US Army Corps of Engineers®
Seattle District

SKAGIT COUNTY
Serving With Pride
WASHINGTON

Skagit River Flood Risk Management General Investigation

Skagit County, Washington

Draft Feasibility Report and
Environmental Impact Statement

Appendix B – Hydraulics and Hydrology

May 2014

Hydraulics and Hydrology Appendix

1. Hydraulic Analysis, Final Report, August 2013
2. Hydraulic Technical Documentation, Final Report, August 2013
3. Hydrology Technical Documentation, Final Report, August 2013
4. Sediment Budget and Fluvial Geomorphology, June 2008

U.S. ARMY CORPS OF ENGINEERS
SEATTLE DISTRICT

Downtown Mount Vernon October 2003

Flood fighting October 2003

SKAGIT RIVER BASIN GENERAL INVESTIGATION
FLOOD RISK REDUCTION – HYDRAULIC ANALYSIS

FINAL STUDY REPORT

AUGUST 2013

northwest hydraulic consultants

Skagit River Basin General Investigation
Flood Risk Reduction – Hydraulic Analysis

Final Study Report

Prepared for:

US Army Corps of Engineers
Seattle District
4735 East Marginal Way South
Seattle, WA 98134

Prepared by:

Northwest Hydraulic Consultants Inc.
16300 Christensen Road, Suite 350
Seattle, WA 98188

August 2013

NHC project #200074

Table of Contents

List of Tables

List of Figures

List of Appendices

1 Introduction

This report documents the hydrologic and hydraulic analyses performed by Northwest Hydraulic Consultants Inc. (NHC) in support of the Skagit River Basin General Investigation (Skagit River GI) under contract W912DW-11-D-1006, Task Order No. 3. The work comprised five major technical tasks as follows:

- Hydraulic modeling of the BNSF railroad bridge, which crosses the Skagit River between Mount Vernon and Burlington and which provides an important hydraulic control under extreme flow conditions (Chapter 2).
- Analysis of the effectiveness of a potential increase in early season flood regulation storage at Upper Baker dam on regulated peak flows at Concrete (Chapter 3).
- Analysis of the flood peak discharge reductions achievable from potential new flood regulation storage at Lower Baker Dam (Chapter 4).
- Hydraulic analysis of the lower Skagit River to support economic flood damage analysis under existing conditions, with additional early season flood regulation storage at Upper Baker Dam, and with improved levees (Chapter 5).
- Initial hydraulic design of three flood risk reduction alternatives specified by the Seattle District, including hydraulic analysis in support of economic flood damage analysis for preferred configurations of those alternatives (Chapter 6).

In addition to this study report, NHC also updated existing condition hydrology and hydraulic technical documentation (USACE 2013a and USACE 2013b) to incorporate relevant information from the above technical tasks. The hydrology and hydraulic technical documentation provide extensive background information on hydrologic and hydraulic conditions in the Skagit River basin study area and detailed descriptions of the development of existing condition hydrologic and hydraulic models.

The work conducted under this contract builds on a considerable body of previous hydrologic and hydraulic modeling and analysis performed by the Seattle District and other parties over the past decade and more.

The emphasis of much of this report is on hydraulic modeling for the lower Skagit River basin downstream from Sedro-Woolley. A location map of this area, showing key features referred to in the following chapters of this report, is provided in Figure 1- 1.

1.1 Datum

The vertical datum used for hydraulic modeling in this study, for both the FLO-2D and HEC-RAS models and their output, is NAVD88. The horizontal datum is the Washington State Plane Coordinate System North Zone, 1983/91 North American Datum. All elevations in this document are reported in feet to the NAVD88 datum unless specifically stated otherwise.

1.2 River Stationing

River stationing for the HEC-RAS models used in this study is understood to have originated from the hydraulic model created for a 1984 Flood Insurance Study. It should be noted that the model stationing reported as River Miles (RM), is inconsistent with current measured river lengths. The distance between RM 10.1, just upstream of the North and South Fork split, to RM 22.27, on the downstream side of the Highway 9 Bridge at Sedro-Woolley, is 12.17 miles based on the RM difference. However, the channel distance within the HEC-RAS model between the same two cross sections is 13.25 miles in the 2004 model and 13.42 miles in the updated, geo-referenced 2011 and current (2013) models. The difference in reach distance between the 2004 and 2011/2013 HEC-RAS models is relatively small - around 1,000 feet (1.4%) - and easily explained by slight variations in the channel centerline selected for measurement between the two models. In contrast, the River Mile distance per the model stationing is over a mile less than calculated channel distance in both HEC-RAS models. There are no known major channel shifts, avulsions or meander cutoffs that can explain this discrepancy.

For consistency with previous work, the distributary point of the North and South Forks is set at RM 9.48 and with the exception of water surface profile plots, river miles (RM) in this report refer to the stationing as used in 2004. Cross-section locations (XS) similarly refer to the 2004 stationing.

Water surface profile plots of the system downstream from Sedro-Woolley provided in the report show actual distances as determined from the current (2013) geo-referenced HEC-RAS model. Thus, as an example, the model cross-section with the name RM 22.27 (or XS 22.27 since this is a specific model cross-section location), on the downstream side of the Highway 9 bridge, is the same cross-section in the current work as in prior work. In the profile plots, this cross-section is positioned according to measured channel distances at river mile 23.575. Table 1-1 lists the model cross-section RMs and corresponding actual river miles for the mainstem downstream from Sedro-Wooley and for the North and South forks.

2 BNSF Bridge Hydraulic Modeling

The Burlington Northern Santa Fe Railway (BNSF) bridge is located just east (upstream) of the Interstate 5 and Riverside Drive bridges in Mount Vernon. The BNSF bridge is the most important hydraulic structure in this reach of the river. The bridge has a relatively low deck elevation and a history of entrapping and retaining debris during high flows. A debris jam estimated at about 450 to 500 feet wide by 10 to 20 feet thick at its maximum formed on the bridge in the November 1995 flood[1], providing the basis for debris loading assumptions in recent hydraulic modeling for the Skagit River GI. Previous HEC-RAS hydraulic modeling for the Skagit River GI, reported in the April 2011 draft Hydraulic Technical Documentation (USACE, 2011b), showed head loss through the bridge of the order of three to four feet for 25-year events and larger under the assumed 500 feet wide by 20 feet thick debris blockage. Modeled backwater effects from the BNSF bridge extend upstream to approximately the Highway 9 crossing of the Skagit River at Sedro-Woolley, inducing additional flooding of the left bank Nookachamps basin and resulting in potentially substantial spill from the right bank of the Skagit in the vicinity of Sterling (from approximately RM 21 to RM 22 or roughly 12 to 13 miles upstream from the junction of the North and South Forks). Right bank spill upstream from the BNSF bridge flows north and west across the floodplain and does not re-enter the mainstem Skagit River. Previous modeling showed that spill amounts upstream from the BNSF bridge are quite sensitive to the head loss through the bridge. Large spills upstream from the BNSF bridge have the effect of reducing flows and hence flood risk downstream from the bridge. The hydraulic performance of the bridge is therefore a potentially critical factor in analysis and design of flood management measures and alternatives throughout the lower Skagit River.

The head loss through the BSNF bridge in previous modeling (of the order of three to four feet for 25-year events and larger, as noted above) is significantly larger than observed during recent large floods (November 1990, November 1995, October 2003 and November 2006) raising concerns about the reliability of previous modeling. The purpose of the work described in this report section was: to reexamine the computational approach to modeling the hydraulic performance of the bridge; to perform sensitivity analyses to determine how various model parameters and assumptions influence the computed water surface profile through the bridge opening; and to recommend a computational approach and set of model parameters and assumptions for future modeling. The following factors were considered in the sensitivity analyses:

- discharge rate (from approximately 120,000 to 320,000 cfs)
- debris blockage (from zero to 20,000 square feet)
- HEC-RAS contraction and expansion coefficients
- HEC-RAS right bank station placement
- Steady state vs. unsteady flow modeling

---

[1] The peak discharge during the 1995 flood was 141,000 cfs (about a 25-year return period) at the USGS Skagit River near Mount Vernon gage, located 0.5 miles downstream from the BNSF bridge.

Consideration was also given to scour potential at the bridge. The scour assessment was performed after the sensitivity studies and the order of presentation in this report section reflects the order in which the work proceeded.

Part way through this work, it was determined, from examination of photographs from the November 1995 flood (see Section 2.1), that the existing condition bridge geometry in the HEC-RAS model was incorrect, and had apparently been incorrect for many years. Before completing the sensitivity analyses, additional work was therefore conducted to survey the bridge, update the representation of the bridge geometry in the HEC-RAS model, and reassess the model calibration.

2.1 Bridge Survey and Reassessment of HEC-RAS Model Calibration

2.1.1 Bridge Survey

NHC staff completed a partial survey of the BNSF bridge on 6 November 2012. For access and safety reasons, the survey was restricted to the right bank piers and right bank low chord. Survey grade GPS was used to establish control points from which a survey level was used to determine elevations. A nearby WSDOT monument was surveyed before and after the bridge survey as a quality assurance check. The low chord elevation of the over-water spans of the bridge were subsequently estimated from spot elevations of the bridge deck taken from aerial mapping obtained from the City of Burlington, dated 2009.

Substantial differences exist between the surveyed and previously assumed bridge geometry, as shown in Figure 2-1 and Figure 2-2. The main differences are: 1) the bridge deck is approximately 6.4 feet thick, not 10 feet as previously assumed; 2) the bridge deck has a vertical curve, with the right bank deck about three feet lower than the deck in the main channel area; and 3) the low chord is significantly lower than previously assumed (about six feet lower in the right overbank area and 3 feet lower over the main channel). The lower low chord elevation is of particular importance since it results in the bridge going into pressure flow at a lower discharge than previously assumed. The low chord elevation varies from 43.01 ft. NAVD88 in the right overbank area to 45.51 ft. NAVD88 over the main channel and approximately 47.5 ft. NAVD88 at the Whitmarsh Road underpass on the right bank. Bridge overtopping elevations vary from 49.37 ft. NAVD88 in the right overbank to 51.87 ft. NAVD88 over the main channel.

The results from the level survey were processed, the bridge geometry revised in the HEC-RAS model, and the model calibration re-assessed, prior to sensitivity analysis.

2.1.2 1995 High Water Data and Model Calibration

The error in the bridge geometry in the previous HEC-RAS model was identified from photographs of the November 1995 flood (Figure 2-3 and Figure 2-4). These photographs, taken close to the peak of the flood (reportedly at 12:30 pm on 30 November 1995), show the maximum water level close to the low chord of the bridge and inconsistent with the previous model's representation of the bridge.

After surveying the bridge low chord and piers, the photographs were used to estimate a November 1995 high water elevation on the upstream face of the bridge of 41.66 ft. NAVD88. This high water mark

(HWM “A”) is deemed more reliable than a previously reported high water mark of 42.97 ft. downstream of the bridge (HWM “B”), and was given substantial weight when re-assessing the model calibration. In addition to the new high water mark, the same set of photographs shows that there was no significant debris accumulation *during the flood peak*. Photographs of the large debris jam which developed during this event were evidently taken on the receding limb of the flood, some time after the peak stage. For this reason, the 4,000 square foot debris blockage previously assumed in model calibration to the 1995 flood data was removed.

The changes described above (changing the bridge geometry, removing the debris blockage, and adding a new high water mark) were significant enough to warrant a reassessment of the HEC-RAS model calibration. Figure 2-5 shows 1995 water surface profiles for the previous calibration and for the updated model configuration. This flood did not reach the low chord of the bridge, and changes to the bridge pier geometry were minor. Therefore, the approximately 0.75 foot reduction in backwater seen in Figure 2-5 is due to removal of the debris blockage and changing the “low flow” solution method from momentum to energy (see definitions and discussion of computational approaches in Section 2.2 below). Downstream of the bridge, the difference in model results is minimal. The impact on the model calibration of the change in bridge geometry is minimal because the new, lower deck elevation is still above the peak water level for the 1995 flood, and so free surface flow is maintained.

Overall, calibration to the 1995 high water data was somewhat improved by the above changes. All high water marks discussed here are shown on Figure 2-5. The newly identified high water mark on the upstream face of the BNSF bridge (HWM “A”), determined from the photographs, matches the revised water surface profile reasonably well. The high water mark just below the bridge (HWM “B”), was discounted because it is inconsistent with the photographic evidence and would result in an implausibly large water surface slope between the BNSF bridge and the downstream high water mark (HWM “G”) taken from the USGS stream gage just downstream from the Riverside Drive bridge. HWM “C” is not considered valid and was discounted as an outlier. Two of the remaining three high water marks between the BNSF bridge and Highway 9 (HWMs “D”,”E” and “F”) are better replicated by the revised model. Upstream of Highway 9, the differences between the models dissipate.

The 2003 and 2006 floods were also re-run with the new bridge geometry and energy solution method. For both of these floods, simulated water surface elevations were reduced around 0.2 feet in the Nookachamps area, slightly improving the calibration in this area. As neither of these floods was modeled with a debris load and as neither flood reached the low chord of the bridge, these changes are attributed to a change in solution method. The minor change in the bridge geometry to reflect the pilings driven around the pier that failed in the 1995 flood is unlikely to affect the computed water surface profile.

2.2 Investigation of HEC-RAS Bridge Computational Methods

The original scope of work called for investigating all bridge modeling approaches available in HEC-RAS as part of the sensitivity testing. HEC-RAS allows the use of different computational methods for “low” and “high” flow at bridges. “Low” flow is defined as flow under the bridge with water surface elevations not reaching the bridge low chord. For most significant bridges, this would include all but the largest flood discharges. “High” flow is when water surface elevations result in pressure flow under the bridge and potentially additional weir flow over the deck. For low flows, HEC-RAS modeling options are:

- energy balance
- momentum balance
- Yarnell method
- WSPRO Method

For high flows (pressure flow under the bridge, and weir flow over the bridge), options are:

- energy balance
- pressure/weir method

In addition, HEC-RAS allows the option of converting all bridges in a model to lidded cross sections.

In the course of this work, problems were encountered in application of most of the modeling options. Converting bridges to lidded cross sections was determined not to be an option for this particular application because debris blockage is not accounted for in the conversion process.

For low flow methods, efforts to use the WSPRO method were unsuccessful; the model crashed when using this option. The momentum method gave numerous warnings regarding invalid solutions, although results were still reported. The bridge modeling situation was discussed with Dr. Gary Brunner at the USACE Hydrologic Engineering Center. His opinion was that the debris blockages being modeled exceeded the range for which the Yarnell and momentum methods were appropriate and recommended use of the energy method. He also noted that placing all debris in a single block such that it covered multiple adjacent piers, as in previous modeling, would result in incorrect results. Debris geometries were therefore modified to use multiple debris blockages sized to ensure no overlap with adjacent piers or debris. The energy method was tested over the full range of debris blockages (0 to 20,000 square feet) for low flow conditions and was found to give apparently reasonable results without error or caution notes. Therefore this method was used for all subsequent low flow sensitivity testing.

Under high flow conditions, the energy method resulted in numerous errors and cautions. The pressure/weir flow method produced somewhat higher headwater results, but did not exhibit the same computational issues. High flow modeling methods were also discussed with Dr. Brunner and he concurred that the pressure/weir flow method should be used. All sensitivity testing discussed herein uses this method for high flows. It was determined that the most appropriate trigger elevation to use for pressure/weir flow calculations was the main bridge span low chord elevation of 45.5 ft. NAVD88, as opposed to the highest low chord elevation which occurs on the span crossing Whitmarsh Road near the right bank (Station 1000,Figure 2-2).

2.3 Sensitivity Analysis

Sensitivity analyses were conducted on a number of key model variables. For comparison, a "base" case was selected. This consisted of 10,000 square feet of debris blockage, the right bank station set at 727 feet (i.e. at the edge of the low flow channel – see Figure 2-2), and contraction/expansion coefficients of 0.1 and 0.3 respectively. The amount of debris blockage, right bank station and contraction/expansion coefficients were then varied systematically to explore the sensitivity of the bridge to the various parameters. In all cases, only one variable was changed per run. The base case right bank station and contraction/expansion coefficients were as used in the model calibration.

The existing condition HEC-RAS model was modified for the sensitivity analysis in order to allow evaluation of BNSF bridge performance under extremely high flows. The right bank levees between Sedro-Woolley and the bridge were removed to prevent overtopping flows from leaving the model domain upstream from the bridge. The left bank Nookachamps storage areas were also disconnected in order to improve model speed and stability at high flows. Levees downstream of the bridge were left at their current (existing condition) height. As a result of the miles of overtopping levee downstream from the bridge, tailwater elevations are very similar over a large range of high flows. Minor modification to the levee geometry at the bifurcation of the North and South Forks was also required in order to allow the model to run in HEC-RAS Version 4.1 (previous analyses used Version 4.0).

The 500-year average regulation condition flood from the March 2011 draft Hydrology Technical Documentation (USACE, 2011a) was run for each scenario in order to obtain results over a wide range of flow, including the transition from low flow to high flow hydraulics. Results are presented as ratings curves, selected water surface profiles, and in tabular form as described below. It should be noted that in the rating curve plots, a small hysteresis loop is evident in all runs. The water surface profile plots use nominal flow rates for each profile. Because the model was run in unsteady mode, flows between runs were never exactly the same for a given time step; therefore the water surface profile figures show results for flows that are within a few of percent of each other but not equal. This causes slight variations in results (including tail water elevations), but the dominant variation by far in each comparison is due to the variable being tested. Model results for each group of sensitivity runs are discussed in Sections 2.3.1 through 2.3.4 below. Table 2-1 gives detailed hydraulic output results at the bridge for all sensitivity runs over the full range of flows.

Interpretation of data in Table 2-1 requires some care. In particular, it will be noted that the data show some significant variations in the elevation above which pressure/weir flow calculations are used. Several factors appear to affect the apparent switch to pressure flow as reported in Table 2-1:

i) The model will default to energy calculations if a valid pressure/weir solution cannot be found – this appears to be occurring at the transition to pressure flow.

ii) The upstream water level reported in Table 2-1 is from the cross-section immediately upstream from the bridge, whereas the trigger for switching to pressure flow is at a cross-section internal to the bridge.

iii) Model output is reported at an hourly time step so reporting of the change from energy to pressure flow may be up to one hour later than actual.

2.3.1 Model Sensitivity to Debris Blockage

The BNSF bridge has demonstrated a propensity for spurring the formation of debris jams during high flows. The debris jams do not occur during every large flood, however, and, as shown in the 1995 event, the bridge may remain clear of debris during the peak flow but trap debris later in the event. Because a debris jam could potentially influence discharge rates and water levels upstream and downstream of the bridge, a sensitivity analysis of various debris blockages was conducted. The debris jam sizes considered were: 0, 3000, 6000, 8000, 10000, 14000, and 20000 square feet. Debris was distributed over a number of piers, and the areas quoted above are in addition to the blockage due to the piers themselves. To the extent possible, debris was placed to avoid encroachment on that portion of the main channel between the left bank and Pier 1 (piers are numbered from left to right looking downstream) consistent with past observations. However, because of their size, this was not possible with the 14,000 and 20,000 square foot blockages. The placement of debris for the various size blockages is shown in Figure 2-6 through Figure 2-11.

Figure 2-12 shows the tail water rating downstream from the bridge (“DS RC”) and rating curves immediately upstream of the bridge (“US RC”) with the various blockage configurations and for flows ranging from 30,000 to roughly 320,000 cfs. The tail water rating follows the familiar convex curve. The break in slope and flattening of the tail water rating at a flow of about 175,000 cfs corresponds to the overtopping of the levee system downstream from the bridge. With no debris, the upstream rating closely follows the tail water rating up to a flow of about 220,000 cfs. Above that point, the transition to pressure flow results in an increase in head loss through the bridge opening and a divergence of the upstream and downstream ratings. As debris is added and as the degree of blockage increases, a more severe “step” forms in the upstream rating curves as the transition to pressure flow occurs at lower and lower flows. Debris blockages from 6,000 to 14,000 square feet trigger pressure flow conditions at discharges in the 150,000 to 170,000 cfs range. Figure 2-13, Figure 2-14 and Figure 2-15 show water surface profiles with the various debris blockages at flows of approximately 150,000 cfs, 200,000 cfs and 250,000 cfs respectively.

2.3.2 Model Sensitivity to Contraction and Expansion Coefficients

In unsteady flow models, HEC-RAS develops families of rating curves for each bridge that represent the full range of flows and headwater stage under various tail water elevations. The curves are calculated based on the bridge modeling method chosen, each of which has key parameters that may be varied. For energy method calculations, the contraction and expansion coefficients are multiplied by the change in velocity head between sections to estimates losses. For bridges with large changes in velocity due to contracted openings, the model solution can be quite sensitive to these coefficients. To test the sensitivity to these parameters, simulations were conducted with three sets of coefficients for contraction and expansion: 0.1/0.3 (the base condition), 0.3/0.5, and 0.5/0.7. The downstream tail water rating curve (“DS RC”) and upstream rating curves (“US RC”) from these simulations are shown in Figure 2-16. Water surface profiles at selected discharges are shown in Figure 2-17.

As can be seen in the rating curves (Figure 2-16), for flows up to about 140,000 cfs, altering the coefficients has an approximately linear impact on the upstream rating curves (i.e., the difference between ratings for coefficients of 0.1/0.3 and 0.3/0.5 is about the same as the difference between ratings for coefficients of 0.3/0.5 and 0.5/0.7), along with the expected result that higher coefficients lead to greater head loss and less efficient conveyance. Above about 170,000 cfs, the bridge transitions fully to pressure flow and a HEC-RAS computational approach which does not make use of contraction/ expansion coefficients. Hence the solutions converge, as can be seen in both Figure 2-16 and Figure 2-17.

As noted previously, the set of contraction/expansion coefficients used in the model calibration was 0.1/0.3.

2.3.3 Model Sensitivity to Right Bank Station

Simulations were conducted to assess the impact of the placement of the right bank station immediately upstream and downstream of the BNSF bridge. The right bank in this area is a flat low lying field (see Figure 2-2) which floods at a flow of roughly 50,000 cfs. The area typically has a healthy grass cover but may be covered by sand, which deposits preferentially in this area during floods. The bank station represents the transition between channel and overbank areas, and the choice is a somewhat subjective matter in this case. HEC-RAS uses the bank station as a change in roughness location, as well as a partitioning tool when dividing the cross-section into sections for computation. Two locations for the bank station were tested: the existing location near the edge of the low-flow channel, and at the right edge of the cross-section, which is approximately the edge of water during extremely high flows. The scenario with the bank station placed at the right edge of the cross-section has a channel n-value (0.034) extended across the floodplain to the bank station.

Figure 2-18 shows the downstream tailwater rating curve ("DS RC") and upstream rating curves ("US RC") for these two scenarios and Figure 2-19 shows corresponding water surface profiles for flows of 150,000, 200,000 and 250,000 cfs. Results from the two scenarios are almost indistinguishable. This is in large part because under the assumed 10,000 square foot debris load, flow across much of the right bank area in question is blocked by debris. Greater differences would be expected under lower debris loads.

2.3.4 Steady State vs. Unsteady Flow Modeling

Simulations were performed in steady state mode for flows of 150,000, 200,000 and 250,000 cfs for each of the sensitivity scenarios described in Sections 2.3.1 through 2.3.3 above, and water surface profiles were then compared against the corresponding results from the unsteady flow runs. The results of the steady state simulations and the corresponding unsteady flow simulations were essentially identical. Because the results of the steady and corresponding unsteady flow simulations are so close, comparison plots are not included in the report.

2.4 Effects of Bed Scour on Bridge Hydraulics

All simulations described above assume a fixed channel bed, but a brief review of hydraulic outputs and available sediment data indicates that it is likely that significant scour takes place at the bridge under flood flow conditions. The failure of Pier 8 of the bridge due to scour in the 1995 flood provides additional evidence supporting this hypothesis.

Sediment sampling of the entire lower Skagit River system was undertaken as part of a geomorphology task for the Skagit GI study in 2002 (Cherry and Jackson, 2002). Multiple grab samples and full transect bed material samples were obtained at or around the BNSF bridge. The study results indicate that the bridge is located within the gravel-sand transition of the Skagit River. The report states that the mean bulk sample D_{50} was 5.4 mm upstream of the bridge and 0.6 mm downstream. Bed material samples from this location and further upstream indicate a finer gradation than the bulk surface samples; a D_{50} of less than 1 mm is reported. Recent work by the USGS (Curran, et al., 2009) sampling at the next bridge downstream confirms the sand bed nature of the channel below the BNSF bridge.

Scour potential was investigated for a no-debris and 10,000 square foot debris (base case) scenario. Approach velocities to the bridge are in the range of 6 to 9 feet/second for flows from 150,000 cfs to 250,000 cfs under these scenarios (Figure 2-20). This range of flows is considered to include the range of greatest interest for analysis of the various potential flood management alternatives. In the bridge opening, velocities increase slightly under the no-debris condition to values in the range of from 7 to 11 feet/second, while with debris, the velocity increases to a maximum of about 16 feet/second (Figure 2-21). Estimates of potential scour, due to general and contraction scour only, were generated using the hydraulic design tools in HEC-RAS and some external references. Local abutment and pier scour were not evaluated.

Results using the contraction scour tool in HEC-RAS for the main channel only are presented in Table 2-2 for a flow 150,000 cfs. A conservative D_{50} of 10mm was used (this is the single largest bulk sample value from the vicinity of the bridge) and scour was forced to be live bed. The estimated scoured area was calculated by multiplying the scour depth by the wetted perimeter of main channel (excluding piers) in the cross section.

The analysis has a few notes of interest:

i) Scour is predicted to occur even with no debris on the piers.

ii) Scour area is 70% of debris blockage area. (Note this is at a relatively low flood flow of 150,000 cfs; the bridge is not in pressure flow and velocities are at their minimum [Figure 2-21]).

iii) No right overbank scour is calculated, but the pier failure on this overbank in 1995 is evidence that if flows are sufficient to strip away the vegetative cover, significant scour would also be expected here. (Note, however, that we have no information on the nature or condition of the pier foundation.)

The scour calculations indicate that most of the waterway area reduction from the debris blockage is likely to be compensated by scour of the bed. Not accounted for in these calculations are areas that may be resistant to scour, either from natural bedrock outcroppings or riprap placed over time by the railroad. It is known that the piers in the main channel are protected by riprap (see the 1993 low water photograph in Figure 2-22). Nevertheless, unless the entire channel is armored under the bridge it seems likely that extensive bed scour will occur under flood conditions.

As stated as the beginning of this section, all simulations performed in this work assumed a fixed channel bed. No modeling was performed for the “with scour” condition.

2.5 Further Refinements to HEC-RAS Model Representation of BNSF Bridge

At the conclusion of the sensitivity runs, the following additional refinements were made to the model representation of the BNSF bridge:

i) Skew of approximately 10° was applied to the bridge and the cross-sections immediately upstream and downstream. The correction for skew results in a slight reduction in the effective channel width.

ii) The pier spacing was adjusted to more closely reflect actual spacing based on measurements from aerial photographs.

iii) The shapes of piers 4 through 12 (piers are numbered from left to right looking downstream) were modified (tapered) to more closely reflect the actual pier shapes. Piers 1 through 3 were already tapered in the model.

The final bridge geometry is shown in Figure 2-23. By comparison with Figure 2-2, it can be seen that the principal changes are in the spacing of the main channel piers 1 through 3, an increase in the effective pier widths as a result of the skew adjustment, and a slight reduction in channel width. The impact of these refinements on bridge hydraulics is illustrated in Figure 2-24, Figure 2-25 and Figure 2-26 which show rating curves with the changes (“skewed bridge”) and for the original sensitivity runs (“before skew adjustment”) for scenarios with no debris and with 3,000 and 6,000 square feet of debris. All runs assumed 0.1/0.3 contraction/expansion coefficients and the right bank station at the edge of the low flow channel. The changes (primarily the skew adjustment) result in the bridge transitioning to pressure/weir flow at a somewhat lower discharge and a slightly higher stage for a given discharge. These changes do not affect the conclusions and recommendations presented in Section 2.6 based on the sensitivity runs and assessment of scour potential.

2.6 Conclusions and Recommendations

Debris accumulation at the BNSF bridge is highly variable both from flood to flood and within individual flood events. The largest documented blockage in the recent past formed during the flood of November 1995. This event had a peak flow of 141,000 cfs at Mt. Vernon for a return period of approximately 25-years.

