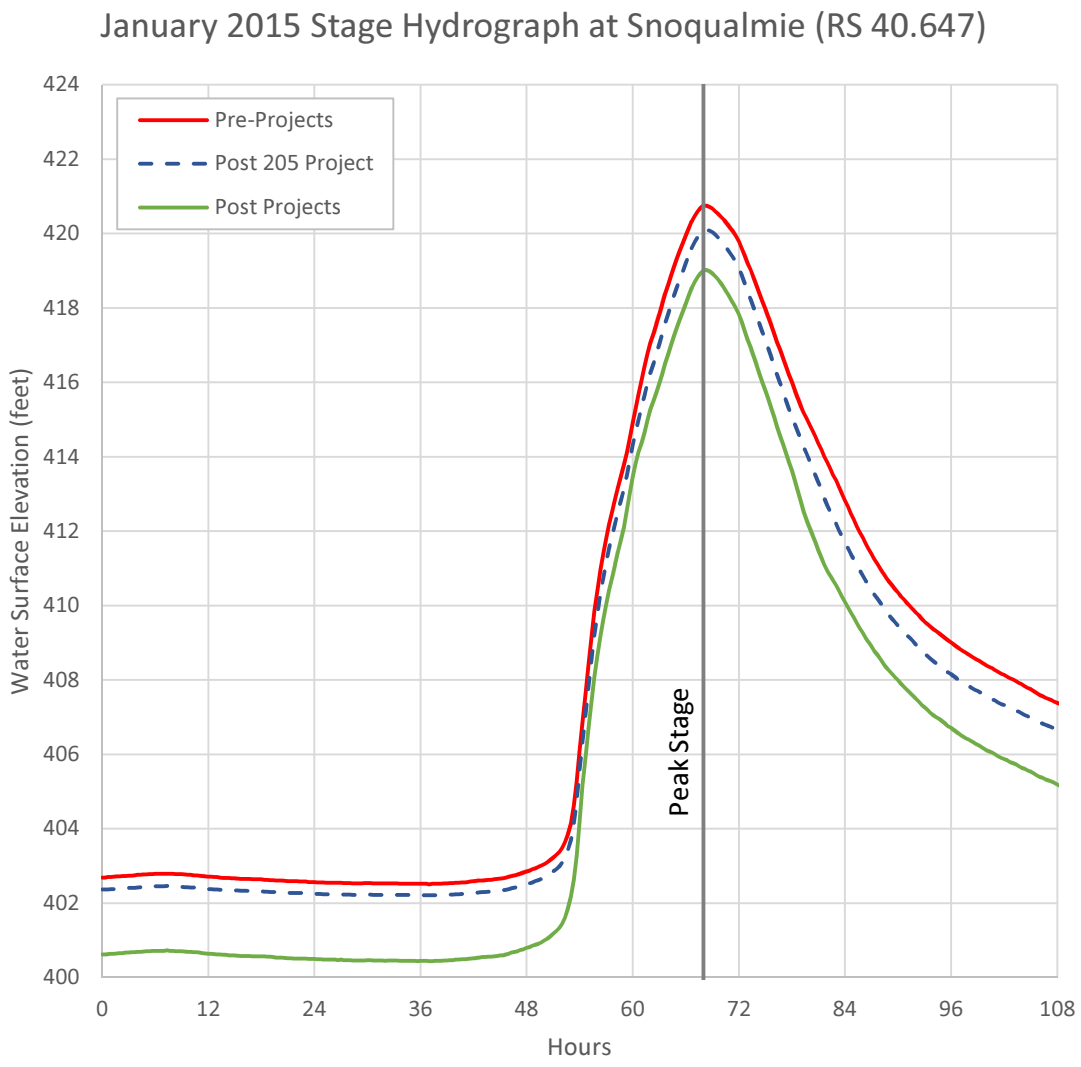

January 2009 Discharge Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Flow
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
0
12
24
36
48
60
72
84
96
108
Hours