Photographs taken during the 1995 flood indicate that the bridge was clear of debris at the time of the peak flow and that the debris jam (subsequently estimated as having maximum dimensions of approximately 450 to 500 feet wide by 10 to 20 feet deep) formed over a relatively short period of time on the receding limb of the flood hydrograph. We speculate that the jam initially formed as a raft of debris lodging on the bridge piers and then trapping other debris moving down river. There is nothing to indicate that the debris jam could not have formed earlier in the event and been in place at the time of the peak discharge. Selection of parameters to model the hydraulic performance of the BNSF bridge should therefore consider scenarios with and without debris blockage.

As shown in the 1995 flood, the BNSF bridge is capable of collecting and building impressive debris jams in a short amount of time. Long term trends will likely increase both the total volume and individual log sizes in the flood-borne debris load. This is due to projected increases in peak flows and hence channel migration associated with climate change, and as the numerous restoration projects on the Skagit River banks mature and begin to provide increasingly large conifers to the river. Debris accumulation on the bridge is a very real risk, however extrapolation of debris loads to extreme flood conditions is a speculative endeavor.

Balancing the impacts on bridge hydraulics of debris accumulation is the expectation that the river bed in the vicinity of the bridge is highly mobile under flood conditions and can be expected to adjust to debris blockage through scour. Analysis of scour potential (Section 2.4) for a scenario with a flow of 150,000 cfs and a debris blockage of 10,000 square feet, resulted in a scour area of approximately 7,000 square feet, or 70% of the debris blockage area. Scour depth and area is expected to increase as both blockage size and discharge increase.

Since the HEC-RAS model is not capable of simulating a mobile bed with the unsteady flow computations (HEC-RAS does have sediment transport modeling capability however this is for "quasi-unsteady" mode and is typically used for estimating long term trends), the effects of scour in scenarios with debris blockage can be most readily accounted for by reducing the assumed blockage area by the estimated scour area. Based on the analysis of scour potential, for example, the hydraulic performance with a 10,000 square foot blockage could be modeled using a net 3,000 square foot blockage, assuming 7,000 square feet of scour area.

Ratings upstream and downstream from the bridge with no blockage and with a 3,000 square foot blockage are shown in Figure 2-25 for the simulations with and without skew adjustment. The impacts of a 3,000 square foot blockage on the upstream rating are insignificant until the flow reaches about 190,000 cfs without skew adjustment and roughly 175,000 cfs with skew adjustment, above which the ratings with and without debris start to diverge. Note that the flattening in the downstream rating at flows above 175,000 cfs is the result of overtopping of levees downstream from the bridge. The downstream levees were kept at their existing height for the purposes of this analysis.

Increasing the blockage area by 50% to 15,000 square feet and continuing to assume a scour area of 70% of the blockage, would result in a net blockage for modeling purposes (i.e. after accounting for scour) of 4,500 square feet. Interpolating from the suite of ratings in Figure 2-12 shows that the impacts of this blockage are minor until the flow reaches about 180,000 cfs (roughly 170,000 cfs with skew adjustment), above which the ratings with and without debris again start to diverge.

If scour offsets the effects of debris blockage to the extent estimated here, then it appears that the hydraulic performance of the BNSF bridge would be relatively insensitive to debris load over a wide range of blockage sizes for flows up to at least 160,000 cfs. Previous hydraulic modeling without debris loads shows that flows much greater than this magnitude are unlikely at the BNSF bridge under existing conditions because of spill over the upstream Dike District 12 (DD12) levees and from the right bank of the Skagit in the vicinity of Sterling (RM 21 to RM 22 or from 3.5 to 4.5 miles upstream from the BNSF bridge). Measures which would allow passage of flows on the order of 200,000 cfs and greater would include raising upstream and downstream levees and construction of a right bank levee at Sterling. Raising the downstream levees would change the downstream bridge rating and affect the bridge hydraulic performance as characterized in this report for flows greater than about 175,000 cfs.

Given the various uncertainties in the size of debris blockages, potential scour depths, and the nature of future flood management measures we recommend adoption of a fixed design debris blockage of 3,000 square feet for current feasibility studies[2]. For the flow range of greatest interest, this produces upstream water levels only slightly higher than scenarios without debris. Recognizing the very limited scour analysis undertaken here and the current lack of detailed information on bed conditions at the bridge (e.g., while the bridge piers are known to have riprap protection, there is no detailed information on the size or extent of existing scour protection), this assumption should only be used for feasibility study purposes and should be revisited before more detailed design is undertaken.

With regard to other hydraulic model parameters, we recommend that the contraction/expansion coefficients remain set at 0.1/0.3 as in model calibration, and that the model's right bank location remain at the edge of the low flow channel also as in model calibration. In both cases, we see no strong justification for departing from the calibration values. Further, model results are insensitive to the right bank station location.

Finally, we recommend that future bridge modeling for this study use the energy approach for low flows and pressure/weir flow for high flows. These methods were found to be robust and to produce plausible results for the full range of flows and blockage conditions examined.

[2] Following review by the Seattle District, a debris blockage of 6,000 square feet was adopted for subsequent hydraulic modeling purposes.

3 Early Season Flood Regulation

Under existing conditions, flood flows on the Skagit River are regulated by flood control operations at Upper Baker and Ross dams. The flood control storage provided at Upper Baker and Ross, as required under the existing project FERC licenses, varies seasonally as shown in Table 3-1.

Note in Table 3-1 that the flood control storage required at Upper Baker Dam under the existing FERC license is slightly different from that described in the Baker River Project Water Control Manual (WCM) (USACE, 2000). The existing condition analyses described in this report section assume flood control storage requirements per the FERC license, as discussed further in Section 3.1.

Hydrologic analyses of existing condition regulated flows described in the August 2004 draft Hydrology Technical Documentation for the Skagit River GI (USACE, 2004b) ignored the seasonal variation of flood control storage and assumed that the required maximum amount of storage (74,000 acre-feet at Upper Baker and 120,000 acre-feet at Ross) would be available for all floods, regardless of the date of occurrence. As shown in Table 3-1, the full amount of flood storage is not required at Upper Baker until November 15 and at Ross until December 1.

The work described here evaluates:

- the impact of existing early season flood control storage requirements on regulated peak flows on the Skagit River near Concrete (i.e. downstream from the Baker River confluence), and
- the effectiveness of increased early season flood control storage at Upper Baker Dam, with the existing early season flood control storage at Ross Dam, for the optional flood control storage requirements summarized in Table 3-2.

As can be seen from Table 3-2, under the option examined here, the full flood control storage requirement at Upper Baker (74,000 acre-feet) would be provided by October 15 as opposed to November 15 under existing flood control operations.

The analyses presented here were performed using unregulated tributary inflows to the Skagit and Baker Rivers dated 13 January 2011, originally provided in digital form with the March 2011 draft Hydrology Technical Documentation (USACE, 2011a) in file: GI_Flows_Revised2_BestWorst.dss. Unregulated peak flows for the Skagit River near Concrete are provided for comparison with regulated peak flows in Table 3-7, Section 3.3.

The evaluation consisted of three principal tasks as follows:

- analysis of the historic daily record of reservoir storage for Upper Baker and Ross (Section 3.1),
- analysis of the impacts of existing flood control storage requirements on regulated peak flows (Section 3.2), and
- analysis of the impacts of increased early season flood control storage at Upper Baker Dam on regulated peak flows (Section 3.3).

3.1 Reservoir Record Analysis

Daily time series of reservoir elevations for Upper Baker and Ross were obtained from Puget Sound Energy (via Skagit County), the USGS, and USACE. For Upper Baker, gaps in the USGS daily data were filled with the Puget Sound Energy data to create a continuous record for water years 1977 through 2009. For Ross, the USGS daily data were filled with data from the USACE to create a continuous record for water years 1962 through 2009. The reservoir elevation time series were converted to time series of reservoir storage using elevation/storage data provided in the project WCMs.

It is recognized that the period of historic reservoir elevation or storage data obtained for this work (1977 through 2009 at Upper Baker, and 1962 through 2009 at Ross) may not be representative of future project operations. Accordingly, discussions were held with representatives from both Puget Sound Energy (PSE) and Seattle City Light (SCL) to determine what period of historic reservoir elevation or storage data is expected to be most representative of future conditions, especially in the early part of the flood control season.

3.1.1 Upper Baker

According to representatives from PSE, prior to 1984, flood control operations at Upper Baker provided 16,000 acre-feet of storage on 1 November and 74,000 acre-feet on 15 November, with more of a "stair-step" change in flood control storage between those two dates than at present. Since 1984, project operations have assumed a linear transition in the storage required between those two dates, hence providing more assured flood control early in the flood control season.

Operations at Upper Baker have also deviated from expected future operations since 2004. In accordance with the requirements of the FERC relicensing agreement, an Interim Protection Plan (IPP) was introduced in 2004 to improve fish habitat in the Baker River by reducing rapid fluctuations in flow. Under IPP-related project operations, more storage than required is generally available in the Baker River project early in the flood control season. IPP operations are expected to continue until approximately 2013, when new turbine units to be installed at the project will be fully operational.

Under the terms of Article 107c of the FERC license issued in October 2008, PSE is required to "develop means and operational changes to operate the Project reservoirs in a manner addressing imminent flood events." These changes may include "additional reservoir drawdown below the maximum established flood pool." It is anticipated that any operational changes to address "imminent floods" would take place after 2012; the nature and impact of any such changes is not yet known.

A further change affecting flood control performance has been the implementation by PSE of flood control pool buffers at both Upper Baker and Lower Baker since about 2006. The buffers provide additional storage above that required for flood control operations per the operating license. At Upper Baker, this additional storage is 26,000 acre-feet, so that the bottom of the buffer is approximately 7 feet below the maximum permissible pool elevation in the flood control season. At Lower Baker, the bottom of the buffer is approximately 5 feet below the spillway crest elevation, representing approximately 9,850 acre-feet of storage below the spillway crest. The purpose of the buffers is to provide PSE with operational flexibility while avoiding, to the extent possible, incursion into the formal flood control storage space at Upper Baker. PSE operates the reservoirs to try to maintain water levels toward the low end of these buffers (water levels are generally maintained 2 to 3 feet above the bottom of the buffer), however there is no formal operating policy for the buffers. It should also be noted that the USACE only manages flood control space at the Upper Baker project.

It was noted in the course of discussion with PSE staff that the flood control storage requirements at Upper Baker as described in the WCM differ slightly from the storage required per the project's FERC license. Under the FERC license, which PSE views as the controlling document, 16,000 acre-feet of storage is required at Upper Baker between 15 October and 1 November. Under the current WCM, flood control storage would be increased from 0 acre-feet on 1 October to 16,000 acre-feet on 1 November. Comment from the USACE (e-mail from Dan Johnson dated 7 June 2010) confirms that PSE will be required to provide 16,000 acre-feet of storage in Upper Baker by 15 October per the current FERC license.

While future operations at Upper Baker are expected to differ from past operations in a number of respects, for current purposes it is assumed that future operations will be most similar to operations in the 20-year period 1984-2003.

3.1.2 Ross

The situation at Ross is less clear than at Upper Baker. As discussed later in this section, Ross Reservoir often provides significantly greater storage early in the flood control season than is required under the terms of its operating license. According to a representative from SCL, Ross reservoir elevations in the early fall are driven by a combination of factors including summer/fall weather conditions, energy demand, fisheries compliance requirements, and conditions in the energy market in general. SCL stressed that while no significant changes in operational practices were anticipated in the foreseeable future, there was also no guarantee that early flood control season storage at Ross would be greater than required in the future. Considering trends in energy demand, SCL suggested that reservoir data from the period 1990 through present would be more indicative of future operations than data from earlier periods.

3.1.3 Analysis of Reservoir Elevation and Storage Data

Data for the periods 1984-2003 at Upper Baker and 1990-2009 at Ross were analyzed to produce summary "hydrographs" and duration curves of reservoir elevation and available storage. Summary hydrographs are provided in Figure 3-1 through Figure 3-4, while duration curves are provided in Figure 3-5 through Figure 3-8.

The summary hydrographs (Figure 3-1 through Figure 3-4) show percentiles of stage or available volume on a given day of the year, as well as the existing and, for Upper Baker, optional flood storage requirements from Table 3-1 and Table 3-2. The Upper Baker plots (Figure 3-1 and Figure 3-2) show that from October 1 to November 15 the median available flood storage is much less than the full 74,000 acre-feet required under existing regulation only after November 15. While this is consistent with the requirements of the 2000 Baker WCM, it demonstrates that it is inappropriate to assume that full flood control storage is available for all floods regardless of their date of occurrence. The plots for Ross (Figure 3-3 and Figure 3-4) show that for most of October, the median available flood storage is close to or exceeds the full 120,000 acre-feet required after December 1. The plots for Ross also show that in many years, the storage available greatly exceeds the flood control requirements. Note in Figure 3-1 through Figure 3-4 that encroachments into the existing flood control pool are indicative of historic flood control operations.

Duration curves (Figure 3-5 through Figure 3-8) were developed for two-week periods in October and November, as well as for the balance of the flood control season from December through February. The duration curves show that in early October, the full flood storage has historically only been provided about 10% of the time at Upper Baker (Figure 3-5 and Figure 3-6) and 45% of the time at Ross (Figure 3-7 and Figure 3-8). After December 1, the full flood storage has historically been available over 90% of the time at both projects. While these data show that project operations are consistent with the respective WCMs, the duration curves again serve to demonstrate that it is inappropriate to assume that the full amount of flood control storage is available early in the flood control season under the existing regulation.

3.2 Impact of Existing Flood Storage Requirements on Regulated Peak Flows

The impact of early season flood storage on regulated peak flows was analyzed using a spreadsheet routing "model" of the Skagit and Baker River projects originally developed by the Seattle District USACE and modified by NHC for previous investigations of flood control operations under contract to Skagit County. The spreadsheet model allows the user to route flows through the Upper Baker and Ross reservoirs according to the flood control regulations described in the 2001 Skagit River Project Water Control Manual (USACE,2001), then downstream to the USGS gage on the Skagit River near Concrete (USGS gage 12194000). The flood control regulations assume that outflow at both projects will be restricted before the **unregulated** flow at Concrete reaches the flood damage threshold of 90,000 cfs. Upper Baker releases are assumed to be set to the minimum of 5,000 cfs three hours before the 90,000 cfs threshold flow is reached at Concrete, while Ross releases are assumed to be set to 5,000 cfs eight hours before the threshold flow is reached at Concrete. These releases are maintained until reservoir levels rise to a point which triggers greater releases as specified under the respective Spillway Gate Regulation Schedules (SGRSs). Channel routing from the project reservoirs downstream to Concrete is accomplished in the spreadsheet using a simple lag model. Interpretation of the requirements of the WCM was facilitated through discussion with staff of the Water Management Section of the Seattle District.

The original spreadsheet model provided by the USACE was modified in previous work as follows:

i) The computational procedures used to represent the Upper Baker Dam SGRS was simplified.

ii) Relevant portions of the Ross Dam SGRS not included in the original spreadsheet were added.

In addition, the work described here used updated tributary inflow hydrographs from the most recent (January 2011) hydrologic analysis for the Skagit River basin.

The modified spreadsheet model was used to route winter (October – March) flood hydrographs with return periods of 5-, 10-, 25-, 50-, 75-, 100-, 250-, and 500-years through the Upper Baker and Ross reservoirs downstream to the USGS Skagit River near Concrete gage. To investigate the effects of existing flood regulation, flood routing was performed with starting reservoir storage conditions on October 1, October 15, November 1, November 15, and December 1 per the **existing** flood storage requirements provided in Table 3-1. Recognizing the limitations of the channel routing component of the spreadsheet model and to ensure consistency with previous work, flood hydrographs output by the spreadsheet model for the Baker River below Lower Baker Dam and the Skagit River at Marblemount (above the confluence with Cascade River) were input to an existing upper basin HEC-RAS model, described in the study Hydraulic Technical Documentation (USACE, 2011b), and re-routed to Concrete to produce final regulated flood hydrographs at Concrete. The existing condition regulated peak discharges from the analyses for the Skagit River near Concrete are summarized in Table 3-3, along with the approximate contribution to the regulated peak discharge from Upper Baker and Ross. Also shown in the bottom two rows of Table 3-3 are references to peak discharges for seasonally weighted hydrographs discussed below.

Note from Table 3-3 that the regulated peak discharge for floods occurring on October 1 with no flood storage available may be up to 24% higher than floods occurring on or after December 1, when the maximum required amount of flood storage is available at both Upper Baker and Ross.

Note also from Table 3-3 that the 2-year peak discharge for the Skagit River near Concrete is less than the 90,000 cfs threshold which triggers flood control operations. Hence no reservoir routing was conducted for this event.

The analysis described above shows, for example, that a 100-year winter flood event occurring on 1 October would result in a regulated peak discharge on the Skagit River near Concrete approximately 24% higher than for similar events occurring after December 1, when the full amount of flood control storage is available at both Ross and Upper Baker. However, to gain insight into the effect of reduced flood storage on flood risk, the probability of damaging floods occurring early in the flood season also has to be considered.

Ideally for this type of analysis, one would determine unregulated flood hydrographs for each return interval of interest for defined periods, such as two-week windows, throughout the flood season, and then route those flows to produced regulated flows for each two-week period. However, the unregulated flood hydrographs available are based on analysis of annual maximum winter (i.e. October through March) flows only; more detailed analyses of unregulated flows by month or by two-week window are not available.

In the absence of more detailed information, assessment of risk was based on a simple analysis of the temporal distribution of annual maximum winter flows within the October through March flood control season. Examination of the reconstructed record of unregulated 1-day winter peak flows for the Skagit

River near Concrete shows that 42% of winter floods occur prior to 1 December. The seasonal distribution of unregulated 1-day peak flows by two-week period is illustrated in Figure 3-9 and tabulated in Table 3-4. The one-day maximum winter discharges for the period of record are also plotted against time of occurrence in Figure 3-10. The record used for this analysis includes four historic events (water years 1898, 1910, 1918 and 1922) and the systematic record from water years 1925 through 2007, for a total of 83 events.

The impact of the seasonal variation of flood storage on regulated flood hydrographs for a specific return period was then determined by simply weighting the existing condition regulated hydrographs for each analysis date through the flood control season (i.e. October 1, October 15, November 1, November 15, and December 1) on the basis of the historic frequency of occurrence of annual maximum winter flows within each of the two-week periods shown in Table 3-4. The weights applied, given in Table 3-5, imply averaging the regulated hydrographs at the start and end of each two week period, and then weighting those average hydrographs by the historic frequency of occurrence of floods in each two-week period.

The existing condition peak discharges for the weighted hydrographs are summarized in Table 3-3, and samples of the October 1, December 1, and weighted regulated hydrographs for 25-year and 100-year events are provided in Figure 3-11 and Figure 3-12. A complete set of regulated hydrographs is available in digital format.

From Table 3-3, it can be seen than that peak discharges for the existing condition weighted hydrographs, considering the seasonal variation of flood control storage, are up to 5% greater than the peak discharges for flood events occurring after December 1, when the full amount of flood control storage is available.

3.3 Impact of Increased Early Season Flood Storage Requirements on Regulated Peak Flows

To determine the impact of increased early season flood storage at Upper Baker Dam on regulated peak flows on the Skagit River near Concrete, the analysis described in Section 3.2 above was repeated using the **optional** flood storage requirements provided in Table 3-2. The results of the analysis are summarized in Table 3-6, and the peak flows of the weighted hydrographs for existing and optional flood storage requirements are compared in Table 3-7. Also shown in Table 3-7 are unregulated peak discharges and the peak discharges for 1 December hydrographs, when the full flood control storage would be available at both Upper Baker and Ross reservoirs under both the existing and optional flood storage scenarios. Samples of the October 1, December 1, and weighted regulated hydrographs for 25-year and 100-year events are provided in Figure 3-13 and Figure 3-14. A complete set of regulated hydrographs is available in digital format.

3.4 Conclusions

The data on weighted regulated hydrographs summarized in Table 3-3 for the existing flood control regulation indicate that consideration of the seasonal variation of flood control storage would increase estimates of the existing regulated peak flow quantiles for the Skagit River near Concrete by up to 5% for 50-year events and larger relative to peak flows with full flood control storage available. Smaller events show a smaller increase.

With increased early season flood control storage at Upper Baker, the peak flows for weighted hydrographs (Table 3-6) are up to 2% larger than peak flows with full flood control storage available. With the increased optional early season flood control storage, peak flows for weighted hydrographs are up to 6,800 cfs (3%) lower than for the existing regulation, as shown in Table 3-7.

4 Lower Baker Dam Flood Regulation

At the present time, no authorized flood control storage is provided at Puget Sound Energy's Lower Baker Dam. During USACE flood control operations at Upper Baker Dam, Lower Baker Dam is currently operated to pass inflows in accordance with the Baker River Project Water Control Manual (WCM) (USACE, 2000). There has, however, been a long term interest in potential flood control storage at Lower Baker, as reflected in Article 107b of the 2004 FERC Settlement Agreement for the relicensing of the Baker River Project which states:

> *(b) Additionally, from October 1 to March 1, licensee shall operate the Lower Baker storage reservoir to provide up to 29,000 acre-feet of storage for flood regulation, at the direction of the District Engineer, Corps of Engineers, acting on behalf of the Secretary of the Department of the Army, subject to the following: (i) such storage shall be provided only in accordance with arrangements that are acceptable to the Corps of Engineers; and (ii) such storage shall be provided only after suitable arrangements have been made to compensate the licensee for the 29,000 acre-feet of storage for flood regulation specified herein.*

This Chapter describes work undertaken to evaluate the peak flow reductions from potential new flood control storage at Lower Baker Dam. The work specifically evaluates peak flow reductions assuming the provision of 20,000 acre-ft of flood control storage at Lower Baker, as directed by the Seattle District. Analysis was undertaken for both the existing flood control operations at Upper Baker and Ross Dams and for the scenario with increased early season flood control storage at Upper Baker Dam as discussed in Chapter 3.

4.1 Lower Baker Reservoir Flood Regulation Plan

4.1.1 Lower Baker Dam Project Features and Spillway Gate Regulation Schedule

Lower Baker Dam is a semi-gravity concrete arch structure 285 feet high and 530 feet long with a center spillway section and left and right non-overflow sections. The spillway has an ogee-crest at elevation 428.62 ft. NAVD88 and 23 gated spillway bays. The 23 spillway gates are all 14.5 feet high and are numbered in ascending order from the right bank (west end of the dam). Gate 1 is 10.2 feet wide, Gate 2 is 10.4 feet wide and Gates 3 through 23 are each 9.4 feet wide (USACE, 2000).

Per information provided by the Seattle District, of the 23 spillway gates, 13 are motorized and can be operated at the push of a button, and the remaining 10 are manually operated by means of a gate car. The motorized gates take about 5 minutes to open. It is assumed that it takes from 2 to 3 hours to fully open (or close) all 10 of the manually operated gates.

A conceptual Spillway Gate Regulation Schedule (SGRS) for Lower Baker was developed following the guidance provided in Chapter 4 of EM 1110-2-3600, Management of Water Control Systems (USACE, 1987), and is provided in Figure 4-1. Key assumptions and sources of information for development of the SGRS were as follows:

- The spillway discharge rating with gates fully open was taken from a recently completed probable maximum flood (PMF) study (Tetra Tech, 2008). The spillway discharge rating represents free overflow up to approximately elevation 439.08 ft. NAVD88. Above that elevation, spillway discharge is affected to some degree by limits on gate openings for several of the gates. A summary of controlling elevations used in determination of the total spillway discharge rating curve is provided in Table 4-1. The total project discharge in Figure 4-1 is shown as the total spillway discharge with gates fully open plus a total powerhouse discharge of 6,000 cfs.
- This work evaluated the peak flow reduction from 20,000 acre-ft of flood control storage at Lower Baker. The flood control storage was assumed to be provided between the normal full pool at elevation 442.35 ft. NAVD88 and a minimum flood control pool elevation of 433.17 ft. NAVD88, as determined from stage-storage data dated 1 October 2004. In consultation with Puget Sound Energy and the Seattle District, no surcharge storage was assumed above the normal full pool elevation.
- The SGRS assumes a minimum discharge of 1,200 cfs, consistent with minimum instream flow requirements specified in Aquatics Tables 1 and 2 of Article 106 of the FERC Settlement Agreement.
- Computation of the gate regulation curves assumed a recession constant of 0.95 days[3] determined from the 500-year unregulated flood hydrograph for the Baker River above its confluence with the Skagit River from the March 2011 draft Hydrology Technical Documentation (unregulated flows dated 13 January 2011). The synthetic unregulated hydrograph is in turn patterned after unregulated flows determined from the 20-24 October 2003 flood. The recession rate of flood event inflows to Lower Baker Dam will in reality be affected by the operation of Upper Baker Dam.

The sensitivity of the gate regulation curves to uncertainty in (and variation in) the recession constant was evaluated by developing a second set of gate regulation curves assuming a recession constant of 1.25 days and is shown in Figure 4-2. The gate regulation curves are relatively insensitive to change in recession constant; all analysis for this work was performed using the SGRS from Figure 4-1 with a recession constant of 0.95 days.

4.1.2 Flood Control Regulation

A conceptual flood regulation plan for Lower Baker was developed with the objective of reducing peak flows on the mainstem Skagit River below the confluence with the Baker River. As in analysis of the impacts of increased early season flood control storage described in Chapter 3, the principal point of reference for evaluating peak flow reduction was the USGS gage site, Skagit River near Concrete. Peak flow reduction would be achieved by simply reducing outflows from Lower Baker Dam to a minimum

[3] The recession constant is defined as the time required for the discharge to decrease from any value Q_A to a value Q_B, where $Q_B = Q_A/2.7$.

release of 1,200 cfs coincident with the arrival of the peak flow on the Skagit and subject to the release requirements of the SGRS from Figure 4-1.

A preliminary assessment was undertaken of the potential forecast lead time for the arrival of the peak discharge at the Skagit River near Concrete gage. This was based on examination of the observed regulated discharge hydrographs from the floods of late November 1990, November 1995, October 2003 and November 2006 from the following USGS gage sites:

- Skagit River near Concrete (USGS gage 12194000)
- Baker River at Concrete (USGS gage 12193500)
- Sauk River near Sauk (USGS gage 12189500)
- Sauk River above White Chuck River, near Darrington (USGS gage 12186000)
- Skagit River at Marblemount (USGS gage 1218100)

Discharge hydrographs for the four events, taken from the USGS Instantaneous Data Archive, are shown in Figure 4-3 through Figure 4-6.

Examination of Figure 4-3 through Figure 4-6 indicates a difference in timing between the flood peak on Skagit River at Marblemount and the Skagit River near Concrete varying from roughly 5 hours in 2006 to as long as 12 hours in 1990. The variation in timing is likely due to differences in the timing of contributing flows from the major tributaries between Marblemount and Concrete, notably the Sauk River and the (regulated) Baker River. For current purposes, we assume that observation of the arrival of the flood peak at the Marblemount gage could be used, in conjunction with meteorological forecasts, to provide a forecast lead time of up to 6 hours for the arrival of the peak flow at the Skagit River near Concrete.

The time required to operate the gates at Lower Baker Dam to reduce releases to 1,200 cfs, or to the release specified by the SGRS, will depend on the gates in use at the time (motorized or manually operated) and the extent to which any of the manually operated gates are open. Operation of the gates could take anywhere from as little as 5 minutes (for operation of a motorized gate) to as long as three hours (for closure of all manually operated gates). In a worst case situation (from the point of view of gate operations), it appears that Lower Baker Dam could be operated for flood regulation in such a way as to reduce releases to 1,200 cfs over a two- to three-hour period starting six hours before the forecast arrival of the unregulated Skagit River peak flow. For conceptual modeling purposes (see Section 4.2), we have ignored the time required to operate the gates and have assumed that releases from Lower Baker would be reduced to 1,200 cfs, or the release specified by the SGRS, six hours before the arrival of unregulated peak flow. This is conservative in the sense that early reduction in releases from Lower Baker would induce additional flood storage and potentially reduce the effectiveness of Lower Baker flood control operation at the time of the peak flow on the Skagit River.

For this feasibility level assessment, it has also been assumed that flood control operations at Upper Baker Dam and Ross Dam would be as described in the current water control manuals. No work has been performed to optimize the joint operation of Lower Baker, Upper Baker and Ross. In this regard, it should be noted that the present work continues to assume that releases from Upper Baker would be reduced to 5,000 cfs three hours before the unregulated (natural) flow on the Skagit River near Concrete is forecast to reach 90,000 cfs. The operation of Upper Baker, as specified in the WCM, is not currently related to the arrival time of the Skagit River peak flow.