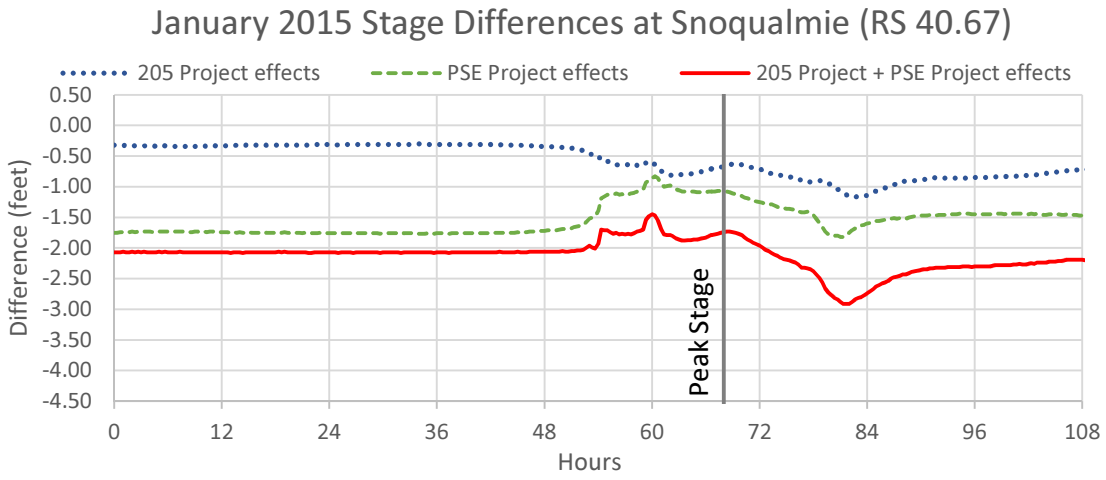

January 2009 Stage Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
416
414
412
410
408
406
404
402
400
398
0
12
24
36
48
60
72
84
96
108
Hours
Peak Stage