Evacuation of flood control storage at Lower Baker would be coordinated with evacuation releases from Upper Baker and Ross to avoid exceeding the regulated peak flow on the Skagit River near Concrete on the receding limb of the flood hydrographs. Because of the limited outlet capacity at Lower Baker at low pool levels, precedence in evacuation of flood control storage would be given to Upper Baker, with the Lower Baker pool being allowed to continue filling during Upper Baker evacuation as necessary, but to an elevation no greater than the normal full pool.

In summary, flood control operations at Lower Baker are assumed as follows:

- 20,000 acre-ft of flood control storage would be provided from October 15 through March 1 between elevations 433.17 ft. and 442.35 ft. NAVD88.
- On the rising limb of the Skagit River flood hydrograph, Lower Baker would be operated to pass inflows until six hours before the forecast unregulated peak flow on the Skagit River near Concrete.
- Releases from Lower Baker would be reduced to a minimum of 1,200 cfs, or the discharge specified by the SGRS, six hours before the forecast arrival of the unregulated Skagit River peak flow. Because of the limited outlet capacity (spillway plus power house) at low pool levels at Lower Baker, encroachment into the 20,000 acre-ft flood control pool may occur due to high inflows before a reduction in release is triggered.
- Coordinated evacuation of Upper Baker and Lower Baker flood control storage would begin approximately four hours after the regulated peak flow on the Skagit River near Concrete. Precedence for evacuation would be given to Upper Baker. Lower Baker releases would be coordinated with evacuation releases from Upper Baker and Ross to avoid exceeding the regulated peak flow on the Skagit River near Concrete on the receding limb of the flood hydrographs.

The overall flood control operation for Ross, Upper Baker and Lower Baker is summarized in Table 4-2.

4.2 Impact of Flood Control Storage at Lower Baker Dam

The impact of flood control storage at Lower Baker Dam on regulated peak flows was analyzed using the spreadsheet routing "model" of the Skagit and Baker River projects described in Chapter 3, further modified to incorporate the flood control operation at Lower Baker Dam described in Section 4.1.

The modified spreadsheet model was used to route winter (October – March) flood hydrographs with return periods of 5, 10, 25, 50, 75, 100, 250, and 500 years through the Upper Baker, Lower Baker, and Ross reservoirs downstream to the USGS Skagit River near Concrete gage. Flood routing was performed with starting reservoir storage conditions on October 1, October 15, November 1, November 15, and December 1 per the existing flood control storage requirements at Upper Baker and Ross as well as with increased early season flood control storage at Upper Baker as in Chapter 3. The starting flood control storage at Lower Baker was assumed to be zero on October 1 and 20,000 acre-ft from October 15 to March 1. The starting flood control storage conditions assumed at the three reservoirs are summarized in Table 4-3 and Table 4-4.

As in Chapter 3, all analyses presented here were performed using unregulated tributary inflows to the Skagit and Baker Rivers dated 13 January 2011, originally provided in digital form with the March 2011 draft Hydrology Technical Documentation in file GI_Flows_Revised2_BestWorst.dss.

Recognizing the limitations of the channel routing component of the spreadsheet model and to ensure consistency with previous work, flood hydrographs output by the spreadsheet model for the Baker River below Lower Baker Dam and the Skagit River at Marblemount (above the confluence with Cascade River) were input to an existing upper basin HEC-RAS model, described in the study Hydraulic Technical Documentation (USACE, 2013b), and re-routed to Concrete to produce final regulated flood hydrographs at Concrete.

The regulated peak discharges from the analyses for the Skagit River near Concrete are summarized in Table 4-5 for the scenarios with new flood control storage at Lower Baker Dam and with existing storage at Upper Baker and Ross and in Table 4-6 for the scenarios with new flood control storage at Lower Baker Dam, increased early season storage at Upper Baker, and existing storage at Ross. Also shown in the two tables are the approximate contribution to the regulated peak discharge from Lower Baker and Ross.

Note in Table 4-5 and Table 4-6, that the contributions to the regulated peak discharge from Lower Baker are moderately sensitive to the timing of the regulated hydrograph at Concrete, which in turn is affected by regulation at Ross. For example, for the 100-year event in Table 4-5, the contribution from Lower Baker to the peak discharge is greater for the 15 November scenario (5,100 cfs) than for the 1 December scenario (3,100 cfs) even though the amount of flood control storage at Upper and Lower Baker are the same for those two scenarios. The difference in contribution is due to a difference in regulation at Ross, which only has 60,000 acre-ft of flood control storage on 15 November as opposed to 120,000 acre-ft on 1 December. It should also be noted that use of the spreadsheet routing model requires judgment and manual intervention on the receding limb of flood hydrographs to meet various soft regulation constraints[4]. User decisions affect evacuation rates from Upper and Lower Baker and may produce minor inconsistencies in the estimated contribution from Lower Baker to the regulated peak discharge for the various simulation scenarios.

Comparisons of peak discharges for scenarios with and without Lower Baker flood control storage are provided in Table 4-7, and absolute and percent reductions in peak flows are summarized in Table 4-8

---

[4] Soft constraints include: regulating flows on the recession limb of flood hydrographs in a way which avoids a secondary peak at Concrete; controlling releases from Ross to avoid discharges at Newhalem greater than 30,000 cfs to the extent possible; and others.

and Table 4-9. The peak discharge data for scenarios without Lower Baker flood control storage are as determined in Chapter 3.

Plots of selected outputs from the spreadsheet reservoir routing model for 1 December simulations with Lower Baker flood control storage are provided in Figure 4-7, Figure 4-8, and Figure 4-9 for 25-, 100-, and 500-year events respectively. The corresponding regulated hydrographs from the HEC-RAS model for the Skagit River near Concrete for scenarios with and without Lower Baker flood control storage are provided in Figure 4-10, Figure 4-11, and Figure 4-12. A complete set of regulated hydrographs is available in digital form in dss file GI2012_T4_LB20K_Routed.dss dated 20 November 2012.

It can be seen from Figure 4-7 that for the 25-year event, releases from Lower Baker can be effectively reduced to 1,200 cfs and held at that level until the peak flow on the Skagit River has passed. However for the larger events (Figure 4-8 and Figure 4-9) the ability to restrict releases from Lower Baker is progressively diminished. At the 500-year event (Figure 4-9), high inflows to Lower Baker cause encroachment into the flood control pool before reductions in release rates are triggered under the proposed flood regulation plan (releases are reduced at 00:00 on 21 October in Figure 4-9, six hours before the unregulated peak flow on the Skagit River near Concrete). When the release from Lower Baker is reduced, it is only reduced to an initial 17,500 cfs as required by the SGRS. Releases are then rapidly increased as the pool level rises, again in accordance with the SGRS.

The effectiveness of the assumed flood control regulation at Lower Baker is determined not only by the reduction in peak discharge achieved, but also by the amount of flood control space used and the time required for evacuation of the flood control pool, as shown in Table 4-10 for 1 December simulations. Except for the 500-year event, the maximum pool elevations achieved at Lower Baker in the 1 December simulations were determined by: i) the rate at which Upper Baker was evacuated and ii) the goal of avoiding an increase in peak discharge on the Skagit River near Concrete on the receding limb of the flood hydrograph. To meet this latter “soft” constraint usually required continuing to fill Lower Baker after the peak has passed to avoid a situation in which the evacuation of Upper Baker could produce a second higher Skagit River peak. The simulation results show that careful coordinated operations of Upper Baker and Lower Baker would be required to expedite evacuation of Upper Baker while avoiding a secondary peak on the Skagit. For the 500-year event, the hydrograph volume is so large that flood control storage at Lower Baker is ineffective.

The time required to evacuate Lower Baker shown in Table 4-10 is reported relative to the time of the unregulated peak flow. The time to evacuate is influenced by the same two operations parameters as determine the maximum pool elevation, i.e. the evacuation rate of Upper Baker and the operations required to meet the “soft” constraint of avoiding a second Skagit River peak. For some events, a faster evacuation time than reported could be achieved through adoption of a more aggressive evacuation policy.

4.3 Conclusions

Provision of 20,000 acre-ft of flood control storage at Lower Baker Dam allows the flow contribution from the Baker River system to the peak flow on the Skagit River to be restricted to 1,200 cfs for events up to the 50-year event for scenarios with full flood control storage available at Upper Baker and Ross. This results in a reduction in main stem peak flows of between about 8,000 and 13,000 cfs, corresponding to peak flow reductions varying from 4% to 9% depending on event return period and scenario (i.e. event date and hence flood control storage amount at Upper Baker and Ross). For larger events, and for scenarios with less than the full amount of flood control storage at Upper Baker, flood storage at Lower Baker becomes progressively less effective. This is due to:

- the relatively small amount of flood control storage assumed at Lower Baker ,
- the dam's limited outlet capacity at low pool levels, and
- the project discharges required under the spillway gate regulation schedule (SGRS).

As can be seen from Figure 4-1, the maximum total discharge from Lower Baker (spillway plus powerhouse) with the water level at the minimum flood control pool elevation is approximately 13,300 cfs. During large events, or early season events with reduced storage at Upper Baker, inflows to Lower Baker exceed this amount early on the rising limb of flood hydrographs, encroaching on the flood control pool before reductions in release rates from Lower Baker are triggered under the proposed flood regulation plan, and hence reduce flood control storage available for regulation of the peak flow on the Skagit.

Further minor improvements in flood control performance may be possible through:

- optimization of the joint flood control operations of Upper Baker, Lower Baker, and Ross;
- refinement of the Lower Baker Spillway Gate Regulation Schedule to better account for the effects of upstream flood control operations at Upper Baker;
- less conservative assumptions regarding the time required for Lower Baker gate operations; and,
- potential use of surcharge storage at Lower Baker above the normal full pool elevation.

5 Hydraulic Modeling for Economic Flood Damage Analysis

This Chapter documents the hydraulic modeling performed to provide information on flooding in the lower Skagit River basin for use in economic flood damage analysis. Flooding is characterized as required for risk and uncertainty analysis following the guidance provided in EM 1110-2-1619 "Risk-Based Analysis for Flood Damage Reduction Studies" (USACE, 1996). Hydraulic models of the lower Skagit River basin used for this analysis consist of 1-D HEC-RAS models of the Skagit River channel and its left-bank floodplain storage areas upstream from Mount Vernon, and 2-D FLO-2D models of the Skagit River floodplain excluding those left-bank storage areas included in the HEC-RAS models. (Note that the Skagit River channel is not modeled in FLO-2D in this study.) Output from the HEC-RAS models includes spill onto the floodplain under various scenarios, due to levee overtopping and/or levee breaches. These spill flows are used as input to the FLO-2D models which route flows across the floodplain to the ultimate downstream receiving waters of Skagit Bay, Padilla Bay and Samish Bay. Development of the existing condition hydraulic models is described in detail in the Hydraulic Technical Documentation (USACE, 2013b).

5.1 Economic Damage Reaches

A total of thirteen economic damage reaches and associated index points were defined for the lower Skagit River basin by the Seattle District. Index points for each damage reach were initially based on existing condition hydraulic modeling results and assessments of potential levee failure locations and failure elevations presented in the draft 2011 Hydraulic Technical Documentation (USACE, 2011b). The initially selected index points and associated levee failure locations and elevations were reviewed and refined for the current work by NHC in collaboration with the Seattle District. The majority of the selected index points are just upstream from associated levee failure locations as discussed further in Section 1.3 below. Index points associated with damage reaches 6, 6A, and 8 are at locations not protected by levees, where flooding occurs either due to out-of-bank high flows or overtopping of natural high ground.

The thirteen damage reaches and their associated index points are shown in Figure 5-1 and listed in Table 5-1. Note that several damage reaches are associated with a common index point. For example, damage reaches 6 and 6A share a common index point indicated as (6,6A) on Figure 5-1. In the following report Sections, index points are referenced by either a nominal River Mile or, more precisely, by the HEC-RAS model cross-section used to represent in-channel hydraulic conditions at the index point.

As described in the Hydraulic Technical Documentation, flooding of the damage reaches was modeled using either HEC-RAS or FLO-2D. HEC-RAS was used to model in-channel flows for the entire Skagit River, overbank flow upstream from the Highway 9 bridge near Sedro-Woolley (damage reach 8), and left bank storage areas upstream from Mount Vernon, including the Nookachamps area (damage reaches 5, 5A, 6, and 6A). The FLO-2D model domain covers the right bank floodplain (damage reaches 1, 1A, 2, 2A, and 7), Fir Island (damage reach 3), and the left bank floodplain from Mount Vernon downstream (damage reaches 4 and 4A). A map showing the domain of the HEC-RAS and FLO-2D models is provided in Figure 1- 1.

5.2 Levee Failure Data and Hydraulic Modeling Approach for Flood Damage Analysis

The principal objectives of the hydraulic modeling described in this Chapter were:

i) to develop flow-frequency and stage-discharge relationships at the various index points and to characterize uncertainty in those relationships, and,

ii) to establish relationships between in-channel stage at each of the selected index points and flooding (and hence economic damages) in the associated damage reach.

In-channel flow-frequency and stage-discharge relationships were developed directly from HEC-RAS model output.

To establish relationships between in-channel stage and flooding in a damage reach, the maximum flood depth and area of inundation across the lower Skagit River floodplain were determined for three floods at each index point: a "minimum flood", the 100-year flood, and an extreme 500-year plus two standard deviation ("500-year + 2SD") flood. The "minimum flood" was loosely defined as the smallest damaging event at the index point in question. In the case of areas protected by levees, this would be the smallest flood which would result in levee failure. In the case of areas not protected by levees (i.e. damage reaches 6, 6A, and 8), the "minimum" flood was selected as that event which would just start to flood developed property. The derivation of the "500-year + 2SD" flood is described in Section 5.3.1.

A key consideration in the analysis is the estimation of Probable Failure and Probable Non-Failure Points for the levee system. The Probable Failure Point (PFP) is defined as the in-channel water surface elevation (WSEL) at which there would be an 85% probability of levee failure. The Probable Non-Failure Point (PNP) is defined at the in-channel WSEL at which there is a 15% probability of levee failure. A Likely Failure Point (LFP) is also defined at which WSEL there is a 50% probability of levee failure. For the present study, the LFP is taken to be midway between the PFP and the PNP. The approach to determining PFPs and PNPs is described in the Hydraulic Technical Documentation along with a detailed listing of the estimated 15% and 85% failure probability elevations under existing conditions for each levee segment. PNPs and PFPs at the index point locations are also discussed in Sections 5.4.2 and 5.6.2 below, which present results for modeling of the existing condition and with improved levee scenario, respectively.

Selection of index points is closely linked to likely levee failure locations. For each damage reach, the single most likely levee failure location which would result in flooding in that damage reach was identified from examination of the existing condition PNP data. Where there were multiple potential levee failure locations with similar failure probabilities, the failure location expected to produce the greatest flood damage was selected. The index point associated with the selected levee failure location was then taken at the next upstream cross-section within the HEC-RAS model for the purpose of reporting in-channel flows and water levels.

For damage reaches not protected by levees (i.e. damage reaches 6, 6A, and 8), index point locations were selected to provide what was judged to be the most reliable relationship between in-channel stage and flood damage in the damage reach.

Hydraulic modeling for flood damage analysis described in this Chapter was conducted for three scenarios: existing conditions (Section 5.4); with additional early season flood regulation storage (Section 5.5); and with improved levees (Section 5.6). Modeling for the various scenarios was done with and without levee breaches as shown below:

Scenario	No-Breach Simulation	With-Breach Simulation
Existing Conditions	Yes	Yes
Additional Early Season Flood Regulation Storage	Yes	No
Improved Levees	Yes	Yes

The modeling procedure for the with-breach simulations was as follows:

i) HEC-RAS simulations were performed for the 2-, 5-, 10-, 25-, 50-, 75-, 100-, 250-, and 500-year floods (referred to as the 2-year through 500-year floods for the remainder of this Chapter) assuming no levee breaches. No-breach 2-year through 500-year water surface profiles were created and the water surface elevations at each levee failure location were compared against the 15% probability of levee failure elevation (the PNP) at that location. For each levee failure location, the smallest flood event resulting in a no-breach water surface elevation exceeding the PNP was selected as the minimum flood for levee breaching at that location. For those index points not associated with a levee failure location, the minimum flood was selected as that event which would just start to flood developed property.

ii) Levee breach simulations were performed within HEC-RAS for each levee failure location for the minimum flood, 100-year flood and 500-year + 2SD flood. For the minimum flood, levee failure is initiated as soon as the water surface elevation reaches the 15% probability of failure elevation at the selected failure location. For the 100-year and 500-year + 2SD floods, a levee breach is initiated when the water surface elevation reaches the mid-point between the 15% and 85% probability of failure elevations. A fully-developed breach width of 300 feet was assumed for the minimum flood and 400 feet for the 100-year flood and larger. All breaches were assumed to take three hours to reach their fully developed sizes.

iii) To identify the maximum flood inundation that might occur in each damage reach due to a levee breach, only a single levee breach was assumed to occur in any one flood event, with overtopping flooding, but no levee breaches, occurring elsewhere in the system.

iv) For each levee breach flood (minimum flood, 100-year flood and 500-year + 2SD flood) at each levee failure location, the breach flows and the concurrent overtopping flows from each lateral structure were written from HEC-RAS to a HEC-DSS file.

v) For each flood and each levee failure, the levee breach hydrograph and overtopping hydrographs stored in HEC-DSS were input to the FLO-2D model and routed across the floodplain. FLO-2D grids of maximum water surface elevation and maximum water depth were stored for post-processing.

vi) The maximum flood depth grids from FLO-2D were merged with similar data from HEC-RAS for the area upstream from Highway 9 (damage reach 8) and for left bank storage areas modeled in HEC-RAS (damage reaches 5, 5A, 6, and 6A) to produce grids of maximum flood depth over all damage reaches. Topographic data were then in turn used with the grids of maximum flood depth to produce gridded data of maximum water surface elevations for use in flood damage analysis. The final result of this GIS merging operation is seamless depth and water surface elevation data, gridded at the FLO-2D model resolution of 400 feet by 400 feet. The data were presented in both shapefile and ESRI grid formats to allow flexibility in processing for flood damage analysis.

5.3 Hydrologic Inputs and Discharge Uncertainty

5.3.1 Hydrologic Inputs

Hydrologic inputs to the HEC-RAS model for the existing condition and improved levee scenarios were 2-year through 500-year weighted regulated hydrographs for the existing flood control regulation with existing flood control storage at Upper Baker and Ross reservoirs.   The development of the existing condition 2-year through 500-year hydrographs is described in the Hydrology Technical Documentation (USACE, 2013a) and in Chapter 3 of this study report.

A 500-year plus two standard deviation (500-year + 2SD) existing condition regulated hydrograph was also developed to cover the full range of events required for economic flood damage analysis. This hydrograph was constructed by scaling the ordinates of the 500-year weighted regulated hydrograph for the Skagit River near Concrete by the ratio (500-year +2D / 500-year) one-day weighted regulated peak flows (a ratio of 1.51). The 500-year and the 500-year + 2SD one-day weighted regulated peak flows for the Skagit River near Concrete were taken from the frequency analyses provided in Appendix D of the Hydrology Technical Documentation. The 500-year weighted regulated hydrograph for the Skagit River near Concrete is shown in Appendix E of the Hydrology Technical Documentation. The 500-year + 2SD event at Concrete was assumed to occur with the 500-year tributary inflow to the Skagit River basin below Concrete and the 500-year flood on the Samish River. The relative timing of the 500-year + 2SD flood at Concrete and downstream 500-year tributary inflows was assumed to be the same as other hydrologic design events, again as described in the Hydrology Technical Documentation.

Hydraulic modeling (for no-breach simulations only) was also conducted for the existing hydraulic condition but with additional early season flood control storage at Upper Baker (see Section 5.5 below). Development of hydrologic inputs for this scenario (i.e. weighted regulated hydrographs with additional early season flood control storage at Upper Baker) is also described under Chapter 3.

5.3.2 Discharge Uncertainty and Equivalent Record Length

Flow-frequency curves at each index point location were produced for use in economic flood damage analysis by extracting simulated 2-year through 500-year peak flows for the various scenarios from the

HEC-RAS model and then applying the graphical exceedance probability approach within HEC-FDA (USACE, 1998).

HEC-FDA computes confidence bounds for frequency analysis using the order statistics approach with a user-specified equivalent record length. A 65-year equivalent record length was selected for this work considering the streamflow record for the Skagit River near Concrete which provides much of the basis for the determination of hydrologic design events. The Skagit River near Concrete record comprises: a 110-year historic record (extending back to 1897); a continuous systematic record of approximately 80 years (extending back to 1924); and a homogeneous record of regulated flows of approximately 50 years (extending back to 1956). The available data are discussed in detail in the Hydrology Technical Documentation.

To allow HEC-FDA to produce realistic confidence bounds at high exceedance probabilities, peak flows with an exceedance probability of 0.999 were estimated from the regulated flood frequency curve provided in Appendix D of the Hydrology Technical Documentation for the Skagit River near Mount Vernon and incorporated into the HEC-FDA analysis.

Note that the confidence bounds computed by FDA and reported here **do not** recognize physical limitations on discharge; peak flows and stage on the Skagit River below Sedro-Woolley are effectively limited by the levee system capacity.

5.4 Hydraulic Modeling for Existing Conditions

Existing condition modeling relied on existing regulation hydrologic inputs and existing condition hydraulic geometry. As noted in the Hydraulic Technical Documentation, the existing condition hydraulic geometry includes the Mount Vernon Flood Wall, construction of which had been partly completed at the time of this report, and fixed debris blockages on the Burlington Northern Railroad and Great Northern Railroad bridges of 6,000 square feet and 4,000 square feet respectively.

As described in detail in Section 5.2 above, the following hydraulic modeling was performed:

- HEC-RAS modeling without levee breaches (no-breach analysis) for the 2- through 500-year floods. Levees were assumed to overtop without breaching.
- HEC-RAS modeling with levee breaches for the minimum flood, the 100-year flood, and the 500-year + 2SD flood. Output from these runs is used as input to FLO-2D to determine flooding extents, depths and water surface elevations over the floodplain.
- FLO-2D modeling for the with-breach scenarios for the minimum flood, the 100-year flood, and the 500-year + 2SD flood.

5.4.1 No-Breach Analysis

The existing condition HEC-RAS model was run without levee breaches but allowing levee overtopping for the existing 2-year through 500-year floods. Flow-frequency curves, stage-discharge curves and water surface profiles are provided in Appendix 5-1, and peak discharge quantiles are provided in Table

5-2. Also shown on the water surface profiles are 15% PNP levee failure elevations for use in the with-breach analyses.

As noted in Section 5.3.2 above, flow-frequency curves were created using the graphical exceedance probability approach within HEC-FDA assuming an equivalent record length of 65 years. The peak flows for flow-frequency analysis were extracted directly from the HEC-RAS model. The HEC-RAS discharge hydrographs for index points XS 22.2 and 21.6 show questionable abrupt local maxima for several events which likely cause overestimation of flood quantiles in two cases, highlighted in Table 5-2. This behavior appears to be related to the model's representation of left bank levees and ineffective flow areas in the reach between the BNSF and SR-9 bridges. The greatest uncertainty is in the 25-year peak flow at XS 21.6 which may be overstated in Table 5-2 by about 9,000 cfs. The apparent instability in discharge appears to have no impact on the corresponding stage hydrographs.

The stage-discharge curves were created by simply plotting stage against discharge at each index point from the HEC-RAS simulation of the 500-year event. Note that the stage-discharge curves show pronounced hysteresis effects for index points XS 22.2 and 21.6 related to the locally flatter river slope and availability of large over bank storage volumes in the Nookachamps and Hart's slough areas.

Full results are provided in the HEC-RAS model and related HEC-DSS files included with the digital deliverable for the study.

5.4.2 With-Breach Analysis

Existing condition with-breach analyses were conducted as described under Section 5.2 above.

For each index point/levee failure location, the minimum flood (defined in Section 5.2) was determined by comparing water surface profiles from the no-breach analysis against the PNP at that location (or against the estimated zero-damage flood elevation for damage reaches not protected by levees). Details of the index points/levee failure locations, existing condition failure elevations and existing condition minimum floods by index point are provided in Table 5-3.

For each index point, HEC-RAS simulations were performed with a levee breach for the minimum flood, 100-year flood and 500-year + 2 SD flood. The peak flows and peak in-channel water levels at each index point are summarized in Table 5-4. For index points associated with a levee failure location, the data in Table 5-4 are for the scenario with a breach at that failure location. Peak in-channel stages at several locations (highlighted in Table 5-4) occur on the rising limb of flood hydrographs immediately before the triggering of a downstream levee breach, which results in a rapid drawdown of in-channel water levels. Stages reported in Table 5-4 are peak post-breach stages which are expected to be better related to maximum flood extent and depth in the associated damage reach. An example of a with-breach stage hydrograph illustrating this point is provided in Figure 5-2. A similar issue arises with the peak flow for the 100-year event for a levee breach at RM 16.8.

As described in Section 5.2 above, breach hydrographs and levee overtopping hydrographs from HEC-RAS used as input to FLO-2D. The FLO-2D and HEC-RAS outputs were then merged to produce grids of maximum flood depth and maximum water surface elevation. A sample grid of maximum flood depths is provided in Figure 5-3 (100-year event with levee failure at approximately RM 21.3; i.e., failure into damage reach 1). Note that in the 100-year event, there is extensive flooding due to levee overtopping unrelated to the single assumed levee breach.

Full results including the HEC-RAS model, levee breach and overtopping hydrographs, FLO-2D model and GIS shape files of maximum flood depths and maximum water surface elevations are provided in the digital deliverables for the study.

5.5 Hydraulic Modeling with Additional Early Season Flood Regulation Storage

Hydraulic modeling with additional early season flood regulation storage at Upper Baker was conducted using weighted regulated hydrographs (see Section 3.3) and the existing condition hydraulic geometry. As noted previously, the existing condition hydraulic geometry includes the Mount Vernon Flood Wall and fixed debris blockages on the Burlington Northern Railroad and Great Northern Railroad bridges of 6,000 square feet and 4,000 square feet, respectively.

The following hydraulic modeling was performed:

- HEC-RAS modeling without levee breaches (no-breach analysis) for the 2- through 500-year floods. Levees were assumed to overtop without breaching.

No modeling of levee breaches or flood inundation was performed for this scenario.

5.5.1 No-Breach Analysis

No-breach analysis was performed similarly to the existing condition analysis described in Section 5.4.1. The existing condition HEC-RAS model was run without levee breaches but allowing levee overtopping for the 2-, 5-, 10-, 25-, 50-, 75-, 100-, 250-, and 500-year floods with additional early season flood regulation storage at Upper Baker. Flow-frequency curves and water surface profiles are provided in Appendix 5-2, and peak discharge quantiles are provided in Table 5-5. As with the existing condition flood quantiles in Table 5-2, quantiles which may be overestimated due to questionable model behavior are highlighted. Since these simulations were performed with the existing condition hydraulic geometry, stage-discharge curves are unchanged from the existing condition simulations (see Section 5.4.1 and Appendix 5-1).

The differences between no-breach flood quantiles with additional early season flood storage (Table 5-5) and no-breach quantiles for the existing regulation condition (Table 5-2) are provided in Table 5-6. It can be seen that the modest flood reduction benefits due to increased early season flood regulation seen just before flows leave the confined Skagit River valley at Sedro-Woolley (cross-section XS 23.2) are further reduced by the impacts of right bank spill and by routing and attenuation of flood hydrographs through the Nookachamps storage area. For the 100-year event, for example, the effect of additional early season flood storage is to reduce the peak flow at Sedro-Woolley (XS 23.2) by 6,700 cfs relative to the existing condition. Downstream from the BNSF bridge (at XS 16.78), however, the peak flow reduction is only a nominal 400 cfs.

5.5.2 With-Breach Analysis

No analysis of with-breach conditions was conducted for this scenario.

5.6 Hydraulic Modeling with Improved Levees

Hydraulic modeling of an improved levee scenario relied on existing regulation hydrologic inputs and minor modifications to the existing condition hydraulic geometry to reflect local levee improvements. The assumed levee improvements (primarily local levee raises), defined by the Seattle District, are shown in the levee profile plots of Figure 5-4 through Figure 5-6. It is assumed that any levee raise is accompanied by an equivalent increase in the levee PNP and PFP elevations. As noted previously, the existing condition hydraulic geometry includes the Mount Vernon Flood Wall and fixed debris blockages on the Burlington Northern Railroad and Great Northern Railroad bridges of 6,000 square feet and 4,000 square feet, respectively. The improved levee condition provides the baseline condition for modeling of flood management alternatives in Chapter 6.

The following hydraulic modeling was performed in a similar manner to the modeling for the existing condition scenario:

- HEC-RAS modeling without levee breaches (no-breach analysis) for the 2- through 500-year floods. Levees were assumed to overtop without breaching.
- HEC-RAS modeling with levee breaches for the minimum flood, the 100-year flood, and the 500-year + 2 SD flood. Output from these runs is used as input to FLO-2D to determine flooding extents, depths and water surface elevations over the floodplain.
- FLO-2D modeling for the with-breach scenarios for the minimum flood, the 100-year flood, and the 500-year + 2 SD flood.

5.6.1 No-Breach Analysis

No-breach analysis was performed similarly to the existing condition analysis described in Section 5.4.1. The HEC-RAS model with improved levees was run without levee breaches but allowing levee overtopping for the existing condition 2-year through 500-year floods. Flow-frequency curves, stage-discharge curves and water surface profiles are provided in Appendix 5-3, and peak discharge quantiles are provided in Table 5-7. As with the existing condition flood quantiles in Table 5-2, quantiles which may be overestimated due to questionable model behavior are highlighted.

The differences between no-breach flood quantiles with improved levees (Table 5-7) and no-breach quantiles for the existing condition (Table 5-2) are provided in Table 5-8. Interpretation of model results is complicated by the effects of levee raises on spill elsewhere in the system. The model results, for example, show fairly significant reductions in peak flows at cross-section XS 17.9 for the 50-year event and larger. This is due to the right bank levee raise immediately upstream from the BNSF bridge (see Figure 5-4) which reduces spill over the lateral structure immediately downstream from XS 17.9, a slight increase in water levels upstream from the BNSF bridge, and a resultant increase in right bank spill elsewhere upstream from XS 17.9. The net result is a decrease in flow at XS 17.9 but essentially no change in in-channel flows at and downstream from the BNSF bridge. With a slight increase in water levels upstream from the BNSF bridge, one would expect flow through the bridge opening (as reflected by flows at XS 16.78) to increase, even if only slightly. The reason for the slight reduction in flow at XS 16.78 and other downstream locations for 100-year events and larger has not been resolved satisfactorily, but may be related to interpolation in the BNSF bridge hydraulic table within HEC-RAS. Minor errors in in-channel flows are unlikely to affect estimates of flood extent and flood depth in the economic damage reaches.

5.6.2 With-Breach Analysis

With-breach analyses for the improved levee condition were conducted in a similar manner to the existing condition with-breach analyses described under Section 5.4.2 above, but with adjustments to the levee breach data to reflect levee improvements. The adjusted levee breach data, comprising increases in levee breach elevations (PNP, LFP and PFP) and increases in levee crest elevations, are shown in Table 5-9, with changes from the existing condition levee data highlighted. The increases in PNP also increased the minimum breach flood at certain index points. Changes in the minimum breach flood are also highlighted in Table 5-9.

For each index point, HEC-RAS simulations were performed with a levee breach for the (revised) minimum flood, 100-year flood and 500-year + 2 SD flood. The peak flows and peak in-channel water levels at each index point are summarized in Table 5-10. As before, peak flows at index points associated with a levee failure location are for the scenario with a breach at that failure location. As in the existing condition with-breach analyses, peak in-channel stages at several locations (highlighted in Table 5-10) occur on the rising limb of flood hydrographs immediately before the triggering of a downstream levee breach. Stages reported in Table 5-10 are peak post-breach stages which are expected to be better related to maximum flood extent and depth in the associated damage reach.

As in the existing condition with-breach analyses, breach hydrographs and levee overtopping hydrographs from HEC-RAS were written to a HEC-DSS data base and then used as input to FLO-2D. The FLO-2D and HEC-RAS outputs were then merged to produce grids of maximum flood depth and maximum water surface elevation.

Full results including the HEC-RAS model, levee breach and overtopping hydrographs, FLO-2D model and GIS shape files of maximum flood depths and maximum water surface elevations are provided in the digital deliverables for the study.

5.7 Stage-Discharge Uncertainty

A number of factors contribute to uncertainty in stage-discharge relationships. For the current study, in-channel stage-discharge relationships were determined by numerical modeling using HEC-RAS. The primary sources of uncertainty in those relationships are expected to be the following:

- Uncertainty in stage-discharge data used for hydraulic model calibration (these include both high water mark data and associated discharge estimates from past floods, and stage-discharge measurements from the USGS Skagit River near Mount Vernon gage)

- Uncertainty in hydraulic geometry data
- Uncertainty in model representation of hydraulic structures (i.e. bridges and levees)
- Uncertainty in hydraulic model parameters

The standard deviation of uncertainty from multiple **independent** sources can be determined as:

$$S_{Total} = \sqrt{(S_1^2 + S_2^2 + S_3^2 + \ldots + S_p^2)}$$

where:

S_{Total} = Total standard deviation of uncertainty

P = Number of independent sources of uncertainty

S_i = Standard deviation of uncertainty from Source i, where uncertainty from Sources 1 through p are independent (and normally distributed).

EM-1110-2-1619 (USACE, 1996) recommends that the total standard deviation of uncertainty in stage be determined from:

$$S_{Total} = \sqrt{(S^2_{natural} + S^2_{model})}$$

where:

S_{Total} = Total standard deviation of stage uncertainty

$S_{natural}$ = Standard deviation of natural stage uncertainty

S_{model} = Standard deviation of model stage uncertainty

For this relationship to hold, natural and model stage uncertainty should be independent and errors in stage should be normally distributed. However in this instance, natural and model uncertainty are not independent and there is no good basis for distinguishing between the two. For example, one of the primary sources of natural stage uncertainty is channel roughness, which is also a primary source of hydraulic model uncertainty. Furthermore, the stage errors in this system, particularly at high stages, are not normally distributed since maximum stages (and discharges) are limited by the capacity of the levee system downstream from Sedro-Woolley.

In the absence of a good basis for quantifying natural and model uncertainty, estimation of total stage uncertainty for study relies on consideration of stage-discharge measurement uncertainty, analysis of hydraulic model calibration to available stage-discharge observations, model sensitivity to uncertainty in key model parameters, and professional judgment.

5.7.1 Model Calibration and Measurement Uncertainty

The HEC-RAS hydraulic model was calibrated to data from the flood of October 2003 and validated against data from the floods of November 1995 and November 2006 as described in the Hydraulic Technical Documentation (USACE, 2013b). Data available for model calibration and validation in the reach downstream from Sedro-Woolley comprised stage and discharge data from the USGS gage Skagit River near Mount Vernon (12200500), and a variety of other High Water Marks.

Analysis of Stage-Discharge Rating Data

Stage-discharge measurements and the current stage-discharge rating (Rating 19) for the USGS Mount Vernon gage were obtained from the USGS National Water Information System. Stage-discharge measurements are available on-line from September 1983 through present. The current stage-discharge rating (Rating 19) is effective from 4 November 2006. An earlier rating (Rating 17) effective from approximately November 1995 through December 2005 was obtained from the USGS Sedro-Woolley field office. A plot of Rating 19, Rating 17, the stage-discharge measurements since September 1983, and a rating at the gage site generated from the existing condition HEC-RAS model are all shown in Figure 5-7. Note that the shape of Rating 19 above about 80,000 cfs appears to be determined by a single discharge measurement of 125,000 cfs made on 7 November 2006 and rated "poor" by the USGS. Data inconsistencies and difficulties in HEC-RAS model validation to the 2006 event are discussed in the Hydraulic Technical Documentation.

Considering the variation of stage measurements about the HEC-RAS model rating for discharges above 67,700 cfs (the 2-year peak discharge from analysis of the gage record), the standard deviation of error is 1.2 feet including the measurement from 7 November 2006, and 0.7 feet excluding that measurement, as summarized in Table 5-11. These figures reflect measurement error, model error and natural variability.

Analysis of Other High Water Mark Data

As noted previously, the HEC-RAS model was calibrated to various high water mark data from the flood of October 2003 and validated against data from the floods of November 1995 and November 2006. Comparisons of simulated water surface elevations against high water marks for those events are provided in Figures 3, 5 and 8 and Tables 7 through 9 of the Hydraulic Technical Documentation. The simulated and reported high water mark elevations in the area of primary interest downstream from Sedro-Woolley are summarized in Table 5-12. The standard deviation of differences between simulated and reported high water mark elevation for the available data from all three floods is 1.2 feet as shown in the summary in Table 5-11. As discussed in the Hydraulic Technical Documentation, a number of the high water marks are of questionable accuracy. The estimated standard deviation of error reflects measurement error, model error and natural variability. Note that since bridge debris was not a significant factor in any of the model calibration and validation events, the HEC-RAS model runs for those events assumed no debris.

5.7.2 Hydraulic Model Parameter Uncertainty

Analysis of the effects of uncertainty in hydraulic model parameters focused on uncertainty in Manning's *n* value and uncertainty in debris loads at the BNSF bridge.

Uncertainty in Roughness

Calibrated Manning's *n* values for the HEC-RAS model in the reach of primary interest downstream from Sedro-Woolley are mostly in the range 0.032 to 0.038 within channel and 0.04 to 0.12 overbank. Estimates of the standard deviation and coefficient of variation of *n* determined from Figure 5.4 of EM-1110-2-1619 are provided in Table 5-13. Assuming a representative average *n* of 0.04 for the reach downstream from Sedro-Woolley, Table 5-13 gives a coefficient of variation of *n* of 0.30. The effects of uncertainty in *n* on stage were evaluated by varying *n* by ± 30% (representing approximately ± one standard deviation per Figure 5.4 of EM-1110-2-1619) and repeating the HEC-RAS modeling for the existing condition scenario for the 500-year flood and for the calibration and validation events .

The effects of varying *n* on the modeled stage-discharge rating at the Skagit River near Mount Vernon gage are shown in Figure 5-8. Simulations with *n* varied by ± 30% result in ratings which encompass all stage-discharge measurements for flows above 40,000 cfs. (The prominent low outlier shown in Figure 5-8 is believed to be a reporting error and is at a discharge below the range of discharges of interest for uncertainty analysis). These simulations, with no bridge debris, similarly encompass all but three of the 40 observed high water marks (92.5% of the HWMS) used for HEC-RAS model calibration and validation. The effects of varying *n* on mainstem water surface profiles are illustrated for the November 1995 validation event in Figure 5-9.

Varying *n* by ± 30% is considered to result in reasonable upper and lower bounds on stage in the absence of debris impacts upstream from the BNSF bridge (see discussion on uncertainty in debris loading below). Since these bounds encompass roughly 95% of the HWMs, the standard deviation of stage uncertainty, absent debris impacts, can be estimated as the stage uncertainty range divided by 4 (see EM-1110-2-1619). For example, for a flow of 140,000 cfs at the Mount Vernon gage, the range of stage is approximately 9 feet (Figure 5-8). If this is assumed to represent 95% of the stage uncertainty range, then the standard deviation of stage uncertainty at the Mount Vernon gage would be approximately 9/4 or 2.25 feet at that discharge. Hydraulic conditions at the Mount Vernon gage site are believed to be reasonably representative of conditions throughout the mainstem Skagit River from the BNSF bridge downstream to the confluence of the North Fork and South Fork.

Stage-discharge ratings for existing conditions with *n* varied by ± 30% (considered as providing upper and lower bounds on stage uncertainty), and with the design bridge debris loads of 6,000 square feet on the BNSF bridge and 4,000 square feet on the Great Northern bridge, are shown for the index points for each damage reach in Figure 5-10. The ratings were developed by plotting stage against discharge for the 500-year event from the model results and then applying a polynomial or spline fit. For locations where the stage-discharge relationship exhibits hysteresis, the ratings in Figure 5-10 are based on the upper (higher stage) portion of the hysteresis loop.

Note that for all index points except that for Damage Reach 8, maximum stages (and discharges) are limited by the capacity of the levee system.

Uncertainty in BNSF Bridge Debris Load

The impact of debris accumulation on the hydraulic performance of the BNSF bridge was described in Chapter 2. A best estimate of a 6,000 square foot debris blockage net of scour was assumed for design purposes and for hydraulic analysis in support of economic flood damage analysis. There is considerable uncertainty in debris blockage estimates in terms of both the gross blockage area and the area net of offsetting scour at the bridge crossing. Lower and upper bounds on net debris blockage of zero and 10,000 square feet were assumed for the purposes of uncertainty analysis. No debris is clearly appropriate for the lower bound based on observations in past flood events. Selection of a 10,000 square foot net blockage upper bound was based on professional judgment.

The effect of uncertainty in debris load on stage upstream from the BNSF bridge was evaluated by repeating HEC-RAS simulations for existing conditions (i.e. existing hydraulic geometry) for the 500-year event but with no debris and with 10,000 square feet of debris. Stage-discharge rating curves for index points upstream from the BNSF bridge were then developed as for the analysis of uncertainty in roughness described above. The resultant rating curves, showing the impacts of uncertainty in BNSF bridge debris, are provided in Figure 5-11.

Under existing conditions, uncertainty in bridge debris affects stage only for flows above about 160,000 cfs and for a distance upstream from the bridge of the order of 3 miles (the BNSF bridge is at XS 17.54). As shown in Chapter 2, a debris blockage of 6,000 square feet has minimal impact on bridge hydraulic performance for flows of less than about 160,000 cfs. For flows above about 150,000 cfs, stage increase due to debris blockage is offset by spill over the DD12 levee and at Sterling, with the great majority of that spill occurring between model XS 17.9 and XS 21.6. Hence, while uncertainty in debris loading has an impact on stage at XS 17.9, there is little impact on stage at and upstream from XS 21.6 (see Figure 5-11). Uncertainty in debris loading would however impact the amount of spill from the system upstream from the BNSF bridge. Note also from Figure 5-11 that for XS 17.9 the stage uncertainty bounds are highly skewed. As such, it would be inappropriate to characterize stage uncertainty due to debris assuming a normal (or log normal) distribution and an estimated standard deviation; stage uncertainty due to uncertainty in debris blockage would be best characterized by means of a triangular distribution.

5.7.3 Estimation of Total Stage-Discharge Uncertainty

Given the relatively modest impact on stage of uncertainty in debris load compared with the assumed uncertainty in roughness (compare relevant panels of Figure 5-10 and Figure 5-11), we recommend adopting the upper and lower stage bounds from Figure 5-10, determined by varying *n* by ± 30%, as representing total stage-discharge uncertainty from all sources for existing conditions. Additional evaluation of stage-discharge uncertainty may be required for other conditions and for flood risk management alternatives.

6 Initial Hydraulic Design of Flood Risk Reduction Alternatives

Initial hydraulic designs were developed for three flood risk reduction alternatives identified by the Project Delivery Team (PDT):

- a setback levee alternative,
- a flood bypass (the Joe Leary Slough Flood Bypass) upstream of Burlington, conveying flood water north into Padilla Bay and, in one variant of the alternative, Samish Bay, and,
- a flood bypass (the Swinomish Flood Bypass) downstream from Burlington, conveying flood water west into Telegraph Slough and Swinomish Slough.

The primary objective for each of the three alternatives is to provide a 100-year level of protection to the urban areas of the cities of Burlington and Mount Vernon.

Initial hydraulic designs were developed for several variants or configurations of each alternative. These initial hydraulic designs were reviewed by the PDT and a single variant or configuration of each of the two flood bypass alternatives was selected by the PDT for more detailed hydraulic analysis to define residual flood risk for use in economic flood damage analysis.

Each of the alternatives developed is assumed to build on the “improved levee” condition described in Section 5.6.

6.1 Setback Levees

This alternative consists of setting back levees on the North Fork, South Fork, and mainstem Skagit River below Mount Vernon, coupled with levee improvements through and upstream from Mount Vernon, and provisions to protect Burlington from right bank spill in the vicinity of Sterling. Numerous configurations were evaluated. The principal project elements considered are shown in Figure 6-1; the preferred configuration of the setback alternative is shown in Figure 6-2. As with all other alternatives, the baseline hydraulic geometry for this alternative is the “improved levee” condition described in Section 5.6.

6.1.1 Primary Design Criteria

The primary design criterion is to reduce peak water levels along urban levee segments to below the Probable Non-Failure Point[5] during the 100-year flood. The Setback Levee alternative seeks to accomplish this by increasing downstream conveyance, thereby lowering flood levels for a given flow. Setbacks alone are unable to meet the design criterion, therefore analysis of the alternative considered various additional measures including improvements to existing levees and a West Mount Vernon Bypass. The Setback Levee alternative was found to be relatively ineffective in reducing water levels

[5] The Probable Non-Failure Point or PNP is defined as the water surface elevation associated with a 15% probability of levee failure. The Probable Failure Point or PFP is defined as the water surface elevation associated with an 85% probability of levee failure. For clarity, these are referred to in this report section as 15% PNP and 85% PFP respectively.

upstream from the BNSF bridge and required not only improvements to existing levees in this area but also allowance for continued right bank spill at Sterling and provisions to protect Burlington from such spill as discussed further in Section 6.1.5 below.

6.1.2 Modeling Methods

All modeling was conducted entirely within HEC-RAS. Setbacks were simulated in the model by extending existing model cross sections in the overbank area. Extensions used existing ground topography. Various widths of setback were simulated by varying the limits of blocked obstructions on the floodplain within HEC-RAS. Overbank roughness values were set at the same values used in the existing conditions model, assuming no change in land cover for areas incorporated into the setback. A few overbank cross sections were modified to reflect assumptions about bridge approach extensions and tidal area excavations as noted in Section 6.1.3 below.

The proposed Burlington Levee, on the right bank Skagit River floodplain, is intended to protect Burlington from right bank spill. The Burlington Levee does not affect the hydraulic performance of the setback levee alternative and is outside the domain of the HEC-RAS model. Hence the Burlington Levee was not modeled for initial hydraulic design purposes.

6.1.3 Project Elements for the Preferred Configuration

Levee setback alignments were selected to generally be implemented on only one bank of the river in a given reach. Small adjustments to alignments were made where avoidance of additional properties or structures was feasible. Project elements described here are for the preferred configuration. Other configurations analyzed are summarized in Section 6.1.4. The preferred configuration was developed through an iterative process starting at the downstream end of the system at Skagit Bay and working upstream, progressively adding project elements in an attempt to meet the project design criterion in the most effective manner.

6.1.3.1 North Fork

The North Fork setback is proposed for the left bank, on Fir Island. Between the head of the North Fork and the Best Road bridge, the setback is approximately 1,000 feet. The approach fill for the bridge is assumed to be removed and replaced with a trestle spanning the setback area to allow unrestricted overbank flow. Just downstream of the bridge, the setback expands significantly, following Browns Slough for some distance before following Hall Slough to the sea dike on Skagit Bay. This lower alignment generally follows the boundary of potential restoration projects proposed for the area. There is a strip of salt marsh outside the sea dike in this area that is significantly higher than interior elevations. Preliminary modeling indicated this restricted flow exiting to Skagit Bay. Therefore, three 1000-foot wide breaches are assumed to be excavated through the marsh. Tidal channels would be excavated or allowed to develop naturally through each breach.

6.1.3.2 South Fork

The South Fork setback is proposed for the right bank, also on Fir Island, and ties into the North Fork setback at the head of the forks. The setback is approximately 1,000 feet wide and extends down the

South Fork to where unconfined channel widths expand significantly. It is assumed that the Fir Island Road bridge is extended on trestles across the setback distance. The active unconfined delta between Fir Island and the mainland has numerous relic dikes on islands throughout the area. While few of them exclude water completely, they still result in blockages to conveyance of floodwater. It is assumed these dikes are completely removed to allow full natural use of the area for conveyance.

6.1.3.3 Mount Vernon to Forks

The levees on the reach of river between the downstream end of Mount Vernon and the forks are setback in several locations already. An additional setback is proposed beginning on the left bank at the terminus of the Mount Vernon Flood Wall (MVFW) project and extending downstream to tie in with the existing levee where it is already set back. This additional setback averages around 600 feet from the existing levee.

6.1.3.4 Other Required Improvements

Levee Improvements Downstream from the BNSF Bridge

The limited reduction in water surface elevation between the BNSF bridge and Mount Vernon achieved through downstream levee setbacks requires that the right bank levees protecting West Mount Vernon and Burlington below the BNSF Bridge be improved to meet the objective of reducing failure probabilities below 15% in the 100-year flood. Similarly, the left bank levee in Riverbend and upstream to the BNSF bridge requires improvement to protect North Mount Vernon.

DD12/Burlington Levee

Improvements are required in the DD12 right bank levees upstream of the BNSF bridge to prevent overtopping and failure into Burlington. Extensive spill will still occur across SR-20 in the Sterling area. Burlington is proposed to be protected from this spill by a "horseshoe" levee extending around the northern side of the City to Interstate-5 (see Figure 6-2). The improvements to the DD12 levees were included in the HEC-RAS simulations by simply raising the levee elevation above the 100-year water surface profile. The Burlington Levee was not modeled in analysis of this alternative. The horseshoe levee alignment shown is for illustration only; additional analysis will be required using the FLO-2D model if this alternative is developed further in the future.

6.1.4 Other Configurations Analyzed

Initial hydraulic modeling of several other combinations of levee setback widths and project elements was conducted. The elements of these configurations are summarized in Table 6-1 and Table 6-2, but results are not presented. Table 6-2 shows the configurations that were modeled to evaluate hydraulic performance in the process of developing the preferred configuration. The preferred configuration is also included in Table 6-2 for completeness

6.1.5 Project Performance

100-year in-channel water surface profiles for the preferred setback levee configuration are shown for the upper mainstem, lower mainstem, and the North and South forks in Figure 6-3 through Figure 6-6. Water surface profiles for the preferred setback configuration are shown for two conditions: with a

6,000 square foot debris blockage on the BNSF bridge (red line), and with no debris on the BNSF bridge (black line). Also shown are the 100-year baseline water surface profile for the "improved levee" condition (described in Section 5.6) and the levee elevations and 15% PNP profiles for the baseline "improved levee" condition. The water surface profiles for the Setback Levee alternative reflect the effects of the project elements discussed in Section 6.1.3 above, including, for example, the levee improvements through and upstream from Mount Vernon.

The levee improvements associated with the baseline condition should not be confused with those required for the Setback Levee alternative; the improvements associated with the baseline condition are local levee raises and corrections to underseepage problems of very limited extent, whereas improvement associated with the Setback Levee alternative would involve substantial levee raises and other improvements over relatively long distances. The extent of improvement required under the preferred Setback Levee configuration can be seen by comparing the Setback Levee water surface profiles against the baseline levee elevation and 15% PNP profiles in Figure 6-3 and Figure 6-4.

With a debris blockage on the BNSF bridge, the Setback Levee alternative reduces peak water levels in the North Fork by up to six feet and in the South Fork by up to three feet. The peak water level on the mainstem near the confluence of the forks is reduced by a little over three feet. Water level reductions taper off upstream to around one foot at the Division Street Bridge and essentially zero by the Anacortes Water Treatment Plant in mid-Riverbend. Upstream of the BNSF bridge, water levels are increased approximately 1.5 feet due to the levee improvements which prevent spill into Burlington over the DD12 levee, which extends up to approximately model River Mile 21.2 on Figure 6-4. Significant spill would occur over SR-20 in the Sterling area (upstream from the DD12 levee) resulting in flooding of a wide area of the right bank floodplain (Figure 6-2).

If it is assumed that there is no debris on the BNSF bridge, the model results change significantly, as shown in Figure 6-3 through Figure 6-6. In the Nookachamps area, water levels are nearly the same as the baseline conditions. Downstream of the BNSF bridge, flows are increased due to improved conveyance through the bridge opening, so water levels increase by approximately two feet above baseline conditions. The increase is around one foot upstream of the Division Street bridge. Downstream of Mount Vernon, the setback alternative's effectiveness increases and water levels decrease compared to baseline conditions despite the higher flows.

A summary of peak flows and water surface elevations at key locations for the 100-year event for the preferred Setback Levee configuration with and without debris on the BNSF bridge is provided in Table 6-3. Also shown in Table 6-3 are data for the 100-year event for the baseline improved levee condition with BNSF bridge debris.

6.1.6 Hydraulic Modeling for Economic Flood Damage Analysis

Due to the relative ineffectiveness of the Setback Levee alternative in reducing water levels along the urban levee segments, particularly upstream from the BNSF bridge, the Project Delivery Team determined that no additional hydraulic modeling of this alternative in support of economic flood damage analysis was warranted.

6.2 Joe Leary Slough Flood Bypass

The Joe Leary Slough Flood Bypass alternative consists of an inlet structure upstream from Burlington, in the Sterling area, a westward overland flood bypass, and outlet structures to Padilla Bay and, in one variant of the alternative, Samish Bay. Three variants of the alternative were evaluated: two confined bypass channels and one partially confined bypass. Under existing conditions, the Sterling area serves as a natural overflow; the combination of downstream levees, the BNSF bridge and the topography in the area means that most flows above approximately 165,000 cfs leave the main river system in this area. The bypass seeks to continue this function, but collect the overflow water and discharge it through a flow corridor rather than allow the current unconfined overflow. In addition to the bypass itself, the alternative also includes a number of new and improved or upgraded levee segments. The various configurations of the alternative are shown in Figure 6-7 through Figure 6-9 as discussed in more detail below.

6.2.1 Primary Design Criteria

The primary design criterion for sizing the bypass is to reduce peak water levels along urban levee segments to below the 15% PNP during the 100-year flood, i.e., to reduce flows downstream from the bypass sufficiently that the need for significant improvements to urban levee segments elsewhere in the system is avoided. Comparison of water surface profiles with the baseline left and right bank 15% PNP profiles shows that targeting a flow of 150,000 cfs downstream of the BNSF bridge largely meets this criteria. However, local improvements would still be needed to parts of the levee system as discussed in Section 6.2.3.4 below.

6.2.2 Modeling Methods

The two confined bypass flow corridor options were modeled entirely within HEC-RAS. The partially confined option was modeled by applying diversion hydrographs developed within HEC-RAS to a FLO-2D model modified to represent the partially confined bypass corridor.

The bypass corridor is crossed by Interstate-5 and the BNSF railroad just north of Burlington. These crossings were included in the hydraulic modeling as they are elevated above surrounding ground and form important controls on upstream water levels within the bypass. No other road crossings were included in the HEC-RAS representation of the two confined bypass options – local roads are generally close to grade and hence do not significantly affect hydraulics. The FLO-2D model for the partially confined option is as described in the Hydraulic Technical Documentation (USACE, 2013b) but modified to include the south bank confining levee as noted above.

6.2.3 Project Elements

6.2.3.1 Inlet Structure

The inlet structure selected consists of twenty five gates, each 36 feet wide, for a total conveyance width of 900 feet. Allowing for stem walls, abutments and guide banks the total structure length would likely be around 950 to 1,000 feet long. The opening invert is set to 35.5 feet. Although referred to as gates, the method for closure of each bay could be true gates, stop logs, or earthen fuse plugs. Each gate is kept closed until a trigger elevation of 46 feet (approximately a 25-year event) is reached by the Skagit River at the structure. The gates are then opened and flows allowed to enter the bypass channel for the duration of the flood.

The inlet structure would be constructed of concrete with riprap blankets extending outwards on either side. The floodplain elevation between the structure and the Skagit River is on average approximately 40 feet compared with a gate invert elevation of 35.5 feet. Excavation of the floodplain on the riverward side of the structure (see inset in Figure 6-7 through Figure 6-9) will be necessary to allow unimpeded flows to reach the structure and to ramp down to the gate invert elevation. The inlet structure is identical for all variants modeled.

6.2.3.2 Bypass Channel

Confined bypass channel alignments were selected to minimize required acquisition of developed properties and to follow natural topography in order to minimize excavation. It is assumed that continued agricultural use of land within the bypass corridor will occur, although winter cover crops may be required to reduce erosion risk during the flood season. Two widths of corridor were modeled: one designed to generally keep velocities under 4 ft/s, and one to keep velocities generally under 6 ft/s.

Excavation of existing ground is proposed at the upper and lower ends of the bypass. Excavation at the upper end is required to match the gate invert elevations and convey water through higher floodplain areas near Sterling. Excavation is also proposed at the lower end where locally higher ground again impedes flow.