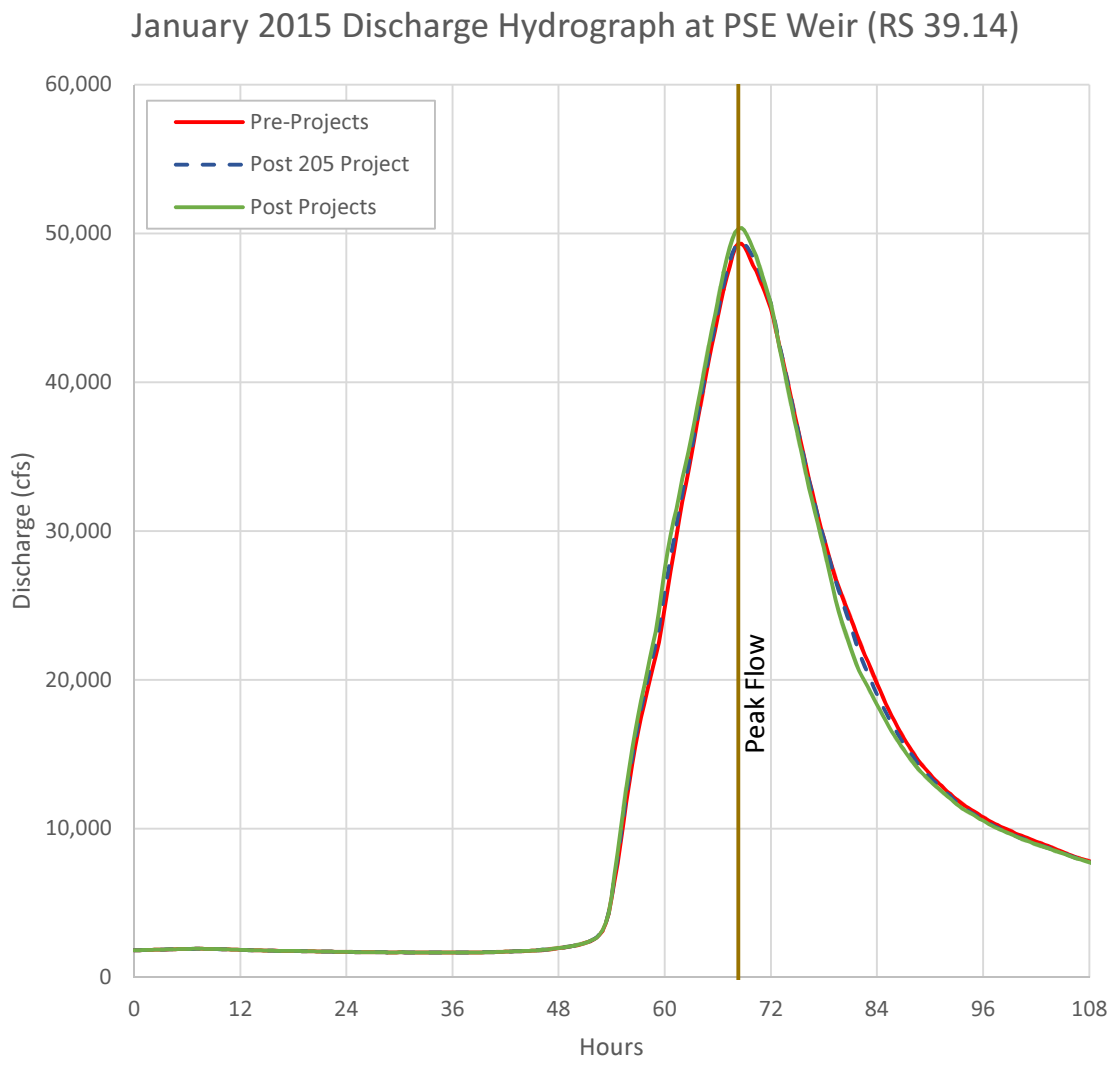

January 2009 Stage Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
-3.50
-4.00
-4.50
0
12
24
36
48
60
72
84
96
108
Hours
Peak
Stage

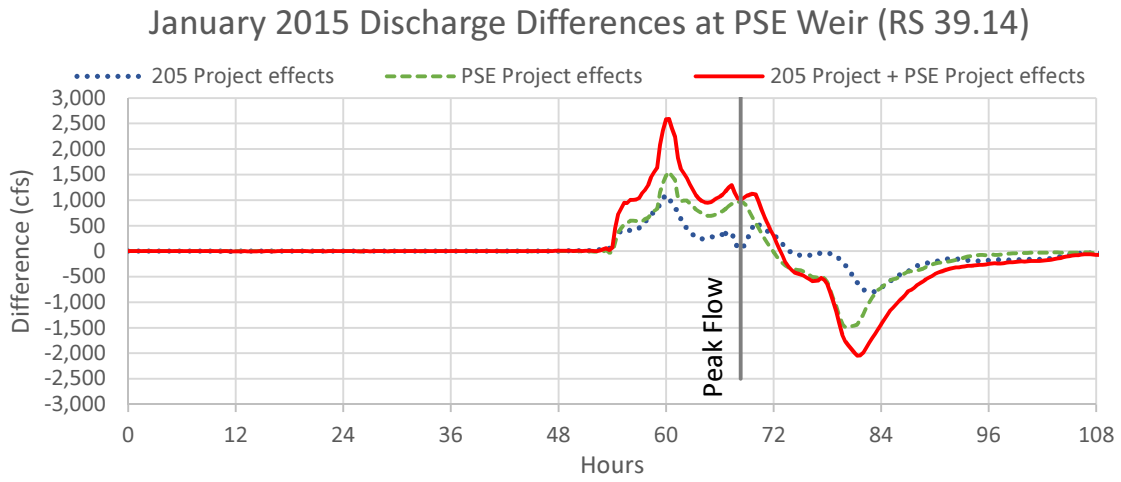

January 2015 Discharge Hydrograph at Snoqualmie
(RS 40.647)
Pre-Projects
Post 205 Project
Post Projects
Peak Flow
Discharge (cfs)
60,000
50,000
40,000
30,000
20,000
10,000
0
0
12
24
36
48
60
72
84
96
108
Hours