All existing roads and railroads are assumed to be left as is within the bypass channel. This implies that for the duration of the bypass discharge, these facilities will be closed, including Interstate-5 and the BNSF railroad. The bypass would operate during 25-year events and larger implying closure of Interstate-5 and the BNSF railway about once every 25 years on average. Under existing conditions, closure of Interstate-5 and the BNSF railway might be expected about once every 50 years on average.

The partially confined bypass channel utilizes the wide variant of the confinement channel from the Skagit River intake structure to Interstate-5 and then continuation of the left (south) bank confinement only, to prevent flood flows from moving south towards Burlington and Skagit Bay.

6.2.3.3 Outlet Structure

The two confined bypass options both terminate at Padilla Bay with an outlet structure at the approximate location of the current outlet of Joe Leary Slough. The outlet structure conceptual design criteria qualitatively balanced outlet capacity with structure cost. The outlet structure must not allow saltwater intrusion back into the bypass channel under post-flood conditions – this prevents using the alternative of a full depth fuse plug type design that would take multiple tide cycles to rebuild.

The proposed design consists of a 1,000-foot long fuse dike segment built on a hardened sill with a gated low level outlet structure set below the sill elevation. The top of the fuse section would be at an elevation of 14 feet to match typical sea dike elevations. The sill, constructed of riprap or concrete, would be at an elevation of 9 feet. This elevation is above MHHW and would allow rebuilding of the fusible section under all but the highest high tides, preventing significant salinity intrusion into the bypass channel. The remainder of the terminus section of the bypass channel would be a standard non-overtopping sea dike design. The proposed design for the low level outlet structure consists of ten flap gates, each 5 feet high and 10 feet wide. The dimensions and elevations were selected based on local topography and existing drainage structures. These gates serve to provide residual floodwater drainage below elevation 9 feet. A subset of these gates would be set at the existing tide gate elevations to provide normal agricultural drainage as exists currently.

As shown in Figure 6-9, the partially confined variant results in flood waters during the 100-year event spilling from the Joe Leary Slough basin into the Samish River basin and discharging into both Padilla Bay and Samish Bay. Outlet structures for this variant consist of two 1,000-foot long fuse dike segments built on hardened sills, one providing for discharge to Padilla Bay and the second to Samish Bay. The top of the fuse section would again be at an elevation of 14 feet to match typical sea dike elevations. The sill, constructed of riprap or concrete, would be at an elevation of 9 feet. It is assumed that a low level gated outlet would be included at each outlet structure to allow for drainage of residual flood waters below elevation 9 feet. On the basis of modeling results for the confined bypass variants, each of these low level outlets are assumed to consist of five flap gates, each 5 feet high by 10 feet wide. The fuse dike segments are included in the FLO-2D model, but because of FLO-2D limitations, the low level outlets are not.

6.2.3.4 Other Required Improvements

DD12/Sterling Levee

The proposed alternative does not reduce maximum flood levels in the Sterling/Nookachamps area enough to prevent overtopping of SR-20 upstream of the inlet structure, therefore a new levee is required paralleling the highway upstream towards Sedro-Woolley. This is the same area that has required flood fighting during large floods historically.

A new levee section would also be required running south from the west end of the inlet structure to tie in to the existing DD12 levee as shown in Figure 6-7 through Figure 6-9. The existing levee in this area would be removed as shown in the inset in Figure 6-7 through Figure 6-9.

Other Levee Improvements/Riverbend Cutoff

Left bank levee baseline 15% PNPs in the DD17 Riverbend area are lower than right bank values. Rather than targeting lower flows to reduce flood levels further, it is assumed that the Riverbend Cutoff Levee is part of this alternative. This separates the levee system into urban and rural protection segments. Some improvements will be necessary to other parts of the river levee in the urban protection area, although the lengths requiring treatment are greatly lessened due to the cutoff levee. The parts of the levee system requiring upgrades (raising of the 15% PNP elevations only) are shown in Figure 6-7 through Figure 6-9 and comprise the left bank DD17 levee between the BNSF and Riverside Drive bridges, and the left bank DD17 levee between the downstream end of the Riverbend Cutoff Levee and Freeway Drive. The crest elevation of this section of levee is assumed unchanged to allow overtopping from the landward side in the event of an upstream breach in the left bank DD17 levee between Interstate-5 and the upstream end of the cutoff levee.

6.2.3.5 Other Design Considerations

Modifications to SR-20 to accommodate the bypass were not explicitly addressed in the modeling. Options include building the bypass intake south of the Highway, in which case the Highway must either be lowered across the upper bypass channel to allow overflow, or span the channel on a bridge. Given the cost of bridging the bypass channel, it is recommended that the intake structure be incorporated into the bridge if this approach is taken. This would require a somewhat longer approach channel but would likely be more cost effective than two independent structures. Elevating SR-20 also offers a relatively short detour route for Interstate-5 during times when the bypass channel is operating.

Sensitivity testing of the inlet parameters showed that gate opening trigger elevations appear to have a greater effect on project performance than structure width or invert elevation for the currently sized structure. Opening the structure at a lower elevation appears to effectively keep more Nookachamps storage available to attenuate the flood peak. As a result, water levels, flows and the resultant amount of required levee improvements could be reduced throughout the system even though bypass channel peak flows are approximately the same. A fully passive inlet system would necessarily be triggered during more frequent floods with a lower trigger elevation. However, with appropriate forecasting knowledge, an inlet requiring human operation, such as a stop log system, gates, or mechanically failed fuse plugs, could be used to limit use of the bypass to less frequent events while still obtaining the increased performance that the lower trigger elevations offer.

6.2.4 Project Performance

100-year water surface elevations and the baseline levee and 15% PNP left bank and right bank profiles are shown in Figure 6-10 and Figure 6-11 for the mainstem reach downstream and upstream of the BNSF bridge respectively, demonstrating that all variants of the bypass alternative come close to meeting the primary design objective without a need for significant improvements to the existing levee system. The water surface profiles for the bypass alternative reflect the effects of all project elements discussed in Section 6.2.3 above. Note that the water surface profiles for the wide bypass and the partially confined variant are identical. Additional no-breach water surface profiles for the mainstem and forks with bridge debris for the wide variant of the alternative are provided in Appendix 6-1.

The Joe Leary Slough Flood Bypass alternative reduces peak 100-year water levels upstream of the BNSF bridge in the Nookachamps area by around 3 feet, and downstream of the BNSF bridge by around 1.5 feet. Areas where segments of the baseline levee do not meet the design objective and are assumed to be upgraded are shown in Figure 6-7 through Figure 6-9. Note also that the majority of the left bank levee (DD17) in the Riverbend area does not serve as an urban area protection feature due to the Riverbend Cutoff Levee; therefore baseline 15% PNP elevations below water surface profiles here are not relevant to meeting the design objective.

Figure 6-12 shows the water surface elevation profile in the bypass channel for the two confined flow variants for the 100-year event. Maximum flow depths reach 20 feet in places. In most areas, the confining levees are set on higher ground and constructed levee heights will not need to be as great as implied by the figure, which shows the lowest ground profile. Also of note is the large effect on water levels due to the elevated I-5 and BNSF railroad crossings. The BNSF railroad is immediately upstream from I-5 and the two embankments are modeled as a single structure in HEC-RAS. Maximum flow depths over the roadway are approximately 4 and 7 feet for the wide and narrow bypass configurations respectively. The road embankment will act as a spillway with a large head drop on the downstream side. Extensive armoring of these facilities will be required to prevent erosion during floods.

Average cross sectional velocities at peak 100-year water levels in the confined bypasses are shown in Figure 6-13. Narrowing the channel increases velocities as expected. The higher velocities in the upper mile of the channel are within the area of excavation where flows are mostly confined in a narrow corridor with higher water surface slopes. Figure 6-14 shows the maximum water surface top width within the corridor for the two variants for the 100-year event.

The 100-year flooded area for the partially confined variant, modeled in FLO-2D, is shown in Figure 6-9. Corresponding grids of water depth, water surface elevation and maximum velocity are included in the digital files accompanying the report.

A summary of peak flows and water levels at key locations for the 100-year event for the wide and narrow variants of the Joe Leary Slough Flood Bypass is provided in Table 6-4.

All results described above, and provided in Table 6-4, are for the 100-year event with a 6,000 square foot debris blockage on the BNSF bridge. The effect of the Joe Leary Slough Flood Bypass is to reduce the 100-year peak flow at the BNSF bridge to approximately 152,000 cfs for the narrow bypass variant and 150,000 cfs for the wide bypass variant. At these flows, the water surface elevation is below the low chord of the bridge and a 6,000 square foot debris blockage has minimal effect on the bridge hydraulic performance (see the detailed BNSF bridge hydraulic analysis described in Chapter 2). Consequently, analysis was not necessary for the 100-year event with no bridge debris.

Following review of the above modeling and analyses, the Project Delivery Team selected the wide confined variant of the Joe Leary Slough Flood Bypass Channel for more detailed modeling to provide information for use in economic flood damage analysis, as described in the report sections below.

6.2.5 Hydraulic Modeling for Economic Flood Damage Analysis

Hydraulic modeling of the Joe Leary Slough Flood Bypass was performed to provide information for economic flood damage analysis following the procedures described in Chapter 5 for the existing condition and improved levee condition. The modeling involved the following basic steps:

- HEC-RAS modeling without levee breaches (no-breach analysis) for the 2- through 500-year floods to identify minimum floods for levee breach modeling.
- HEC-RAS modeling with levee breaches for three floods at each index point. Output from these runs is used as input to FLO-2D to determine flooding extents, depths and water surface elevations over the floodplain.
- FLO-2D modeling for the with-breach scenarios for the three floods per index location.

To accommodate the physical changes associated with the Joe Leary Bypass, revisions were required to the index points and levee failure locations for damage reaches 1 and 5, as follows:

- The levee breach originally associated with Damage Reach 1 is in a section of levee (Layfayette Road) which would be replaced with the construction of the intake structure for the Joe Leary Bypass. A failure at this location with the Joe Leary Bypass alternative in place, is considered unlikely, and, in any event, would probably understate future flood risk in Damage Reach 1 since flows from a breach here would be prevented from flowing north by the confining levee on the south side of the bypass. Consequently, the breach location for Damage Reach 1 was moved upstream to the area known as Sterling Dam (approximately river mile 22.6). The cross-section for reporting no-breach and with-breach modeling results for the associated Index Point was similarly moved upstream from model XS 21.6 to XS 22.27.
- In the existing and improved levee condition HEC-RAS models, damage reaches 5 (River Bend) and 5A (North Mount Vernon) share a common levee failure location and index point immediately downstream from I-5. With the inclusion of the north-south Riverbend Cutoff Levee (see Figure 6-7), a levee failure at this location would cause a breach into Damage Reach 5A only; Damage Reach 5 would be protected from flooding by the cutoff levee. A new index point for Damage Reach 5 was therefore added, with a left bank levee failure location toward the downstream end of the Riverbend, breaching into Damage Reach 5. This breach location was selected on the basis of the estimated 15% PNP profile, which is between the 10-year and 25-year water surface elevation profiles in this reach (see profiles in Appendix 6-1). We note however that a breach further upstream (for example, immediately west of the upstream end of the cutoff levee) would result in a greater depth of flooding in Damage Reach 5 and hence greater damages. Model results for this new index point are reported from XS 13.8.

The revised index points for analysis of the Joe Leary Slough Flood Bypass alternatives are shown in Figure 6-15. The original index points were shown in Figure 5-1.

A number of points should be noted with respect to the HEC-RAS and FLO-2D models used for hydraulic modeling for economic flood damage analysis:

- The confining levees for the Joe Leary Bypass are included in the HEC-RAS and FLO-2D models as “infinite height” levees for modeling convenience. Actual levee heights should be determined from the modeled bypass water surface profiles with appropriate consideration for uncertainty. The implications for flood damage analysis of using “infinite height” confining levees are discussed in Section 6.2.5.2.
- The crest elevations for the new river levees upstream and downstream from the bypass intake structure (see Figure 6-7) were set at approximately 3 feet above the 100-year water surface profile with a transition to existing high ground at the upstream end. The assumed levee profile can be seen in the no-breach water surface profiles provided in Appendix 6-1 and discussed in Section 6.2.5.1 below.

- The Riverbend Cutoff Levee was only included in the HEC-RAS model geometry for modeling of levee breaches into Damage Reaches 5 and 5A. This geometry also included the proposed closure structure across the BNSF railway tracks at the northern end of the Mount Vernon Flood Wall which had been omitted from the existing condition and improved levee models. In the HEC-RAS model, this closure shuts off flow from Damage Reach 5A (North Mount Vernon) to Damage Reach 4A (Mount Vernon), resulting in a minor increase (of the order of 0.1 feet in a 100-year event) in water levels in Damage Reach 5A.
- All HEC-RAS modeling for economic flood damage analysis was performed with debris blockages of 6,000 square feet on the BNSF bridge and 4,000 square feet on the Great Northern bridge as in the existing condition and improved levee condition simulations.

6.2.5.1 No-Breach Analysis

The HEC-RAS model with the wide variant of the Joe Leary Slough Flood Bypass was run without levee breaches but allowing levee overtopping for the existing regulation 2- through 500-year floods. Flow-frequency curves, stage-discharge curves and water surface profiles are provided in Appendix 6-1, and peak discharge quantiles are provided in Table 6-5. Also shown in Table 6-5 are corresponding flood quantiles for the bypass flows.

The differences between no-breach flood quantiles with the Joe Leary Slough Flood Bypass (Table 6-5) and no-breach quantiles for the baseline improved levee condition are provided in Table 6-6. The net result of the bypass is a significant decrease in peak flows downstream of the bypass (at approximately model XS 21.6) during the 25- through 100-year events, with lesser reductions for events larger than the 100-year event, and no appreciable change in peak flows for events smaller than the 25-year event. For floods larger than the 100-year event, water overtops the levee system in many locations with or without the bypass, so in-channel peak flows downstream from the bypass are largely unaffected and are limited to the within-levee channel capacity. The bypass is not activated for events smaller than a 25-year event, hence peak flows for those events are essentially unchanged. The effect of the bypass is also seen in the water surface profiles shown in Appendix 6-1, in which the 25- through 100-year profiles (the events in which the bypass is active, and overtopping elsewhere in the system is limited) are all contained within a narrow band.

6.2.5.2 With-Breach Analysis

With-breach analysis for the Joe Leary Slough Flood Bypass alternative was conducted in a similar manner to the existing condition and improved levee condition with-breach analysis, but with adjustments to accommodate the specific hydraulic characteristics of the flood bypass alternative. The adjusted levee breach data, with revised index point and levee breach locations for damages reaches 1 and 5, differing breach flood events, and differing breach trigger elevations are shown in Table 6-7.

With-breach modeling of the bypass necessitated some significant changes to the breach flood events and breach trigger elevations compared with those used for the baseline improved levee scenario. As the bypass diverts water, some prior breach elevation triggers are never reached with the bypass in place. Revised trigger elevations were selected to correspond to PNP or LFP elevations when possible, but in some instances the 100-year flood with the bypass failed to reach even the PNP elevation. In these instances, the 100-year event was taken as the minimum flood for modeling of with-breach conditions, with the 100-year water surface elevation assumed as the minimum breach trigger elevation.

For each index point, HEC-RAS simulations were performed with a levee breach for the three flood events indicated in Table 6-7. The peak flows and peak in-channel water levels at each index point are summarized in Table 6-8. Peak flows and water levels at index points associated with a levee failure location are for the scenario with a breach at that failure location. As in the improved levee condition with-breach analyses, peak in-channel flows and stages at several locations (highlighted in Table 6-8) occur on the rising limb of flood hydrographs immediately before the triggering of a downstream levee breach. Flows and stages reported are for peak post-breach stages which are expected to be better related to maximum flood extent and depth in the associated damage reach.

As in the improved levee condition with-breach analyses, breach hydrographs and levee overtopping hydrographs from HEC-RAS were written to a HEC-DSS data base and then used as input to FLO-2D. The FLO-2D and HEC-RAS outputs were then merged to produce grids of maximum flood depth and maximum water surface elevation.

As noted earlier, the confining levees for the Joe Leary Bypass are included in the HEC-RAS and FLO-2D models as "infinite height" levees. This results in some local overestimation of the modeled depth on the floodplain where, with a more realistic confining levee height, floodplain flows would spill from the floodplain over the bypass confining levee and into the bypass. To test the significance of this issue, a confining levee crest elevation was set at 3 feet above the 100-year bypass water surface profile and compared against FLO-2D water surface elevations on the adjacent flood plain for various scenarios with "infinite height" levees.

The "worst case" scenario for potential overtopping from the floodplain into the bypass occurs for the 500-year + 2 standard deviation flood with a breach at Sterling Dam into Damage Reach 1. In this scenario, the bypass north confining levee would be overtopped by between 1 and 1.5 feet from Interstate-5 upstream to the intake. Minor overtopping of the north confining levee also occurs between the intake and Sterling Hill for the 100-year and 250-year floods with a breach at Sterling Dam and for the 500-year + 2 standard deviation flood with no breach. Minor overtopping of the western end of the southern confining levee (just east of Bayview Ridge) also occurs in the 500-year + 2 standard deviation flood as a result of overtopping of the DD12 levee downstream from the bypass intake.

Full results including the HEC-RAS model, levee breach and overtopping hydrographs, FLO-2D model and GIS shape files of maximum flood depths and maximum water surface elevations are provided in the digital deliverables for the study.

6.3 Swinomish Flood Bypass

The Swinomish Flood Bypass consists of an inlet structure on the right bank of the Skagit River downstream of Interstate-5, a westward overland flood bypass generally paralleling SR-20, and an outlet structure to the Swinomish Channel. Three variants of the alternative were evaluated: two confined bypass channels and one unconfined bypass (i.e., allowing for spill onto the floodplain at the inlet structure, but without confining levees). The Swinomish bypass seeks to remove sufficient flow from the main channel that water levels, and hence failure risk, are reduced substantially along urban levees. In addition to the bypass itself, the alternative also includes a number of new and improved or upgraded levee segments. The various configurations of the alternative are shown in Figure 6-16 through Figure 6-18 and discussed in more detail below.

6.3.1 Primary Design Criteria

The primary design criterion for sizing the bypass is to reduce peak water levels along urban levee segments to below the 15% PNP during the 100-year flood, such that the need for significant improvements to urban levee segments elsewhere in the system is minimized. Modeling indicates this objective cannot be achieved with any of the three bypass configurations alone, and significant improvements would still be needed to parts of the levee system as discussed in Section 6.3.3.4 below.

6.3.2 Modeling Methods

The two confined bypass flow corridor variants were modeled entirely within HEC-RAS by modifying the baseline improved levee HEC-RAS model described in Section 5.6. The unconfined variant was modeled by applying diversion hydrographs developed within HEC-RAS to the FLO-2D model of the overbank area. The FLO-2D model, described in the Hydraulic Technical Documentation (USACE 2013b), was modified to include additional protection for Burlington as described in Section 6.3.3.4 below.

The inlet structure was modeled using levee breach routines to simulate operation of a fuse plug type structure. Road crossings of the bypass corridor were assumed to either be on-grade or trestles that caused negligible losses and were therefore not modeled explicitly. The outlet structure was modeled as an in-line structure with an overflow weir section and low-flow flap-gated culverts.

Modeling was conducted for two debris assumptions (no debris and 6,000 square feet of debris) at the BNSF bridge. The debris assumptions affect both water levels and flows in the system. For design purposes, the no-debris case, which maximizes flows downstream of the BNSF bridge, was used for bypass performance evaluation. The with-debris case, which results in higher water levels in the Nookachamps, will be used in designing required levee improvements upstream from the BNSF bridge. The with-debris case was also used for modeling conditions both upstream and downstream of the BNSF bridge to provide data for economic flood damage analysis.

6.3.3 Project Elements

6.3.3.1 Inlet Structure

The inlet structure selected was based on prior Skagit GI study concepts of the bypass. It consists of three identical fuse plugs, with a total bottom width of 800 feet. The individual fuse plugs are triggered sequentially when water surface elevations in the channel reach 38.3, 38.5, and 38.7 feet. These trigger elevations are between the 10- and 25-year flood event water surface elevations. Upon triggering, the fuse plugs are allowed to erode down to an elevation of 28 feet.

The inlet structure would be constructed of concrete with riprap blankets extending outwards on either side. Each fuse plug section would have an easily erodible soil plug placed on the concrete sill. The inlet structure is identical for all variants modeled.

6.3.3.2 Bypass Channel

Confined bypass channel alignments followed those used in previous analyses. The alignments were selected to minimize corridor areas and lengths required and to follow natural topography in order to minimize excavation. It is assumed that continued agricultural use of land within the bypass corridor will occur, although winter cover crops may be required to reduce erosion risk during flood season. Two widths of corridor were modeled: a roughly 2,000 to 3,000-foot wide bypass designed to generally keep velocities under 4 ft/s, and an approximately 1,000-foot wide bypass designed to keep velocities generally under 6 ft/s.

A 100-foot wide channel is assumed to be excavated through the bypass corridor. The channel ranges from 7 to 10 feet deep and is intended to provide post-flood drainage of the corridor and a source of borrow material for levee construction.

The unconfined bypass uses the same inlet structure but then allows unrestricted spread of floodwater across the floodplain.

6.3.3.3 Outlet Structure

The two confined bypass options both terminate at the Swinomish Channel. It is assumed the area between the outlet structure and the Swinomish Channel has been restored and is under tidal influence. The outlet structure must not allow saltwater intrusion back into the bypass channel under post-flood conditions – this precludes using the alternative of a full depth fuse plug type design that would take multiple tide cycles to rebuild.

The proposed design consists of a 1,000-foot long fuse dike segment built on a hardened sill with a gated low level outlet structure set below the sill elevation. The top of the fuse section would be at an elevation of 14 feet to match typical sea dike elevations. The sill, constructed of riprap or concrete, would be at an elevation of 9 feet. This elevation is above MHHW and would allow rebuilding of the fusible section under all but the highest high tides, preventing significant salinity intrusion into the bypass channel. The remainder of the terminus section of the bypass channel would be a standard non-overtopping sea dike design. The proposed design for the low level outlet structure consists of five flap gates, each 5 feet high and 6 feet wide. The dimensions and elevations were selected based on local topography and existing drainage structures. These gates serve to provide residual floodwater drainage below elevation 9 feet.

No outlet structures were modeled for the unconfined variant. It is likely that construction of a series of smaller outlet structures at various locations along Skagit Bay, Padilla Bay and the Swinomish Channel would be required to provide effective flood drainage for this variant.

6.3.3.4 Other Required Improvements

DD12/Burlington Levee

The Swinomish Flood Bypass is not effective in reducing water levels to the target baseline 15% PNP elevations upstream from the BNSF bridge. This is especially the case with the assumed 6,000 sq. ft. debris blockage on the BNSF bridge, for which upstream water level reductions due to the bypass are minimal (see discussion of project performance in Section 6.3.4 below). As a result, the DD12 levees from the BNSF upstream to SR-20 must be improved to prevent overtopping and to raise the 15% PNP elevations.

It is assumed that the existing ground elevation along SR-20 and in the Sterling area will remain unchanged, thereby allowing continued spill onto the right bank flood plain in the vicinity of Sterling (with or without bridge debris) and the potential for continued flooding of Burlington. Burlington would be protected from this spill by a "horseshoe" Burlington Levee extending around the northern side of the City to Interstate-5 (see Figure 6-16 and Figure 6-17).

The improvements to the DD12 levees upstream from the BNSF bridge and the Burlington "horseshoe" levee were both included in modeling of the Swinomish Flood Bypass alternative. The improvements to the DD12 levees were included in the HEC-RAS simulations by simply raising the levee elevation above the 100-year water surface profile. The Burlington "horseshoe" levee was included in the FLO-2D floodplain model. The "horseshoe" levee alignment chosen was based on preliminary modeling; additional analysis will be required using the FLO-2D model if this alternative is to be developed further in the future.

Other Levee Improvements

Some improvements will be necessary to other parts of the river levee in the urban protection area to raise the 15% PNP elevation above the 100-year water surface profile. These are the left bank DD17 levee between the BNSF and Riverside Drive bridges, and section of the left bank DD17 levee toward the downstream end of the Riverbend area. The parts of the levee system requiring improvement are shown in Figure 6-16 through Figure 6-18.

6.3.4 Project Performance

Mainstem 100-year water surface elevations and the baseline levee and 15% PNP left bank and right bank profiles are shown in Figure 6-19 and Figure 6-20 for the mainstem reach downstream and upstream of the BNSF bridge, respectively. The water surface profiles shown are for conditions without bridge debris for the lower mainstem profile (Figure 6-19) and with bridge debris for the upper mainstem profile (Figure 6-20). (Project performance evaluation is based on the no-debris condition downstream from the BNSF bridge and with-debris upstream). The width and trigger elevations of the fuse plug control the amount of water diverted into the bypass. The width of the bypass itself has no effect on the flow entering, at least within the range of widths studied herein. Therefore, flows in the bypass are the same for the wide and narrow variants, and the reductions in mainstem flows are likewise independent of bypass width. As a result, the mainstem water surface profiles are essentially identical for all variants of the bypass. Peak flows and water levels for the 100-year event are shown for key locations in Table 6-9. Additional no-breach water surface profiles for the mainstem and forks with bridge debris are provided in Appendix 6-2.

For the Swinomish Flood Bypass alternative, the largest reductions in water level occur downstream of the BNSF bridge. A maximum reduction of about 4 feet is achieved in the 100-year event at the diversion point, and reductions of 1.4 to 1.9 feet depending on the debris assumption persist downstream to the confluence of the forks. Similar reductions are observed in both forks until near their mouths.

The alternative is less effective in reducing water levels in the Nookachamps area due to the influence of the BNSF bridge, with or without debris. Water level reductions range from a maximum of 0.8 feet with debris and 2.3 feet without just upstream of the bridge, to less than 0.5 feet at the upper end of the Nookachamps near SR-9. Under the with-debris case, most of the right bank DD12 levee (under the baseline condition) and SR-20 continue to overtop at significant depth. Without debris, water levels are lowered to around the crest of the DD12 levee (again under the baseline condition). Overtopping continues across SR-20 at Sterling with or without debris on the BNSF bridge.

Figure 6-21 shows the maximum 100-year water surface profile in the bypass channel for the two confined flow variants with no debris on the BNSF bridge. Bypass channel water surface profiles with bridge debris are up to about 0.5 feet lower. Maximum flow depths reach 20 feet in places, although this is measured from the bottom of the excavated drainage channel. In most areas, depths of flow against the confining levees are on the order of 10 feet. The profiles indicate that the outlet structure acts as a control on water level for the lower 2 to 2.5 miles of the wide variant. The narrow variant (which has the same width as the outlet structure) has a more consistent water surface slope, indicating that for this configuration, water levels in the lower part of the bypass are controlled more by overall channel geometry than by the hydraulic capacity of the outlet structure.

Average cross sectional velocities at peak 100-year water levels in the confined bypasses are shown in Figure 6-22. Narrowing the channel increases velocities as expected. Figure 6-23 shows the 100-year top width of the water surface within the corridor for the two variants.

The 100-year flooded area for the unconfined variant, modeled using the existing condition FLO-2D model with the addition of the Burlington horseshoe levee, is shown in Figure 6-18 assuming no debris on the BNSF bridge. The distribution of flooding would be slightly different with bridge debris, with greater spill upstream from the BNSF bridge and lower discharges on to the flood plain via the bypass intake structure.

Following review of the above modeling and analyses, the Project Delivery Team selected the wide variant of the Swinomish Flood Bypass Channel for more detailed modeling to provide information for use in economic flood damage analysis, as described in the report sections below.

6.3.5 Hydraulic Modeling for Economic Flood Damage Analysis

Hydraulic modeling of the Swinomish Flood Bypass was performed to provide information for economic flood damage analysis following the procedures described in Chapter 5 for the existing condition and improved levee condition. The modeling involved the following basic steps:

- HEC-RAS modeling without levee breaches (no-breach analysis) for the 2- through 500-year floods to identify minimum floods for levee breach modeling.
- HEC-RAS modeling with levee breaches for three floods at each index point. Output from these runs is used as input to FLO-2D to determine flooding extents, depths and water surface elevations over the floodplain.
- FLO-2D modeling for the with-breach scenarios for the three floods per index location.

Index points and associated levee breach locations are as for the existing condition and improved levee condition scenarios.

The following points should be noted with respect to the HEC-RAS and FLO-2D models used for hydraulic modeling for economic flood damage analysis:

- The crest elevations for the DD12 levee upstream from the BNSF bridge were assumed to be raised 3 feet above the with-alternative 100-year water surface profile. The assumed levee profile can be seen in the no-breach water surface profiles provided in Appendix 6-2. 15% PNP elevations for the raised levee were assumed 2 feet below the levee crest as in existing condition and improved levee condition modeling.
- The Burlington "horeshoe" levee was represented in the FLO-2D model as having a crest elevation 3 feet above the 100-year floodplain water surface elevation immediately north of the levee from the with-alternative with-debris no-breach scenario. This levee is overtopped from the north into Burlington during the 500-year + 2 standard deviation flood. For modeling purposes it is assumed that the levee does not breach during overtopping.
- In a departure from the modeling approach used for initial hydraulic design, the Swinomish Flood Bypass was included in the FLO-2D model for the purposes of hydraulic modeling for economic flood damage analysis. This was done to model the impacts of the east-west bypass on the north-south movement of water across the floodplain. The crest elevations for the north and south confining levees for the bypass were assumed to be 3 feet above the 100-year water surface profile within the bypass from the design scenario with no debris at the BNSF bridge. Water spilling out of the system at Sterling flows west and south around (and in larger events through) Burlington and spills into the bypass over its north confining levee. In the 500-year + 2SD event, the bypass capacity is exceeded and flow spills over the south confining levee to flood the area to the south and ultimately discharge into Skagit Bay. Again, it is assumed that the bypass confining levees do not breach during overtopping. Some flood water trapped on the north side of the north confining levee flows west, ultimately discharging into Padilla Bay.
- All HEC-RAS modeling for economic flood damage analysis was performed with debris blockages of 6,000 square feet on the BNSF bridge and 4,000 square feet on the Great Northern bridge as in the existing condition and improved levee condition simulations.

6.3.5.1 No-Breach Analysis

The HEC-RAS model with the Swinomish Flood Bypass was run without levee breaches but allowing levee overtopping for the existing regulation 2- through 500-year floods. Flow-frequency curves, stage-discharge curves and water surface profiles are provided in Appendix 6-2, and peak discharge quantiles are provided in Table 6-10. Also shown in Table 6-10 are corresponding flood quantiles for the bypass flows.

The differences between no-breach flood quantiles with the Swinomish Flood Bypass (Table 6-10) and no-breach quantiles for the baseline improved levee condition are provided in Table 6-11. The bypass is activated for 25-year floods and larger. The net result of the bypass is a significant decrease in peak flows downstream from the bypass and an increase in peak flows upstream from the bypass to the Nookachamps due to the drawdown at the bypass intake and the resultant increase in upstream water surface slope. Within the Nookachamps reach, for example at XS 21.6, the model shows an increase in peak flows for the 50-and 75-year events due to increased water surface slope, but a decrease for larger events. The decrease in peak flows at XS 21.6 during the larger events is due to the raising of the DD12 levees which reduces right bank spill between XS 21.6 and the next downstream index point with flow reported at XS 17.9. The effect of the bypass can also be seen in the water surface profiles shown in Appendix 6-2.

6.3.5.2 With-Breach Analysis

With-breach analyses for the Swinomish Flood Bypass alternative were conducted in a similar manner to the existing condition and improved levee condition with-breach analyses, but with adjustments to accommodate the specific hydraulic characteristics of the alternative. The adjusted levee breach data, differing breach flood events, and differing breach trigger elevations are shown in Table 6-12.

As with the Joe Leary Bypass, the with-breach modeling of the Swinomish Flood Bypass necessitated some significant changes to the breach flood events and breach trigger elevations. As the bypass diverts water, some prior breach elevation triggers are never reached with the bypass in place. Revised trigger elevations were selected to correspond to PNP or LFP elevations when possible, but in some instances the 100-year flood with the bypass failed to reach even the PNP elevation. In these instances, the 100-year event was taken as the minimum flood for modeling of with-breach conditions, with the 100-year water surface elevation assumed as the minimum breach trigger elevation.

For each index point, HEC-RAS simulations were performed with a levee breach for the three flood events indicated in Table 6-12. The peak flows and peak in-channel water levels at each index point are summarized in Table 6-13. Peak flows and water levels at index points associated with a levee failure location are for the scenario with a breach at that failure location. As in the with-breach analyses for other scenarios, peak in-channel stages at several locations (highlighted in Table 6-13) occur on the rising limb of flood hydrographs immediately before the triggering of a downstream levee breach. Stages reported are peak post-breach stages, which are expected to be better related to maximum flood extent and depth in the associated damage reach.

As in other with-breach analyses, breach hydrographs and levee overtopping hydrographs from HEC-RAS were written to a HEC-DSS data base and then used as input to FLO-2D. The FLO-2D and HEC-RAS outputs were then merged to produce grids of maximum flood depth and maximum water surface elevation.

Full results including the HEC-RAS model, levee breach and overtopping hydrographs, FLO-2D model and GIS shape files of maximum flood depths and maximum water surface elevations are provided in the digital deliverables for the study.

7 References

Cherry, S. and Jackson, G., 2002. Geomorphology and Sediment Transport Study of Skagit River Flood Hazard Mitigation Project -Skagit County, Washington: Phase 1 INTERIM REPORT. Prepared for U.S. Army Corps of Engineers, Seattle District, by Pentec Environmental. December 20, 2002.

Curran, C.A., Grossman, E.E., and Mastin, M.C., 2009. Measurements of suspended sediment and flow distribution with implications for habitat restoration in the Skagit River Delta, Washington, 2009 Puget Sound Georgia Basin Ecosystem Conference, February 8-11, 2009. Obtained at http://puget.usgs.gov/posters/Curran+Grossman+Mastin.Meas-Susp-Sed_poster.pdf .

Tetra Tech, 2008. Baker River Project Part 12 Probable Maximum Flood Study, Final Report. Report prepared for Puget Sound Energy, Bellevue, Washington.

U.S. Army Corps of Engineers, 1987. Management of Water Control Systems, EM 1110-2-3600. Washington, D.C.

U.S. Army Corps of Engineers, 1996. Risk-Based Analysis for Flood Damage Reduction Studies, EM 1110-2-1619. Washington, D.C.

U.S. Army Corps of Engineers, 1998. HEC-FDA Flood Damage Reduction Analysis User's Manual, Hydrologic Engineering Center, Davis, California.

U.S. Army Corps of Engineers, 2000. Water Control Manual. Baker River Project, Baker River, Washington. June 2000.

U.S. Army Corps of Engineers, 2001. Water Control Manual. Skagit River Project, Skagit River, Washington. June 2001.

U.S. Army Corps of Engineers, 2004a. Draft Report, Hydraulic Technical Documentation, Skagit River Basin, Skagit River Flood Damage Reduction Feasibility Study, Seattle District. May 2004.

U.S. Army Corps of Engineers, 2004b. Draft Report, Hydrology Technical Documentation, Skagit River Basin, Skagit River Flood Damage Reduction Feasibility Study, Seattle District. August 2004.

U.S. Army Corps of Engineers, 2011a. Draft Report, Hydrology Technical Documentation, Skagit River Basin, Skagit River Flood Damage Reduction Feasibility Study, Seattle District. March 2011.

U.S. Army Corps of Engineers, 2011b. Draft Report, Hydraulic Technical Documentation, Skagit River Basin, Skagit River Flood Damage Reduction Feasibility Study, Seattle District. April 2011.

U.S. Army Corps of Engineers, 2013a. Hydrology Technical Documentation, Skagit River Basin, Skagit River Flood Risk Management Feasibility Study, Seattle District. August 2013.

U.S. Army Corps of Engineers, 2013b. Hydraulic Technical Documentation, Skagit River Basin, Skagit River Flood Risk Management Feasibility Study, Seattle District. August 2013.

TABLES

Table 1-1: Correspondence between HEC-RAS model River Miles and actual stationing

Location	Model Cross-Section RM	Actual Stationing		Model Cross-Section RM	Actual Stationing		Model Cross-Section RM	Actual Stationing
Mainstem Skagit River			North Fork			South Fork		
	23.2	24.387		941	9.410		941	9.412
	22.5	23.759		940	9.406		940	9.408
	22.4	23.645		925	9.243		925	9.283
u/s Great Northern RR Bridge	22.39	23.634		914.6	9.156		916.5	9.202
	22.38	23.629		900.4	9.038		907.8	9.120
	22.3	23.603		885	8.909		895.5	9.004
	22.29	23.592		879.9	8.855		883.5	8.891
d/s Hwy 9 bridge	22.28	23.585		865.4	8.702		875	8.811
	22.27	23.575		854.2	8.583		865.7	8.705
	22.2	22.958		847.1	8.508		860.7	8.647
	22	22.421		839.8	8.431		852.4	8.553
	21.93	22.008		833	8.359		837.92	8.386
	21.6	21.507		829	8.317		823.44	8.219
	20.9	20.882		819.5	8.194		808.96	8.052
	20	19.989		810	8.072		794.48	7.885
	19.48	19.548		800	7.954		780	7.718
	18.57	18.713		790	7.835		765.833	7.598
	17.9	18.165		775.75	7.706		751.666	7.477
	17.56	17.705		761.5	7.577		737.5	7.357
u/s BNSF bridge	17.55	17.680		747.25	7.448		723.333	7.236
	17.53	17.669		733	7.318		709.166	7.116
	17.52	17.644		726.5	7.229		695	6.995
	17.08	17.204		720	7.140		684.166	6.879
u/s Riverside Drive bridge	17.07	17.188		708	7.026		673.333	6.764
	17.05	17.169		696	6.911		662.5	6.648
USGS gage	17.04	17.119		684	6.796		651.666	6.533
	16.82	16.958		672	6.681		640.833	6.417
u/s Interstate-5 bridge	16.81	16.930		660	6.566		630	6.302
	16.79	16.916		646.666	6.466		615	6.164
	16.78	16.890		633.333	6.366		600	6.025
	16.6	16.625		620	6.265		585	5.887
	16.3	16.307		606.666	6.126	u/s Fir Island Road bridge	581	5.811
	15.9	15.939		593.333	5.986		579	5.800
Anacortes WTP	15.1	15.213	u/s Best Road bridge	580	5.847		565.5	5.682

Table 1-1 (cont.): Correspondence between HEC-RAS model River Miles and actual stationing

Location	Model Cross-Section RM	Actual Stationing		Model Cross-Section RM	Actual Stationing		Model Cross-Section RM	Actual Stationing
Mainstem Skagit River			North Fork			South Fork		
	14.6	14.909		570	5.821		552	5.563
	14	14.343		563.333	5.744		538.5	5.445
	13.8	13.929		556.666	5.667		525	5.326
	13.1	13.134		550	5.590		513	5.183
	13.05	13.068		540.666	5.490		501	5.040
	13	12.894		531.333	5.391		489	4.897
	12.96	12.784		522	5.291		477	4.754
u/s Division Street bridge	12.95	12.779		510.25	5.168		465	4.611
	12.93	12.773		498.5	5.046		440	4.478
	12.92	12.766		486.75	4.923		415	4.346
	12.4	12.340		475	4.800		390	4.214
	11.7	11.674		466.666	4.710		365	4.082
	11.2	11.153		458.333	4.620		340	3.950
	10.6	10.781		450	4.529		325	3.818
	10.55	10.714		440	4.417		310	3.686
	10.51	10.660		430	4.304		295	3.554
	10.45	10.574		417	4.209		280	3.422
	10.39	10.497		406	4.096		265	3.290
	10.35	10.435		390	4.001		250	3.158
	10.31	10.383		380	3.932		230	2.400
	10.28	10.340		370	3.862		210	1.643
	10.23	10.277		360	3.793		190	0.885
	10.18	10.203		305	3.414		170	0.128
	10.14	10.157		266.25	2.940		150	-0.630
	10.1	10.101		227.5	2.467			
	10.06	10.064		188.75	1.993			
	10.02	10.019		150	1.520			
	9.97	9.972						
	9.9	9.897						
	9.76	9.758						
	9.62	9.626						
	9.56	9.558						
	9.49	9.491						
Confluence	9.48	9.488						

Table 2-1: Selected BNSF Bridge Hydraulic Model Output Assuming No Scour

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
Base	20OCT2003 0600	37.17	36.80	Energy only	94615		94615	8.8
Base	20OCT2003 0700	37.31	36.92	Energy only	95515		95515	8.8
Base	20OCT2003 0800	37.44	37.05	Energy only	96434		96434	8.8
Base	20OCT2003 0900	37.58	37.19	Energy only	97391		97391	8.9
Base	20OCT2003 1000	37.75	37.36	Energy only	98689		98689	8.9
Base	20OCT2003 1100	37.98	37.58	Energy only	100433		100433	9.0
Base	20OCT2003 1200	38.24	37.83	Energy only	102278		102278	9.1
Base	20OCT2003 1300	38.62	38.20	Energy only	105269		105269	9.3
Base	20OCT2003 1400	39.00	38.57	Energy only	108169		108169	9.4
Base	20OCT2003 1500	39.45	39.01	Energy only	111632		111632	9.5
Base	20OCT2003 1600	39.92	39.47	Energy only	115271		115271	9.7
Base	20OCT2003 1700	40.33	39.87	Energy only	118052		118052	9.7
Base	20OCT2003 1800	40.58	40.12	Energy only	119749		119749	9.7
Base	20OCT2003 1900	40.87	40.40	Energy only	122022		122022	9.8
Base	20OCT2003 2000	41.22	40.73	Energy only	124829		124829	9.8
Base	20OCT2003 2100	41.87	41.36	Energy only	130956		130956	10.0
Base	20OCT2003 2200	42.64	42.11	Energy only	137473		137473	10.2
Base	20OCT2003 2300	43.42	42.87	Energy only	144060		144060	10.3
Base	20OCT2003 2400	44.20	43.62	Energy only	150994		150994	10.4
Base	21OCT2003 0100	44.98	44.38	Energy only	158324		158324	10.5
Base	21OCT2003 0200	45.88	45.29	Energy only	162146		162146	10.6
Base	21OCT2003 0300	46.94	46.37	Press/Weir	164504	56	164560	9.5
Base	21OCT2003 0400	48.08	47.52	Press/Weir	167572	78	167650	9.7
Base	21OCT2003 0500	49.28	48.75	Press/Weir	171049	106	171154	9.9
Base	21OCT2003 0600	50.35	49.75	Press/Weir	186599	334	186933	10.8
Base	21OCT2003 0700	51.07	50.34	Press/Weir	209355	974	210330	12.1
Base	21OCT2003 0800	51.77	50.96	Press/Weir	223182	1635	224817	12.9
Base	21OCT2003 0900	52.41	51.53	Press/Weir	235968	2700	238668	13.6
Base	21OCT2003 1000	53.00	52.05	Press/Weir	246403	5369	251772	14.2
Base	21OCT2003 1100	53.54	52.51	Press/Weir	255444	8336	263780	14.7
Base	21OCT2003 1200	54.01	52.92	Press/Weir	263436	11482	274918	15.2
Base	21OCT2003 1300	54.42	53.28	Press/Weir	270256	14565	284820	15.6
Base	21OCT2003 1400	54.76	53.57	Press/Weir	276064	17421	293485	15.9
Base	21OCT2003 1500	55.05	53.81	Press/Weir	280773	19993	300765	16.2
Base	21OCT2003 1600	55.29	54.02	Press/Weir	284280	21716	305996	16.4
Base	21OCT2003 1700	55.47	54.18	Press/Weir	286897	23171	310068	16.5
Base	21OCT2003 1800	55.60	54.30	Press/Weir	288634	24283	312917	16.6
Base	21OCT2003 1900	55.69	54.37	Press/Weir	289742	25008	314750	16.7

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
0debris	20OCT2003 0600	36.93	36.53	Energy only	95523		95523	5.1
0debris	20OCT2003 0700	37.04	36.65	Energy only	96295		96295	5.1
0debris	20OCT2003 0800	37.17	36.77	Energy only	97188		97188	5.1
0debris	20OCT2003 0900	37.30	36.89	Energy only	98135		98135	5.1
0debris	20OCT2003 1000	37.44	37.03	Energy only	99150		99150	5.1
0debris	20OCT2003 1100	37.59	37.18	Energy only	100377		100377	5.2
0debris	20OCT2003 1200	37.80	37.38	Energy only	102031		102031	5.2
0debris	20OCT2003 1300	38.11	37.68	Energy only	104554		104554	5.3
0debris	20OCT2003 1400	38.53	38.08	Energy only	108074		108074	5.3
0debris	20OCT2003 1500	39.00	38.54	Energy only	111744		111744	5.4
0debris	20OCT2003 1600	39.48	39.00	Energy only	115523		115523	5.5
0debris	20OCT2003 1700	39.97	39.49	Energy only	119485		119485	5.5
0debris	20OCT2003 1800	40.29	39.80	Energy only	121391		121391	5.5
0debris	20OCT2003 1900	40.56	40.07	Energy only	123525		123525	5.6
0debris	20OCT2003 2000	40.89	40.38	Energy only	126242		126242	5.6
0debris	20OCT2003 2100	41.32	40.79	Energy only	130203		130203	5.7
0debris	20OCT2003 2200	42.13	41.57	Energy only	137713		137713	5.8
0debris	20OCT2003 2300	42.91	42.33	Energy only	144586		144586	5.9
0debris	20OCT2003 2400	43.69	43.09	Energy only	151729		151729	6.0
0debris	21OCT2003 0100	44.48	43.85	Energy only	159297		159297	6.2
0debris	21OCT2003 0200	45.28	44.62	Energy only	167101		167101	6.4
0debris	21OCT2003 0300	46.12	45.44	Energy only	174426		174426	6.5
0debris	21OCT2003 0400	46.96	46.24	Energy only	183725		183725	6.8
0debris	21OCT2003 0500	47.62	46.81	Energy only	199120		199120	7.4
0debris	21OCT2003 0600	48.20	47.30	Energy only	213478		213478	7.9
0debris	21OCT2003 0700	48.79	47.82	Energy only	224243		224243	8.3
0debris	21OCT2003 0800	49.41	48.39	Energy only	233942		233942	8.6
0debris	21OCT2003 0900	49.99	48.89	Press/Weir	246015	33	246048	9.1
0debris	21OCT2003 1000	50.41	49.18	Press/Weir	262857	273	263130	9.7
0debris	21OCT2003 1100	50.70	49.33	Press/Weir	277898	647	278544	10.3
0debris	21OCT2003 1200	51.00	49.56	Press/Weir	286891	959	287850	10.6
0debris	21OCT2003 1300	51.27	49.76	Press/Weir	294974	1292	296266	10.9
0debris	21OCT2003 1400	51.50	49.93	Press/Weir	301910	1625	303534	11.1
0debris	21OCT2003 1500	51.69	50.08	Press/Weir	307563	1956	309519	11.3
0debris	21OCT2003 1600	51.84	50.19	Press/Weir	311726	2262	313987	11.5
0debris	21OCT2003 1700	51.95	50.28	Press/Weir	314723	2488	317211	11.6
0debris	21OCT2003 1800	52.03	50.34	Press/Weir	316630	2682	319312	11.7
0debris	21OCT2003 1900	52.06	50.36	Press/Weir	317515	2787	320302	11.7

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
3kdebris	20OCT2003 0600	36.99	36.60	Energy only	95238		95238	6.1
3kdebris	20OCT2003 0700	37.11	36.72	Energy only	96108		96108	6.1
3kdebris	20OCT2003 0800	37.24	36.84	Energy only	97017		97017	6.1
3kdebris	20OCT2003 0900	37.37	36.97	Energy only	97971		97971	6.1
3kdebris	20OCT2003 1000	37.51	37.11	Energy only	98987		98987	6.1
3kdebris	20OCT2003 1100	37.68	37.27	Energy only	100356		100356	6.2
3kdebris	20OCT2003 1200	37.93	37.51	Energy only	102472		102472	6.2
3kdebris	20OCT2003 1300	38.21	37.79	Energy only	104555		104555	6.2
3kdebris	20OCT2003 1400	38.65	38.21	Energy only	108189		108189	6.3
3kdebris	20OCT2003 1500	39.10	38.65	Energy only	111797		111797	6.4
3kdebris	20OCT2003 1600	39.58	39.11	Energy only	115549		115549	6.4
3kdebris	20OCT2003 1700	40.07	39.59	Energy only	119477		119477	6.5
3kdebris	20OCT2003 1800	40.36	39.87	Energy only	121101		121101	6.5
3kdebris	20OCT2003 1900	40.63	40.14	Energy only	123283		123283	6.5
3kdebris	20OCT2003 2000	40.96	40.45	Energy only	126031		126031	6.5
3kdebris	20OCT2003 2100	41.44	40.91	Energy only	130674		130674	6.6
3kdebris	20OCT2003 2200	42.23	41.68	Energy only	137760		137760	6.7
3kdebris	20OCT2003 2300	43.01	42.44	Energy only	144581		144581	6.8
3kdebris	20OCT2003 2400	43.79	43.19	Energy only	151675		151675	6.9
3kdebris	21OCT2003 0100	44.58	43.95	Energy only	159185		159185	7.1
3kdebris	21OCT2003 0200	45.37	44.72	Energy only	166943		166943	7.2
3kdebris	21OCT2003 0300	46.21	45.54	Energy only	174222		174222	7.4
3kdebris	21OCT2003 0400	47.06	46.35	Energy only	183275		183275	7.6
3kdebris	21OCT2003 0500	47.79	47.01	Energy only	195677		195677	8.1
3kdebris	21OCT2003 0600	48.62	47.82	Energy only	203269		203269	8.5
3kdebris	21OCT2003 0700	49.45	48.62	Press Only	212454		212454	8.8
3kdebris	21OCT2003 0800	50.16	49.24	Press/Weir	227332	227	227559	9.4
3kdebris	21OCT2003 0900	50.66	49.60	Press/Weir	246870	283	247153	10.2
3kdebris	21OCT2003 1000	51.04	49.85	Press/Weir	262949	778	263727	10.9
3kdebris	21OCT2003 1100	51.36	50.07	Press/Weir	275861	1379	277240	11.5
3kdebris	21OCT2003 1200	51.68	50.32	Press/Weir	284390	1894	286284	11.8
3kdebris	21OCT2003 1300	51.98	50.56	Press/Weir	292009	2481	294490	12.1
3kdebris	21OCT2003 1400	52.24	50.77	Press/Weir	298425	3226	301651	12.4
3kdebris	21OCT2003 1500	52.46	50.94	Press/Weir	303613	4040	307652	12.6
3kdebris	21OCT2003 1600	52.63	51.08	Press/Weir	307578	4776	312354	12.8
3kdebris	21OCT2003 1700	52.76	51.18	Press/Weir	310384	5398	315781	12.9
3kdebris	21OCT2003 1800	52.85	51.25	Press/Weir	312212	5872	318084	13.0
3kdebris	21OCT2003 1900	52.89	51.29	Press/Weir	313187	6102	319289	13.0

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
6kdebris	20OCT2003 0600	37.11	36.73	Energy only	94866		94866	7.5
6kdebris	20OCT2003 0700	37.24	36.85	Energy only	95765		95765	7.5
6kdebris	20OCT2003 0800	37.37	36.98	Energy only	96690		96690	7.5
6kdebris	20OCT2003 0900	37.50	37.11	Energy only	97657		97657	7.5
6kdebris	20OCT2003 1000	37.66	37.26	Energy only	98840		98840	7.6
6kdebris	20OCT2003 1100	37.84	37.44	Energy only	100219		100219	7.6
6kdebris	20OCT2003 1200	38.12	37.71	Energy only	102449		102449	7.7
6kdebris	20OCT2003 1300	38.47	38.05	Energy only	105348		105348	7.7
6kdebris	20OCT2003 1400	38.85	38.42	Energy only	108310		108310	7.7
6kdebris	20OCT2003 1500	39.30	38.85	Energy only	111863		111863	7.8
6kdebris	20OCT2003 1600	39.76	39.30	Energy only	115583		115583	7.8
6kdebris	20OCT2003 1700	40.21	39.74	Energy only	118992		118992	7.8
6kdebris	20OCT2003 1800	40.46	39.99	Energy only	120569		120569	7.8
6kdebris	20OCT2003 1900	40.74	40.26	Energy only	122819		122819	7.8
6kdebris	20OCT2003 2000	41.08	40.58	Energy only	125619		125619	7.8
6kdebris	20OCT2003 2100	41.64	41.12	Energy only	131145		131145	7.9
6kdebris	20OCT2003 2200	42.41	41.87	Energy only	137900		137900	8.0
6kdebris	20OCT2003 2300	43.19	42.62	Energy only	144648		144648	8.1
6kdebris	20OCT2003 2400	43.96	43.37	Energy only	151660		151660	8.1
6kdebris	21OCT2003 0100	44.74	44.12	Energy only	159088		159088	8.2
6kdebris	21OCT2003 0200	45.53	44.88	Energy only	166764		166764	8.4
6kdebris	21OCT2003 0300	46.45	45.81	Energy only	170959		170959	8.4
6kdebris	21OCT2003 0400	47.49	46.87	Energy only	174454		174454	8.5
6kdebris	21OCT2003 0500	48.57	47.94	Energy only	180390		180390	8.6
6kdebris	21OCT2003 0600	49.61	48.97	Press/Weir	189313	69	189382	9.0
6kdebris	21OCT2003 0700	50.30	49.50	Press/Weir	213994	285	214279	10.1
6kdebris	21OCT2003 0800	50.88	49.98	Press/Weir	230413	606	231020	10.9
6kdebris	21OCT2003 0900	51.41	50.41	Press/Weir	245194	892	246086	11.6
6kdebris	21OCT2003 1000	51.88	50.79	Press/Weir	257995	1681	259677	12.2
6kdebris	21OCT2003 1100	52.30	51.13	Press/Weir	269195	2716	271911	12.8
6kdebris	21OCT2003 1200	52.67	51.43	Press/Weir	278284	4275	282559	13.2
6kdebris	21OCT2003 1300	52.99	51.68	Press/Weir	285860	6085	291945	13.6
6kdebris	21OCT2003 1400	53.26	51.89	Press/Weir	292288	7687	299975	13.9
6kdebris	21OCT2003 1500	53.49	52.08	Press/Weir	297060	9050	306111	14.1
6kdebris	21OCT2003 1600	53.67	52.23	Press/Weir	300665	10185	310850	14.3
6kdebris	21OCT2003 1700	53.82	52.36	Press/Weir	303172	11157	314329	14.4
6kdebris	21OCT2003 1800	53.92	52.44	Press/Weir	304916	11820	316736	14.5
6kdebris	21OCT2003 1900	53.98	52.49	Press/Weir	305922	12182	318105	14.5

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
8kdebris	20OCT2003 0600	37.15	36.77	Energy only	94704		94704	8.3
8kdebris	20OCT2003 0700	37.29	36.90	Energy only	95605		95605	8.3
8kdebris	20OCT2003 0800	37.42	37.03	Energy only	96528		96528	8.3
8kdebris	20OCT2003 0900	37.55	37.16	Energy only	97488		97488	8.3
8kdebris	20OCT2003 1000	37.72	37.33	Energy only	98756		98756	8.4
8kdebris	20OCT2003 1100	37.94	37.54	Energy only	100490		100490	8.4
8kdebris	20OCT2003 1200	38.20	37.79	Energy only	102351		102351	8.5
8kdebris	20OCT2003 1300	38.57	38.15	Energy only	105315		105315	8.6
8kdebris	20OCT2003 1400	38.95	38.52	Energy only	108242		108242	8.7
8kdebris	20OCT2003 1500	39.40	38.96	Energy only	111733		111733	8.8
8kdebris	20OCT2003 1600	39.87	39.41	Energy only	115398		115398	8.9
8kdebris	20OCT2003 1700	40.29	39.83	Energy only	118382		118382	8.9
8kdebris	20OCT2003 1800	40.54	40.07	Energy only	120048		120048	8.9
8kdebris	20OCT2003 1900	40.83	40.35	Energy only	122331		122331	8.9
8kdebris	20OCT2003 2000	41.17	40.68	Energy only	125156		125156	8.9
8kdebris	20OCT2003 2100	41.78	41.27	Energy only	131136		131136	9.1
8kdebris	20OCT2003 2200	42.56	42.02	Energy only	137775		137775	9.1
8kdebris	20OCT2003 2300	43.32	42.76	Energy only	144506		144506	9.2
8kdebris	20OCT2003 2400	44.09	43.51	Energy only	151510		151510	9.2
8kdebris	21OCT2003 0100	44.87	44.26	Energy only	158922		158922	9.3
8kdebris	21OCT2003 0200	45.75	45.14	Energy only	163375		163375	9.3
8kdebris	21OCT2003 0300	46.78	46.19	Press Only	166051		166051	8.7
8kdebris	21OCT2003 0400	47.91	47.34	Press Only	168752		168752	8.8
8kdebris	21OCT2003 0500	49.13	48.59	Press/Weir	171383	13	171395	8.9
8kdebris	21OCT2003 0600	50.10	49.45	Press/Weir	192256	182	192438	10.0
8kdebris	21OCT2003 0700	50.74	49.96	Press/Weir	214311	594	214905	11.2
8kdebris	21OCT2003 0800	51.35	50.48	Press/Weir	228873	1041	229915	11.9
8kdebris	21OCT2003 0900	51.91	50.96	Press/Weir	242274	1643	243916	12.6
8kdebris	21OCT2003 1000	52.43	51.40	Press/Weir	253955	2919	256874	13.2
8kdebris	21OCT2003 1100	52.90	51.80	Press/Weir	263574	5062	268635	13.7
8kdebris	21OCT2003 1200	53.31	52.14	Press/Weir	272065	7447	279512	14.2
8kdebris	21OCT2003 1300	53.67	52.44	Press/Weir	279323	9754	289077	14.6
8kdebris	21OCT2003 1400	53.97	52.69	Press/Weir	285285	11992	297277	14.9
8kdebris	21OCT2003 1500	54.22	52.90	Press/Weir	289973	13820	303794	15.1
8kdebris	21OCT2003 1600	54.43	53.08	Press/Weir	293527	15266	308793	15.3
8kdebris	21OCT2003 1700	54.60	53.22	Press/Weir	296167	16393	312560	15.4
8kdebris	21OCT2003 1800	54.71	53.32	Press/Weir	297901	17265	315167	15.5
8kdebris	21OCT2003 1900	54.78	53.38	Press/Weir	298976	17766	316742	15.6