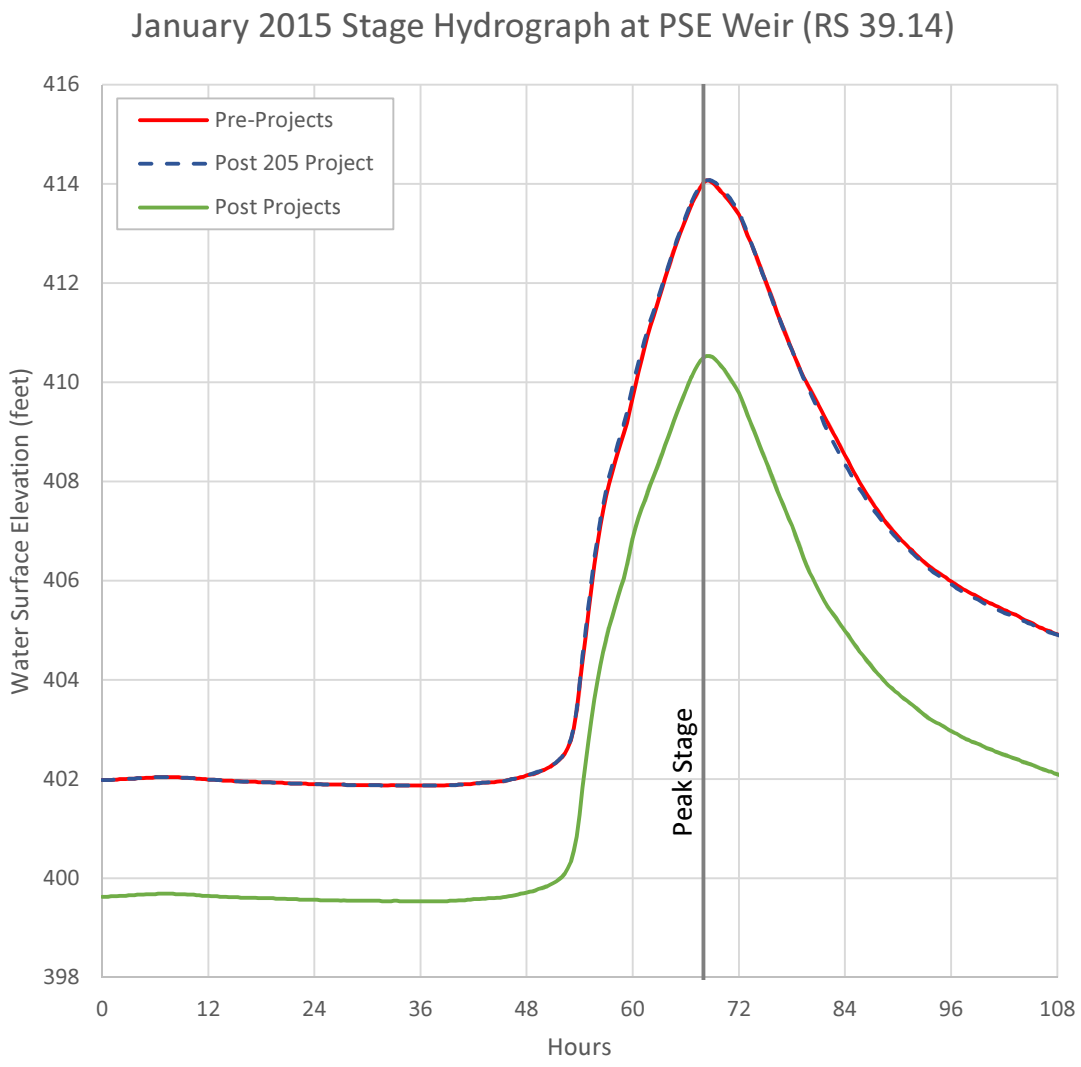

January 2015 Discharge Differences at Snoqualmie
(RS 40.67)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Peak Flow
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
0
12
24
36
48
60
72
84
96
108
Hours

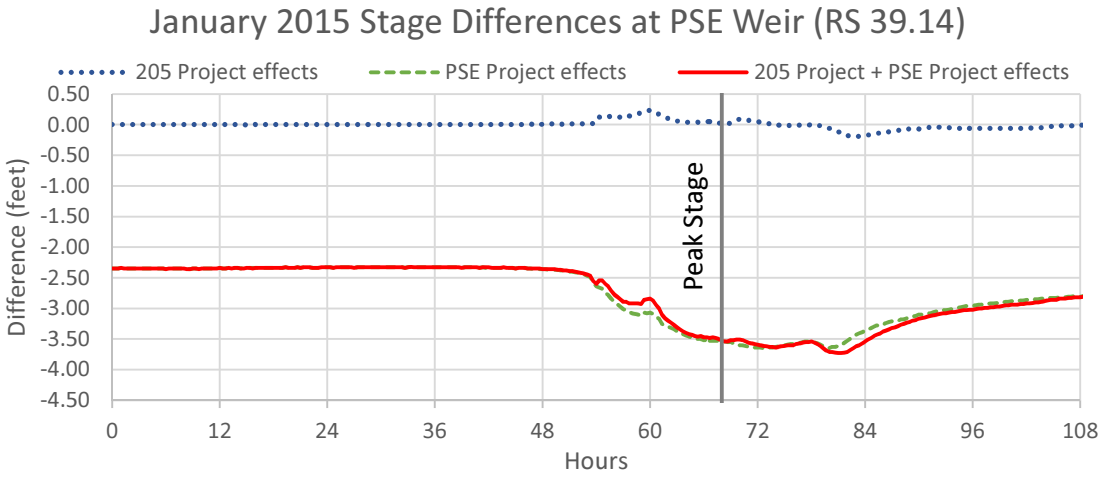

January 2015 Stage Hydrograph at Snoqualmie (RS 40.647)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
424
422
420
418
416
414
412
410
408
406
404
402
400
0
12
24
36
48
60
72
84
96
108
Hours
Peak Stage


January 2015 Stage Differences at Snoqualmie (RS 40.67)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
-3.50
-4.00
-4.50
0
12
24
36
48
60
72
84
96
108
Hours
Peak Stage


January 2015 Discharge Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Discharge (cfs)
60,000
50,000
40,000
30,000
20,000
10,000
0
Peak Flow
0
12
24
36
48
60
72
84
96
108
Hours


January 2015 Discharge Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (cfs)
3,000
2,500
2,000
1,500
1,000
500
0
-500
-1,000
-1,500
-2,000
-2,500
-3,000
Peak Flow
0
12
24
36
48
60
72
84
96
108
Hours


January 2015 Stage Hydrograph at PSE Weir (RS 39.14)
Pre-Projects
Post 205 Project
Post Projects
Water Surface Elevation (feet)
416
414
412
410
408
406
404
402
400
398
0
12
24
36
48
60
72
84
96
108
Hours
Peak Stage


January 2015 Stage Differences at PSE Weir (RS 39.14)
205 Project effects
PSE Project effects
205 Project + PSE Project effects
Difference (feet)
0.50
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
-3.50
-4.00
-4.50
0
12
24
36
48
60
72
84
96
108
Hours
Peak Stage