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
14kdebris	20OCT2003 0600	37.37	37.00	Energy only	94017		94017	11.2
14kdebris	20OCT2003 0700	37.51	37.14	Energy only	94924		94924	11.3
14kdebris	20OCT2003 0800	37.64	37.27	Energy only	95847		95847	11.4
14kdebris	20OCT2003 0900	37.82	37.44	Energy only	97147		97147	11.5
14kdebris	20OCT2003 1000	38.05	37.67	Energy only	98785		98785	11.6
14kdebris	20OCT2003 1100	38.27	37.88	Energy only	100258		100258	11.7
14kdebris	20OCT2003 1200	38.61	38.21	Energy only	102823		102823	11.8
14kdebris	20OCT2003 1300	38.94	38.53	Energy only	105034		105034	12.0
14kdebris	20OCT2003 1400	39.34	38.92	Energy only	108077		108077	12.2
14kdebris	20OCT2003 1500	39.79	39.36	Energy only	111427		111427	12.4
14kdebris	20OCT2003 1600	40.26	39.82	Energy only	114965		114965	12.6
14kdebris	20OCT2003 1700	40.55	40.11	Energy only	116613		116613	12.7
14kdebris	20OCT2003 1800	40.81	40.36	Energy only	118500		118500	12.8
14kdebris	20OCT2003 1900	41.12	40.66	Energy only	120816		120816	12.9
14kdebris	20OCT2003 2000	41.48	41.01	Energy only	123698		123698	13.0
14kdebris	20OCT2003 2100	42.28	41.79	Energy only	130851		130851	13.4
14kdebris	20OCT2003 2200	43.05	42.54	Energy only	137071		137071	13.6
14kdebris	20OCT2003 2300	43.83	43.30	Energy only	143328		143328	13.8
14kdebris	20OCT2003 2400	44.61	44.06	Energy only	150108		150108	14.1
14kdebris	21OCT2003 0100	45.49	44.94	Energy only	154053		154053	14.2
14kdebris	21OCT2003 0200	46.49	45.96	Energy only	156746		156746	14.2
14kdebris	21OCT2003 0300	47.61	47.09	Press/Weir	158662	546	159208	11.8
14kdebris	21OCT2003 0400	48.80	48.31	Press/Weir	161188	649	161837	12.0
14kdebris	21OCT2003 0500	50.05	49.58	Press/Weir	164850	806	165656	12.2
14kdebris	21OCT2003 0600	51.24	50.73	Press/Weir	175307	1354	176661	13.0
14kdebris	21OCT2003 0700	52.17	51.57	Press/Weir	193762	3561	197323	14.4
14kdebris	21OCT2003 0800	53.02	52.35	Press/Weir	205893	7399	213291	15.3
14kdebris	21OCT2003 0900	53.82	53.09	Press/Weir	215243	11700	226943	16.0
14kdebris	21OCT2003 1000	54.56	53.77	Press/Weir	223269	16619	239888	16.6
14kdebris	21OCT2003 1100	55.22	54.37	Press/Weir	231388	21155	252544	17.2
14kdebris	21OCT2003 1200	55.81	54.90	Press/Weir	238060	26452	264512	17.7
14kdebris	21OCT2003 1300	56.31	55.35	Press/Weir	243874	31487	275361	18.1
14kdebris	21OCT2003 1400	56.73	55.73	Press/Weir	248762	36076	284838	18.5
14kdebris	21OCT2003 1500	57.00	55.91	Press/Weir	255906	42964	298870	19.0
14kdebris	21OCT2003 1600	57.32	56.26	Press/Weir	255137	42209	297346	19.0
14kdebris	21OCT2003 1700	57.60	56.52	Press/Weir	257179	44318	301498	19.1
14kdebris	21OCT2003 1800	57.82	56.72	Press/Weir	258960	46130	305090	19.2
14kdebris	21OCT2003 1900	57.97	56.86	Press/Weir	260084	47637	307721	19.3

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
20kdebris	20OCT2003 0600	42.28	42.09	Energy only	83147		83147	29.3
20kdebris	20OCT2003 0700	42.50	42.30	Energy only	83820		83820	29.3
20kdebris	20OCT2003 0800	42.73	42.53	Energy only	84529		84529	29.3
20kdebris	20OCT2003 0900	43.21	43.01	Energy only	86047		86047	29.3
20kdebris	20OCT2003 1000	43.67	43.47	Energy only	87522		87522	29.0
20kdebris	20OCT2003 1100	44.14	43.95	Energy only	89029		89029	29.5
20kdebris	20OCT2003 1200	44.64	44.44	Energy only	90584		90584	29.7
20kdebris	20OCT2003 1300	45.17	44.97	Energy only	91756		91756	29.7
20kdebris	20OCT2003 1400	45.75	45.56	Energy only	92093		92093	29.7
20kdebris	20OCT2003 1500	46.42	46.24	Energy only	92478		92478	29.8
20kdebris	20OCT2003 1600	47.14	46.97	Energy only	92891		92891	29.8
20kdebris	20OCT2003 1700	47.91	47.74	Energy only	93329		93329	29.8
20kdebris	20OCT2003 1800	48.72	48.56	Press/Weir	93137	660	93798	12.5
20kdebris	20OCT2003 1900	49.60	49.45	Press/Weir	93606	695	94301	12.5
20kdebris	20OCT2003 2000	50.54	50.38	Press/Weir	94814	763	95577	12.7
20kdebris	20OCT2003 2100	51.37	51.19	Press/Weir	105233	2087	107319	14.1
20kdebris	20OCT2003 2200	52.15	51.94	Press/Weir	112456	4857	117313	15.1
20kdebris	20OCT2003 2300	52.89	52.66	Press/Weir	118278	9034	127312	15.8
20kdebris	20OCT2003 2400	53.60	53.34	Press/Weir	123246	13785	137031	16.5
20kdebris	21OCT2003 0100	54.29	54.00	Press/Weir	127581	19065	146646	17.1
20kdebris	21OCT2003 0200	54.97	54.65	Press/Weir	131771	24777	156548	17.6
20kdebris	21OCT2003 0300	55.65	55.30	Press/Weir	135482	31551	167033	18.1
20kdebris	21OCT2003 0400	56.33	55.95	Press/Weir	139114	38344	177458	18.6
20kdebris	21OCT2003 0500	57.01	56.59	Press/Weir	142225	45600	187825	19.0
20kdebris	21OCT2003 0600	57.68	57.23	Press/Weir	144899	53244	198143	19.4
20kdebris	21OCT2003 0700	58.33	57.85	Press/Weir	147390	61304	208693	19.7
20kdebris	21OCT2003 0800	58.98	58.46	Press/Weir	149864	69283	219147	20.1
20kdebris	21OCT2003 0900	59.60	59.05	Press/Weir	152335	77264	229599	20.4
20kdebris	21OCT2003 1000	60.14	59.55	Press/Weir	154286	84513	238799	20.6
20kdebris	21OCT2003 1100	60.56	59.95	Press/Weir	168715	77636	246351	22.6
20kdebris	21OCT2003 1200	60.95	60.31	Press/Weir	170538	82654	253193	22.8
20kdebris	21OCT2003 1300	61.36	60.70	Press/Weir	172458	88103	260561	23.1
20kdebris	21OCT2003 1400	61.78	61.09	Press/Weir	174311	93716	268027	23.3
20kdebris	21OCT2003 1500	62.16	61.45	Press/Weir	176106	98787	274893	23.6
20kdebris	21OCT2003 1600	62.53	61.79	Press/Weir	177836	104140	281976	23.8
20kdebris	21OCT2003 1700	62.85	62.09	Press/Weir	179404	108966	288370	24.0
20kdebris	21OCT2003 1800	63.11	62.33	Press/Weir	180699	112904	293603	24.2
20kdebris	21OCT2003 1900	63.31	62.52	Press/Weir	181650	115989	297639	24.3

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
Cont 0.3 0.5	20OCT2003 0600	37.49	37.13	Energy only	93660		93660	8.6
Cont 0.3 0.5	20OCT2003 0700	37.63	37.27	Energy only	94565		94565	8.7
Cont 0.3 0.5	20OCT2003 0800	37.80	37.43	Energy only	95763		95763	8.7
Cont 0.3 0.5	20OCT2003 0900	38.02	37.64	Energy only	97350		97350	8.8
Cont 0.3 0.5	20OCT2003 1000	38.22	37.84	Energy only	98647		98647	8.8
Cont 0.3 0.5	20OCT2003 1100	38.48	38.10	Energy only	100626		100626	8.9
Cont 0.3 0.5	20OCT2003 1200	38.80	38.41	Energy only	102772		102772	9.0
Cont 0.3 0.5	20OCT2003 1300	39.11	38.71	Energy only	104986		104986	9.1
Cont 0.3 0.5	20OCT2003 1400	39.53	39.12	Energy only	108084		108084	9.3
Cont 0.3 0.5	20OCT2003 1500	39.98	39.55	Energy only	111374		111374	9.4
Cont 0.3 0.5	20OCT2003 1600	40.42	39.99	Energy only	114575		114575	9.4
Cont 0.3 0.5	20OCT2003 1700	40.66	40.23	Energy only	115948		115948	9.4
Cont 0.3 0.5	20OCT2003 1800	40.93	40.49	Energy only	117930		117930	9.5
Cont 0.3 0.5	20OCT2003 1900	41.24	40.79	Energy only	120294		120294	9.5
Cont 0.3 0.5	20OCT2003 2000	41.69	41.23	Energy only	124365		124365	9.7
Cont 0.3 0.5	20OCT2003 2100	42.47	41.98	Energy only	130820		130820	9.8
Cont 0.3 0.5	20OCT2003 2200	43.22	42.72	Energy only	136886		136886	9.9
Cont 0.3 0.5	20OCT2003 2300	43.98	43.46	Energy only	143087		143087	10.0
Cont 0.3 0.5	20OCT2003 2400	44.78	44.24	Energy only	148815		148815	10.1
Cont 0.3 0.5	21OCT2003 0100	45.69	45.16	Energy only	152233		152233	10.1
Cont 0.3 0.5	21OCT2003 0200	46.71	46.19	Energy only	155165		155165	10.2
Cont 0.3 0.5	21OCT2003 0300	47.82	47.32	Press/Weir	157847	25	157872	9.1
Cont 0.3 0.5	21OCT2003 0400	48.99	48.50	Press/Weir	161800	40	161840	9.3
Cont 0.3 0.5	21OCT2003 0500	50.04	49.50	Press/Weir	176265	161	176426	10.2
Cont 0.3 0.5	21OCT2003 0600	50.74	50.07	Press/Weir	199748	636	200384	11.5
Cont 0.3 0.5	21OCT2003 0700	51.38	50.62	Press/Weir	214722	1196	215918	12.4
Cont 0.3 0.5	21OCT2003 0800	52.01	51.18	Press/Weir	227029	1892	228920	13.1
Cont 0.3 0.5	21OCT2003 0900	52.60	51.70	Press/Weir	238881	3046	241927	13.8
Cont 0.3 0.5	21OCT2003 1000	53.13	52.17	Press/Weir	248022	6241	254263	14.3
Cont 0.3 0.5	21OCT2003 1100	53.63	52.60	Press/Weir	256381	9106	265487	14.8
Cont 0.3 0.5	21OCT2003 1200	54.08	52.99	Press/Weir	263841	12274	276115	15.2
Cont 0.3 0.5	21OCT2003 1300	54.46	53.32	Press/Weir	270525	15196	285721	15.6
Cont 0.3 0.5	21OCT2003 1400	54.80	53.60	Press/Weir	276064	18082	294146	15.9
Cont 0.3 0.5	21OCT2003 1500	55.08	53.85	Press/Weir	280270	20209	300479	16.2
Cont 0.3 0.5	21OCT2003 1600	55.31	54.05	Press/Weir	283602	22067	305669	16.3
Cont 0.3 0.5	21OCT2003 1700	55.50	54.21	Press/Weir	286287	23457	309744	16.5
Cont 0.3 0.5	21OCT2003 1800	55.63	54.33	Press/Weir	288154	24561	312714	16.6
Cont 0.3 0.5	21OCT2003 1900	55.71	54.40	Press/Weir	289337	25281	314618	16.7

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
Cont 0.5 0.7	20OCT2003 0600	37.81	37.47	Energy only	92961		92961	8.5
Cont 0.5 0.7	20OCT2003 0700	38.04	37.69	Energy only	94520		94520	8.6
Cont 0.5 0.7	20OCT2003 0800	38.24	37.88	Energy only	95725		95725	8.6
Cont 0.5 0.7	20OCT2003 0900	38.42	38.06	Energy only	96860		96860	8.7
Cont 0.5 0.7	20OCT2003 1000	38.69	38.33	Energy only	98788		98788	8.7
Cont 0.5 0.7	20OCT2003 1100	38.96	38.59	Energy only	100540		100540	8.8
Cont 0.5 0.7	20OCT2003 1200	39.24	38.87	Energy only	102400		102400	8.9
Cont 0.5 0.7	20OCT2003 1300	39.60	39.22	Energy only	104956		104956	9.0
Cont 0.5 0.7	20OCT2003 1400	40.02	39.63	Energy only	107929		107929	9.1
Cont 0.5 0.7	20OCT2003 1500	40.46	40.06	Energy only	111075		111075	9.2
Cont 0.5 0.7	20OCT2003 1600	40.73	40.32	Energy only	112529		112529	9.2
Cont 0.5 0.7	20OCT2003 1700	40.97	40.56	Energy only	114256		114256	9.2
Cont 0.5 0.7	20OCT2003 1800	41.25	40.84	Energy only	116373		116373	9.3
Cont 0.5 0.7	20OCT2003 1900	41.59	41.16	Energy only	118865		118865	9.3
Cont 0.5 0.7	20OCT2003 2000	42.28	41.83	Energy only	124730		124730	9.5
Cont 0.5 0.7	20OCT2003 2100	43.01	42.55	Energy only	130480		130480	9.6
Cont 0.5 0.7	20OCT2003 2200	43.75	43.27	Energy only	136225		136225	9.7
Cont 0.5 0.7	20OCT2003 2300	44.51	44.01	Energy only	142184		142184	9.8
Cont 0.5 0.7	20OCT2003 2400	45.34	44.84	Energy only	145983		145983	9.8
Cont 0.5 0.7	21OCT2003 0100	46.29	45.80	Energy only	148919		148919	9.9
Cont 0.5 0.7	21OCT2003 0200	47.32	46.85	Press only	151677		151677	8.7
Cont 0.5 0.7	21OCT2003 0300	48.45	48.00	Press only	154298		154298	8.9
Cont 0.5 0.7	21OCT2003 0400	49.65	49.21	Press/Weir	157178	24	157202	9.1
Cont 0.5 0.7	21OCT2003 0500	50.36	49.76	Press/Weir	187150	343	187494	10.8
Cont 0.5 0.7	21OCT2003 0600	50.95	50.25	Press/Weir	205068	799	205867	11.8
Cont 0.5 0.7	21OCT2003 0700	51.56	50.78	Press/Weir	217469	1331	218800	12.5
Cont 0.5 0.7	21OCT2003 0800	52.14	51.30	Press/Weir	229245	2049	231293	13.2
Cont 0.5 0.7	21OCT2003 0900	52.69	51.79	Press/Weir	239412	4027	243439	13.8
Cont 0.5 0.7	21OCT2003 1000	53.21	52.24	Press/Weir	248497	6715	255212	14.3
Cont 0.5 0.7	21OCT2003 1100	53.68	52.65	Press/Weir	256546	9705	266251	14.8
Cont 0.5 0.7	21OCT2003 1200	54.11	53.02	Press/Weir	263917	12716	276633	15.2
Cont 0.5 0.7	21OCT2003 1300	54.48	53.34	Press/Weir	270381	15638	286018	15.6
Cont 0.5 0.7	21OCT2003 1400	54.81	53.62	Press/Weir	275384	18273	293657	15.9
Cont 0.5 0.7	21OCT2003 1500	55.10	53.87	Press/Weir	279573	20392	299965	16.1
Cont 0.5 0.7	21OCT2003 1600	55.33	54.07	Press/Weir	282922	22252	305174	16.3
Cont 0.5 0.7	21OCT2003 1700	55.51	54.23	Press/Weir	285613	23673	309286	16.5
Cont 0.5 0.7	21OCT2003 1800	55.65	54.35	Press/Weir	287523	24800	312323	16.6
Cont 0.5 0.7	21OCT2003 1900	55.74	54.43	Press/Weir	288760	25552	314311	16.6

HEC-RAS Plan	Profile	Upstream Energy Grade (ft)	Upstream WSEL (ft)	Bridge Computation Method	Q Bridge (cfs)	Q Weir (cfs)	Q Total (cfs)	Velocity thru Bridge Opening (ft/s)
BankSta	20OCT2003 0600	37.17	36.86	Energy only		94562	94562	8.7
BankSta	20OCT2003 0700	37.30	36.99	Energy only		95465	95465	8.7
BankSta	20OCT2003 0800	37.43	37.12	Energy only		96384	96384	8.8
BankSta	20OCT2003 0900	37.56	37.25	Energy only		97331	97331	8.8
BankSta	20OCT2003 1000	37.74	37.43	Energy only		98703	98703	8.9
BankSta	20OCT2003 1100	37.98	37.66	Energy only		100537	100537	9.0
BankSta	20OCT2003 1200	38.23	37.91	Energy only		102390	102390	9.0
BankSta	20OCT2003 1300	38.61	38.28	Energy only		105386	105386	9.2
BankSta	20OCT2003 1400	38.99	38.65	Energy only		108334	108334	9.3
BankSta	20OCT2003 1500	39.43	39.09	Energy only		111809	111809	9.4
BankSta	20OCT2003 1600	39.90	39.55	Energy only		115465	115465	9.5
BankSta	20OCT2003 1700	40.28	39.93	Energy only		118084	118084	9.6
BankSta	20OCT2003 1800	40.53	40.17	Energy only		119836	119836	9.6
BankSta	20OCT2003 1900	40.82	40.45	Energy only		122146	122146	9.6
BankSta	20OCT2003 2000	41.16	40.79	Energy only		124993	124993	9.7
BankSta	20OCT2003 2100	41.80	41.41	Energy only		131203	131203	9.9
BankSta	20OCT2003 2200	42.57	42.16	Energy only		137752	137752	10.0
BankSta	20OCT2003 2300	43.33	42.91	Energy only		144413	144413	10.1
BankSta	20OCT2003 2400	44.10	43.67	Energy only		151399	151399	10.2
BankSta	21OCT2003 0100	44.87	44.42	Energy only		158825	158825	10.4
BankSta	21OCT2003 0200	45.82	45.38	Energy only		161431	161431	10.4
BankSta	21OCT2003 0300	46.89	46.47	Press/Weir	163901	21	163923	9.4
BankSta	21OCT2003 0400	48.04	47.64	Press/Weir	166906	36	166942	9.6
BankSta	21OCT2003 0500	49.26	48.88	Press/Weir	170278	55	170333	9.8
BankSta	21OCT2003 0600	50.26	49.82	Press/Weir	189020	282	189302	10.9
BankSta	21OCT2003 0700	50.98	50.46	Press/Weir	209060	767	209828	12.0
BankSta	21OCT2003 0800	51.69	51.11	Press/Weir	222491	1307	223798	12.8
BankSta	21OCT2003 0900	52.35	51.73	Press/Weir	234274	3075	237349	13.5
BankSta	21OCT2003 1000	52.96	52.29	Press/Weir	244388	5872	250260	14.1
BankSta	21OCT2003 1100	53.51	52.79	Press/Weir	253277	8926	262203	14.6
BankSta	21OCT2003 1200	54.00	53.24	Press/Weir	261018	12237	273255	15.0
BankSta	21OCT2003 1300	54.42	53.62	Press/Weir	267875	15386	283261	15.4
BankSta	21OCT2003 1400	54.79	53.97	Press/Weir	273207	18249	291456	15.7
BankSta	21OCT2003 1500	55.11	54.26	Press/Weir	277757	20633	298390	16.0
BankSta	21OCT2003 1600	55.37	54.49	Press/Weir	281417	22606	304023	16.2
BankSta	21OCT2003 1700	55.56	54.67	Press/Weir	284170	24248	308418	16.4
BankSta	21OCT2003 1800	55.71	54.80	Press/Weir	286169	25455	311624	16.5
BankSta	21OCT2003 1900	55.80	54.89	Press/Weir	287410	26305	313715	16.6

Table 2-2: BNSF Bridge Main Channel Scour Depth and Area at 150,000 cfs

Condition	Depth of Scour (ft)	Estimated Scoured Area (sq. ft.)
No Debris	2.5	1,500
10,000 SF Debris	11.4	7,000

Table 3-1: Existing Flood Control Storage Requirements at Upper Baker and Ross Dams

Upper Baker	
Date	(ac-ft)
October 1	0
October 15	16,000
November 1	16,000
November 15	74,000
March 1	74,000
April 1	0

Ross	
Date	(ac-ft)
October 1	0
October 15	20,000
November 1	43,000
November 15	60,000
December 1	120,000
March 15	120,000

Table 3-2: Optional Flood Control Storage Requirements at Upper Baker Dam with Existing Flood Control Storage at Ross Dam

Upper Baker	
Date	(ac-ft)
October 1	0
October 15	74,000
November 1	74,000
November 15	74,000
March 1	74,000
April 1	0

Ross	
Date	(ac-ft)
October 1	0
October 15	20,000
November 1	43,000
November 15	60,000
December 1	120,000
March 15	120,000

Table 3-3: Existing Condition Regulated Peak Discharge, Skagit River near Concrete

Date	Starting Flood Storage (acre-ft)		2-yr Event	5-yr Event			10-yr Event			25-yr Event		
			Peak Discharge (cfs)[1]	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Ross		From Upper Baker	From Ross		From Upper Baker	From Ross		From Upper Baker	From Ross	
1 October	0	0	**77,300**	12,600	14,300	**111,800**	19,000	15,100	**146,200**	29,400	26,400	**193,400**
15 October	16,000	20,000	**77,300**	5,700	10,400	**101,800**	11,400	12,100	**134,800**	23,600	22,100	**183,800**
1 November	16,000	43,000	**77,300**	5,700	6,300	**101,200**	11,400	8,000	**132,100**	23,600	17,100	**180,100**
15 November	74,000	60,000	**77,300**	5,000	5,000	**100,700**	5,000	5,000	**125,500**	5,000	9,700	**159,800**
1 December	74,000	120,000	**77,300**	5,000	5,000	**100,700**	5,000	5,000	**125,500**	5,000	5,000	**159,300**
Ratio 1 October to 1 December	n/a	n/a	**1.00**	n/a	n/a	**1.11**	n/a	n/a	**1.16**	n/a	n/a	**1.21**
Weighted	n/a	n/a	**77,300**	n/a	n/a	**101,100**	n/a	n/a	**127,700**	n/a	n/a	**165,300**
Ratio Weighted to 1 December			**1.00**			**1.00**			**1.02**			**1.04**

Note: 1. Peak flow in 2-year event is below threshold which triggers flood regulation

Table 3-3 (cont.): Existing Condition Regulated Peak Discharge, Skagit River near Concrete

Date	Starting Flood Storage (acre-ft)		50-yr Event			75-yr Event			100-yr Event		
			Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Ross	From Upper Baker	From Ross		From Upper Baker	From Ross		From Upper Baker	From Ross	
1 October	0	0	33,600	26,700	**223,400**	37,000	31,600	**249,400**	39,300	36,000	**265,500**
15 October	16,000	20,000	28,600	22,600	**214,500**	33,100	28,500	**241,500**	36,600	31,200	**258,600**
1 November	16,000	43,000	28,600	17,800	**210,400**	33,100	21,800	**237,300**	36,600	26,000	**254,000**
15 November	74,000	60,000	5,000	14,500	**184,200**	5,400	20,000	**207,000**	7,500	24,300	**221,800**
1 December	74,000	120,000	5,000	5,000	**180,300**	5,000	7,900	**200,700**	7,200	10,900	**214,200**
Ratio 1 October to 1 December	n/a	n/a	n/a	n/a	**1.24**	n/a	n/a	**1.24**	n/a	n/a	**1.24**
Weighted	n/a	n/a	n/a	n/a	**189,100**	n/a	n/a	**211,400**	n/a	n/a	**225,900**
Ratio Weighted to 1 December					**1.05**			**1.05**			**1.05**

Table 3-3 (cont.): Existing Condition Regulated Peak Discharge, Skagit River near Concrete

Date	Starting Flood Storage (acre-ft)		250-yr Event			500-yr Event		
			Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Ross	From Upper Baker	From Ross		From Upper Baker	From Ross	
1 October	0	0	45,000	56,600	**318,400**	51,000	68,600	**358,500**
15 October	16,000	20,000	44,900	50,600	**313,700**	50,400	68,500	**353,900**
1 November	16,000	43,000	44,900	43,700	**308,500**	51,000	54,600	**348,100**
15 November	74,000	60,000	20,800	46,500	**277,500**	33,800	50,900	**325,900**
1 December	74,000	120,000	20,800	31,600	**267,400**	33,800	33,000	**313,300**
Ratio 1 October to 1 December	n/a	n/a	n/a	n/a	**1.19**	n/a	n/a	**1.14**
Weighted	n/a	n/a	n/a	n/a	**279,700**	n/a	n/a	**324,400**
Ratio Weighted to 1 December					**1.05**			**1.04**

Table 3-4: Distribution of 1-Day Winter Peak Flows

Period	No. of Events in Period	Incremental Percentage in Period	Cumulative Percentage to End of Period
Oct 1-15	3	4	4
Oct 16-31	14	16	20
Nov 1-15	9	11	31
Nov 16-30	9	11	42
Dec 1-15	14	17	59
Dec 16-31	7	8	67
Jan 1-15	8	10	77
Jan 16-31	9	11	88
Feb 1-15	4	5	93
Feb 16-28	3	3	96
Mar 1-15	2	3	99
Mar 16-31	1	1	100
Total	83		

Table 3-5: Weights Applied to Regulated Flood Hydrographs, Skagit River near Concrete

Hydrograph Date	Weight Applied (%)
Oct 1	2
Oct 15	10
Nov 1	13.5
Nov 15	11
Dec 1	63.5

Table 3-6: Regulated Peak Discharge with Increased Upper Baker Early Season Flood Storage, Skagit River near Concrete

Date	Starting Flood Storage (acre-ft)		2-yr Event	5-yr Event			10-yr Event			25-yr Event		
			Peak Discharge (cfs)[1]	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Ross		From Upper Baker	From Ross		From Upper Baker	From Ross		From Upper Baker	From Ross	
1 October	0	0	**77,300**	12,600	14,300	**111,800**	19,000	15,100	**146,200**	29,400	26,400	**193,400**
15 October	74,000	20,000	**77,300**	5,000	10,400	**101,200**	5,000	12,100	**128,400**	5,000	22,100	**165,400**
1 November	74,000	43,000	**77,300**	5,000	6,300	**100,700**	5,000	7,100	**125,700**	5,000	13,300	**161,700**
15 November	74,000	60,000	**77,300**	5,000	5,000	**100,700**	5,000	5,000	**125,500**	5,000	9,700	**159,800**
1 December	74,000	120,000	**77,300**	5,000	5,000	**100,700**	5,000	5,000	**125,500**	5,000	5,000	**159,300**
Ratio 1 October to 1 December	n/a	n/a	**1.00**	n/a	n/a	**1.11**	n/a	n/a	**1.16**	n/a	n/a	**1.21**
Weighted	n/a	n/a	**77,300**	n/a	n/a	**101,000**	n/a	n/a	**126,200**	n/a	n/a	**161,000**
Ratio Weighted to 1 December			**1.00**			**1.00**			**1.01**			**1.01**

Note: 1. Peak flow in 2-year event is below threshold which triggers flood regulation

Table 3-6 (Cont.): Regulated Peak Discharge with Increased Upper Baker Early Season Flood Storage, Skagit River near Concrete

Date	Starting Flood Storage (acre-ft)		50-yr Event			75-yr Event			100-yr Event		
			Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Ross	From Upper Baker	From Ross		From Upper Baker	From Ross		From Upper Baker	From Ross	
1 October	0	0	33,600	26,700	**223,400**	37,000	31,600	**249,400**	39,300	36,000	**265,500**
15 October	74,000	20,000	5,000	22,600	**190,900**	7,100	31,500	**214,200**	7,500	33,400	**229,600**
1 November	74,000	43,000	5,000	17,800	**187,000**	5,400	23,400	**209,900**	7,200	26,000	**225,000**
15 November	74,000	60,000	5,000	14,500	**184,200**	5,400	20,000	**207,000**	7,500	24,300	**221,800**
1 December	74,000	120,000	5,000	5,000	**180,300**	5,000	7,900	**200,700**	7,200	10,900	**214,200**
Ratio 1 October to 1 December	n/a	n/a	n/a	n/a	**1.24**	n/a	n/a	**1.24**	n/a	n/a	**1.24**
Weighted	n/a	n/a	n/a	n/a	**183,500**	n/a	n/a	**205,000**	n/a	n/a	**219,100**
Ratio Weighted to 1 December					**1.02**			**1.02**			**1.02**

Table 3-6 (Cont.): Regulated Peak Discharge with Increased Upper Baker Early Season Flood Storage, Skagit River near Concrete

Date	Starting Flood Storage (acre-ft)		250-yr Event			500-yr Event		
			Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Ross	From Upper Baker	From Ross		From Upper Baker	From Ross	
1 October	0	0	45,000	56,600	**318,400**	51,000	68,600	**358,500**
15 October	74,000	20,000	20,800	57,700	**286,300**	33,800	68,500	**335,800**
1 November	74,000	43,000	20,800	51,600	**281,000**	33,800	54,600	**330,000**
15 November	74,000	60,000	20,800	46,500	**277,500**	33,800	50,900	**325,900**
1 December	74,000	120,000	20,800	31,600	**267,400**	33,800	33,000	**313,300**
Ratio 1 October to 1 December	n/a	n/a	n/a	n/a	**1.19**	n/a	n/a	**1.14**
Weighted	n/a	n/a	n/a	n/a	**273,200**	n/a	n/a	**320,100**
Ratio Weighted to 1 December					**1.02**			**1.02**

Table 3-7: Comparison of Unregulated Peak Discharges and Regulated Peak Discharges for Existing and Optional Flood Control Storage, Skagit River near Concrete

Scenario	Peak Discharge (cfs) by Return Period (years)								
	2	5	10	25	50	75	100	250	500
Unregulated	77,300	120,500	153,300	201,200	229,300	255,500	272,400	325,400	363,600
Regulated with full flood control storage (1 December)	77,300	100,700	125,500	159,300	180,300	200,700	214,200	267,400	313,300
Existing regulation, weighted hydrograph	77,300	101,100	127,700	165,300	189,100	211,400	225,900	279,700	324,400
Optional regulation, weighted hydrograph	77,300	101,000	126,200	161,000	183,500	205,000	219,100	273,200	320,100
Difference between optional and existing regulation, weighted hydrographs	0	-100	-1,500	-4,300	-5,600	-6,400	-6,800	-6,500	-4,300

Table 4-1: Controlling Elevations for Lower Baker Dam used in Determination of Total Spillway Discharge Rating Curve

Flow Condition					
Gated Free Spill	Orifice	Submerged Weir	Free Flow Weir	Controlling Structure	Controlling Elevation (ft. NAVD88)
X				Spillway Crest	428.62
X	X			Bottom of unremovable gates when fully open (Gates 1 & 2)	439.08
X	X			Bottom of unremovable gate when fully open (Gate 23)	440.62
X	X			Bottom of unremovable gates when fully open (Gates 3 - 10)	444.12
X	X	X		Top of parapet wall at east non-overflow section	444.57
X	X	X	X	Top of wall above head gates near east abutment	444.59
X	X	X	X	Top of parapet wall at west non-overflow section	445.14
X	X	X	X	Transition from submerged weir condition to free flow weir condition at west non-overflow section	447.73
X	X		X	Bottom of gates opening for removable gates (Gates 11 - 22)	449.26
	X		X	Top deck of dam	450.64
	X		X	Top of wall above Gates 3 - 23	453.52
	X		X	Top of unremovable gates when fully open (Gates 1 & 2)	453.58
	X		X	Top of unremovable gate when fully open (Gate 23)	455.12
	X		X	Top of unremovable gates when fully open (Gates 3 - 10)[1]	458.62
	X		X	Top of wall at west non-overflow section above Gates 1 & 2	456.91

Source: Table 5-1, Tetra Tech, 2008

Gate heights 14. 5 ft

1. Tetra Tech reports 458.32 ft NAVD88

Table 4-2: Conceptual Flood Control Regulations, Skagit and Baker River Projects

Reservoir Function		Condition	Operation: ROSS	Operation: UPPER BAKER	Operation: LOWER BAKER
		Datum	(City of Seattle)	(NGVD 1947)	(NAVD 1988)
0		Initial storage	120,000 ac-ft of storage (El 1592.11)	74,000 ac-ft of storage (El 707.93)	20,000 ac-ft of storage (El 433.17)
1	Filling	Forecast of natural flow greater than 90,000 cfs at Concrete.	Start filling by setting outflow to 5,000 cfs 8 hours before natural flow at Concrete forecast to exceed 90,000 cfs	Start filling by setting outflow to 5,000 cfs 3 hours before natural flow at Concrete forecast to exceed 90,000 cfs	Start filling by reducing outflow to 1,200 cfs, or the release specified by the SGRS, starting 6 hours before forecast natural (unregulated) peak at Concrete
2	Filling - SGRS	Below NFP	Follow SGRS for release rates.	Follow SGRS for release rates.	Follow SGRS for release rates.
3	Inducing Surcharge	Exceed NFP	Continue to follow SGRS to maximum surcharge storage (NFP = 1602.0, Max Surcharge Pool = 1608.0).	Continue to follow SGRS to maximum surcharge storage (NFP = 724.0, Max Surcharge Pool = 727.0).	Not applicable - no surcharge storage. Continue to follow SGRS to the normal full pool storage (NFP = 442.35)
4	Evacuating Surcharge Storage	Ross and Upper Baker are operating in their surcharge storage pools, and inflow has peaked and is less than discharge.	Follow SGRS until inflow peaks AND SGRS allows reduced release rates, then hold the last spillway gate positions until the pool has receded to NFP El 1602.5. This step may be skipped if Concrete peaks before SGRS allows reduced release.	Follow SGRS until inflow peaks AND SGRS allows reduced release rates, then hold the last spillway gate positions until the pool has receded to NFP El 724.0. This step may be skipped if Concrete peaks before SGRS allows reduced release.	Manage releases, continuing to fill if necessary, so as not to increase the peak at Concrete.
5	Retaining Flood Control Storage	3 to 4 hours after Concrete has peaked	Ramp up to pass inflows OR continue to evacuate surcharge storage then ramp down to pass inflows.	Increase discharge to evacuate Upper Baker to begin drafting flood storage. Consider Lower Baker Operation.	Manage releases, continuing to fill if necessary, so as not to increase the peak at Concrete. Precedence is given to evacuating storage at Upper Baker. Once Upper Baker flood storage is evacuated, maintain discharge to evacuate Lower Baker flood storage.
6	Evacuating Flood Control Storage	Concrete has peaked and is less than 90,000 cfs	Ramp up to evacuate flood control storage. Pass inflow once fully evacuated (El 1592.11)	Continue evacuating flood control storage then pass inflow once fully evacuated (El 707.93)	Continue evacuating flood control storage then pass inflow once fully evacuated (El 433.17)
		Additional "Soft" Constraints	Rate of rise not to exceed 8,000 cfs/hr	Rate of rise not to exceed 8,000 cfs/hr	Rate of rise not to exceed 8,000 cfs/hr
			Don't increase peak at Concrete	Don't increase peak at Concrete	Don't increase peak at Concrete
			Consider max 30,000 cfs at Newhalem	Consider not overtopping Lower Baker	
			Don't cause Concrete to re-exceed 90,000 cfs during evacuation	Don't cause Concrete to re-exceed 90,000 cfs during evacuation	Don't cause Concrete to re-exceed 90,000 cfs during evacuation

Table 4-3: Flood Control Storage Requirements with 20,000 acre-ft of Flood Control Storage at Lower Baker Dam and Existing Storage at Upper Baker and Ross Dams

Upper Baker		Lower Baker		Ross	
Date	(ac-ft)	Date	(ac-ft)	Date	(ac-ft)
October 1	0	October 1	0	October 1	0
October 15	16,000	October 15	20,000	October 15	20,000
November 1	16,000	November 1	20,000	November 1	43,000
November 15	74,000	November 15	20,000	November 15	60,000
				December 1	120,000
March 1	74,000	March 1	20,000	March 15	120,000
April 1	0	April 1	0		

Table 4-4: Flood Control Storage Requirements with 20,000 acre-ft of Flood Control Storage at Lower Baker Dam, Increased Early Season Storage at Upper Baker Dam, and Existing Storage at Ross Dam

Upper Baker		Lower Baker		Ross	
Date	(ac-ft)	Date	(ac-ft)	Date	(ac-ft)
October 1	0	October 1	0	October 1	0
October 15	74,000	October 15	20,000	October 15	20,000
November 1	74,000	November 1	20,000	November 1	43,000
November 15	74,000	November 15	20,000	November 15	60,000
				December 1	120,000
March 1	74,000	March 1	20,000	March 15	120,000
April 1	0	April 1	0		

Table 4-5: Regulated Peak Discharge, Skagit River near Concrete, with 20,000 acre-ft Flood Control Storage at Lower Baker Dam and Existing Flood Control Storage at Upper Baker and Ross.

Date	Starting Flood Storage (acre-ft)			2-yr Event	5-yr Event			10-yr Event			25-yr Event		
				Peak Discharge (cfs)[1]	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Lower Baker	Ross		From Lower Baker	From Ross		From Lower Baker	From Ross		From Lower Baker	From Ross	
1 October	0	0	0	**77,300**	15,900	14,300	**111,800**	23,200	15,100	**146,200**	35,400	26,400	**193,400**
15 October	16,000	20,000	20,000	**77,300**	1,200	11,400	**93,800**	3,400	12,100	**122,800**	21,800	27,000	**171,100**
1 November	16,000	20,000	43,000	**77,300**	1,200	7,300	**93,300**	3,400	8,000	**120,200**	21,800	21,700	**167,400**
15 November	74,000	20,000	60,000	**77,300**	1,200	5,000	**93,300**	1,200	5,000	**117,700**	1,400	13,600	**149,800**
1 December	74,000	20,000	120,000	**77,300**	1,200	5,000	**93,300**	1,200	5,000	**117,700**	1,200	5,000	**149,200**
Ratio 1 October to 1 December	n/a	n/a	n/a	**1.00**	n/a	n/a	**1.20**	n/a	n/a	**1.24**	n/a	n/a	**1.30**

Note: 1. Peak flow in 2-year event is below threshold which triggers flood regulation

Table 4-5 (Cont.): Regulated Peak Discharge, Skagit River near Concrete, with 20,000 acre-ft Flood Control Storage at Lower Baker Dam and Existing Flood Control Storage at Upper Baker and Ross.

Date	Starting Flood Storage (acre-ft)			50-yr Event			75-yr Event			100-yr Event		
				Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Lower Baker	Ross	From Lower Baker	From Ross		From Lower Baker	From Ross		From Lower Baker	From Ross	
1 October	0	0	0	40,500	26,700	**223,400**	44,400	31,600	**249,400**	46,700	36,000	**265,500**
15 October	16,000	20,000	20,000	32,700	32,400	**205,300**	38,700	38,800	**236,600**	43,200	41,800	**256,500**
1 November	16,000	20,000	43,000	32,700	27,300	**201,000**	38,700	32,500	**232,200**	42,600	27,800	**251,900**
15 November	74,000	20,000	60,000	1,200	14,500	**174,100**	3,400	29,100	**196,300**	5,100	32,000	**210,800**
1 December	74,000	20,000	120,000	1,200	5,000	**169,800**	1,900	9,100	**190,200**	3,100	10,900	**202,800**
Ratio 1 October to 1 December	n/a	n/a	n/a	n/a	n/a	**1.32**	n/a	n/a	**1.31**	n/a	n/a	**1.31**

Table 4-5 (Cont.): Regulated Peak Discharge, Skagit River near Concrete, with 20,000 acre-ft Flood Control Storage at Lower Baker Dam and Existing Flood Control Storage at Upper Baker and Ross.

Date	Starting Flood Storage (acre-ft)			250-yr Event			500-yr Event		
				Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Lower Baker	Ross	From Lower Baker	From Ross		From Lower Baker	From Ross	
1 October	0	0	0	53,000	56,600	**318,400**	59,900	68,600	**358,500**
15 October	16,000	20,000	20,000	54,000	50,600	**314,100**	61,200	68,500	**355,400**
1 November	16,000	20,000	43,000	54,000	43,700	**308,700**	61,900	54,600	**349,600**
15 November	74,000	20,000	60,000	19,300	46,500	**264,700**	38,500	55,700	**318,100**
1 December	74,000	20,000	120,000	19,300	31,600	**254,100**	38,500	38,300	**304,600**
Ratio 1 October to 1 December	n/a	n/a	n/a	n/a	n/a	**1.25**	n/a	n/a	**1.18**

Table 4-6: Regulated Peak Discharge, Skagit River near Concrete, with 20,000 acre-ft Flood Control Storage at Lower Baker Dam, Increased Early Season Flood Control Storage at Upper Baker and Existing Flood Control Storage at Ross.

Date	Starting Flood Storage (acre-ft)			2-yr Event	5-yr Event			10-yr Event			25-yr Event		
				Peak Discharge (cfs)[1]	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Lower Baker	Ross		From Lower Baker	From Ross		From Lower Baker	From Ross		From Lower Baker	From Ross	
1 October	0	0	0	**77,300**	15,900	14,300	**111,800**	23,200	15,100	**146,200**	35,400	26,400	**193,400**
15 October	74,000	20,000	20,000	**77,300**	1,200	11,400	**93,800**	1,200	12,100	**120,600**	1,400	22,100	**155,500**
1 November	74,000	20,000	43,000	**77,300**	1,200	7,300	**93,300**	1,200	8,000	**117,900**	1,400	17,100	**151,800**
15 November	74,000	20,000	60,000	**77,300**	1,200	5,000	**93,300**	1,200	5,000	**117,700**	1,400	13,600	**149,800**
1 December	74,000	20,000	120,000	**77,300**	1,200	5,000	**93,300**	1,200	5,000	**117,700**	1,200	5,000	**149,200**
Ratio 1 October to 1 December	n/a	n/a	n/a	**1.00**	n/a	n/a	**1.20**	n/a	n/a	**1.24**	n/a	n/a	**1.30**

Note: 1. Peak flow in 2-year event is below threshold which triggers flood regulation

Table 4-6 (Cont.): Regulated Peak Discharge, Skagit River near Concrete, with 20,000 acre-ft Flood Control Storage at Lower Baker Dam, Increased Early Season Flood Control Storage at Upper Baker and Existing Flood Control Storage at Ross.

Date	Starting Flood Storage (acre-ft)			50-yr Event			75-yr Event			100-yr Event		
				Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Lower Baker	Ross	From Lower Baker	From Ross		From Lower Baker	From Ross		From Lower Baker	From Ross	
1 October	0	0	0	40,500	26,700	**223,400**	44,400	31,600	**249,400**	46,700	36,000	**265,500**
15 October	74,000	20,000	20,000	1,800	32,400	**181,000**	3,400	38,800	**203,800**	5,100	41,800	**218,600**
1 November	74,000	20,000	43,000	1,800	27,300	**176,900**	3,400	32,500	**199,400**	5,100	35,700	**214,000**
15 November	74,000	20,000	60,000	1,200	14,500	**174,100**	3,400	29,100	**196,300**	5,100	32,000	**210,800**
1 December	74,000	20,000	120,000	1,200	5,000	**169,800**	1,900	9,100	**190,200**	3,100	10,900	**202,800**
Ratio 1 October to 1 December	n/a	n/a	n/a	n/a	n/a	**1.32**	n/a	n/a	**1.31**	n/a	n/a	**1.31**

Table 4-6 (Cont.): Regulated Peak Discharge, Skagit River near Concrete, with 20,000 acre-ft Flood Control Storage at Lower Baker Dam, Increased Early Season Flood Control Storage at Upper Baker and Existing Flood Control Storage at Ross.

Date	Starting Flood Storage (acre-ft)			250-yr Event			500-yr Event		
				Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)	Contribution to Regulated Peak Discharge (cfs)		Regulated Peak Discharge (cfs)
	Upper Baker	Lower Baker	Ross	From Lower Baker	From Ross		From Lower Baker	From Ross	
1 October	0	0	0	53,000	56,600	**318,400**	59,900	68,600	**358,500**
15 October	74,000	20,000	20,000	19,300	57,700	**273,600**	38,500	68,500	**328,200**
1 November	74,000	20,000	43,000	19,300	51,600	**268,100**	40,400	61,800	**322,300**
15 November	74,000	20,000	60,000	19,300	46,500	**264,700**	38,500	55,700	**318,100**
1 December	74,000	20,000	120,000	19,300	31,600	**254,100**	38,500	38,300	**304,600**
Ratio 1 October to 1 December	n/a	n/a	n/a	n/a	n/a	**1.25**	n/a	n/a	**1.18**

Table 4-7: Comparison of Regulated Peak Discharges for Skagit River near Concrete with and without Flood Control Storage at Lower Baker Dam.

Scenario	Date	Regulated Peak Discharge (cfs) by Return Period (years)								
		2	5	10	25	50	75	100	250	500
Existing regulation at Upper Baker and Ross, no flood control storage at Lower Baker.	1 October	77,300	111,800	146,200	193,400	223,400	249,400	265,500	318,400	358,500
	15 October	77,300	101,800	134,800	183,800	214,500	241,500	258,600	313,700	353,900
	1 November	77,300	101,200	132,100	180,100	210,400	237,300	254,000	308,500	348,100
	15 November	77,300	100,700	125,500	159,800	184,200	207,000	221,800	277,500	325,900
	1 December	77,300	100,700	125,500	159,300	180,300	200,700	214,200	267,400	313,300
Existing regulation at Upper Baker and Ross, 20,000 acre-ft flood control storage at Lower Baker.	1 October	77,300	111,800	146,200	193,400	223,400	249,400	265,500	318,400	358,500
	15 October	77,300	93,800	122,800	171,100	205,300	236,600	256,500	314,100	355,400
	1 November	77,300	93,300	120,200	167,400	201,000	232,200	251,900	308,700	349,600
	15 November	77,300	93,300	117,700	149,800	174,100	196,300	210,800	264,700	318,100
	1 December	77,300	93,300	117,700	149,200	169,800	190,200	202,800	254,100	304,600
Existing regulation at Ross, increased early season flood control storage at Upper Baker, no flood control storage at Lower Baker.	1 October	77,300	111,800	146,200	193,400	223,400	249,400	265,500	318,400	358,500
	15 October	77,300	101,200	128,400	165,400	190,900	214,200	229,600	286,300	335,800
	1 November	77,300	100,700	125,700	161,700	187,000	209,900	225,000	281,000	330,000
	15 November	77,300	100,700	125,500	159,800	184,200	207,000	221,800	277,500	325,900
	1 December	77,300	100,700	125,500	159,300	180,300	200,700	214,200	267,400	313,300
Existing regulation at Ross, increased early season flood control storage at Upper Baker, 20,000 acre-ft flood control storage at Lower Baker.	1 October	77,300	111,800	146,200	193,400	223,400	249,400	265,500	318,400	358,500
	15 October	77,300	93,800	120,600	155,500	181,000	203,800	218,600	273,600	328,200
	1 November	77,300	93,300	117,900	151,800	176,900	199,400	214,000	268,100	322,300
	15 November	77,300	93,300	117,700	149,800	174,100	196,300	210,800	264,700	318,100
	1 December	77,300	93,300	117,700	149,200	169,800	190,200	202,800	254,100	304,600

Table 4-8: Reduction in Regulated Peak Discharge (cfs) on Skagit River near Concrete from Flood Regulation at Lower Baker Dam.

Scenarios	Date	Reduction in Regulated Peak Discharge (cfs) by Return Period (years) with 20,000 acre-ft Flood Control Storage at Lower Baker								
		2	5	10	25	50	75	100	250	500
Comparison of existing regulation at Upper Baker and Ross, with and without flood control storage at Lower Baker.	1 October	0	0	0	0	0	0	0	0	0
	15 October	0	8,000	12,000	12,700	9,200	4,900	2,100	-400	-1,500
	1 November	0	7,900	11,900	12,700	9,400	5,100	2,100	-200	-1,500
	15 November	0	7,400	7,800	10,000	10,100	10,700	11,000	12,800	7,800
	1 December	0	7,400	7,800	10,100	10,500	10,500	11,400	13,300	8,700
Comparison of existing regulation at Ross and increased early season flood control storage at Upper Baker, with and without flood control storage at Lower Baker.	1 October	0	0	0	0	0	0	0	0	0
	15 October	0	7,400	7,800	9,900	9,900	10,400	11,000	12,700	7,600
	1 November	0	7,400	7,800	9,900	10,100	10,500	11,000	12,900	7,700
	15 November	0	7,400	7,800	10,000	10,100	10,700	11,000	12,800	7,800
	1 December	0	7,400	7,800	10,100	10,500	10,500	11,400	13,300	8,700
Existing regulation at Ross and increased early season flood control storage at Upper Baker with flood control storage at Lower Baker, compared with existing regulation at Upper Baker and Ross without flood control storage at Lower Baker.	1 October	0	0	0	0	0	0	0	0	0
	15 October	0	8,000	14,200	28,300	33,500	37,700	40,000	40,100	25,700
	1 November	0	7,900	14,200	28,300	33,500	37,900	40,000	40,400	25,800
	15 November	0	7,400	7,800	10,000	10,100	10,700	11,000	12,800	7,800
	1 December	0	7,400	7,800	10,100	10,500	10,500	11,400	13,300	8,700

Table 4-9: Reduction in Regulated Peak Discharge (percent) on Skagit River near Concrete from Flood Regulation at Lower Baker Dam.

Scenario	Date	Reduction in Regulated Peak Discharge (percent) by Return Period (years) with 20,000 acre-ft Flood Control Storage at Lower Baker								
		2	5	10	25	50	75	100	250	500
Comparison of existing regulation at Upper Baker and Ross, with and without flood control storage at Lower Baker.	1 October	0%	0%	0%	0%	0%	0%	0%	0%	0%
	15 October	0%	8%	9%	7%	4%	2%	1%	0%	0%
	1 November	0%	8%	9%	7%	4%	2%	1%	0%	0%
	15 November	0%	7%	6%	6%	5%	5%	5%	5%	2%
	1 December	0%	7%	6%	6%	6%	5%	5%	5%	3%
Comparison of existing regulation at Ross and increased early season flood control storage at Upper Baker, with and without flood control storage at Lower Baker.	1 October	0%	0%	0%	0%	0%	0%	0%	0%	0%
	15 October	0%	7%	6%	6%	5%	5%	5%	4%	2%
	1 November	0%	7%	6%	6%	5%	5%	5%	5%	2%
	15 November	0%	7%	6%	6%	5%	5%	5%	5%	2%
	1 December	0%	7%	6%	6%	6%	5%	5%	5%	3%
Existing regulation at Ross and increased early season flood control storage at Upper Baker with flood control storage at Lower Baker, compared with existing regulation at Upper Baker and Ross without flood control storage at Lower Baker.	1 October	0%	0%	0%	0%	0%	0%	0%	0%	0%
	15 October	0%	8%	11%	15%	16%	16%	15%	13%	7%
	1 November	0%	8%	11%	16%	16%	16%	16%	13%	7%
	15 November	0%	7%	6%	6%	5%	5%	5%	5%	2%
	1 December	0%	7%	6%	6%	6%	5%	5%	5%	3%

Table 4-10: Lower Baker reservoir flood control pool evacuation data for 1 December simulations

Parameter	Lower Baker Reservoir Flood Control Pool Evacuation Data by Return Period (years)								
	2[1]	5	10	25	50	75	100	250	500
Maximum Lower Baker pool elevation (ft NAVD88)[2]	n/a	436.43	439.74	440.17	440.54	441.28	442.11	442.19	442.38
Maximum Lower Baker flood control storage used (acre-ft)[2]	n/a	6,911	14,127	15,092	15,921	17,577	19,457	19,631	20,072
Time to evacuate Lower Baker flood storage (hours)[3]	n/a	46	60	76	87	93	108	124	105

Notes:

1. Peak flow in 2-year event is below threshold which triggers flood regulation.
2. Maximum pool elevation for Lower Baker determined by Upper Baker evacuation except for 500-year event.
3. Time to evacuate storage measured from time of peak unregulated flow on Skagit River near Concrete.

Table 5-1: Damage Reaches and Index Points

Damage Reach ID	Damage Reach Descriptor	Index Point		Damage Reach Flood Inundation Analysis Method
		Model River Mile	Physical Location	
1	Upper Right Bank Floodplain	RM 21.3	Right Bank Lafayette Road	Flo 2-D
1A	Burlington	RM 18.1	Right Bank u/s BNSF bridge	Flo 2-D
2	Lower Right Bank Floodplain	RM 13.1	Right bank u/s of Division Street bridge.	Flo 2-D
2A	West Mount Vernon	RM 13.1	Right bank u/s of Division Street bridge.	Flo 2-D
3	Fir Island	No Fk RM 8.3	Left bank	Flo 2-D
4	Lower Left Bank Floodplain	So Fk RM 4.5	Left bank at Fisher Slough	Flo 2-D
4A	Mount Vernon	RM 11.7	Left bank d/s of Mount Vernon Flood Wall.	Flo 2-D
5	River Bend	RM 16.8	Left bank immediately blw I-5 bridge.	HEC-RAS Storage area
5A	North Mount Vernon	RM 16.8	Left bank immediately blw I-5 bridge.	HEC-RAS Storage area
6	Nookachamps	RM 22.1	Left bank below Hwy 9	HEC-RAS Storage area
6A	Clear Lake	RM 22.1	Left bank below Hwy 9	HEC-RAS Storage area
7	La Conner	RM 13.1	Right bank u/s of Division Street bridge.	Flo 2-D
8	Sedro-Woolley	RM 23.3	Right bank at Sedro-Woolley WWTP	HEC-RAS River WSEL

Table 5-2: Flood Quantiles (cfs): No Breach, Existing Geometry, Existing Flood Control Regulation

Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)								
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	80,400	79,500	77,900	77,100	77,000	76,900	76,900	34,900	41,700
5	105,200	99,000	93,600	91,300	91,300	91,300	91,300	42,400	48,900
10	133,300	116,800	111,300	117,100	117,100	117,100	117,000	55,800	61,200
25	169,600	140,100	132,500	149,200	149,100	149,000	149,000	72,200	76,600
50	197,500	144,900	135,200	170,600	163,300	163,300	163,300	78,300	82,500
75	220,100	156,500	152,500	177,600	165,500	165,500	165,500	78,900	83,200
100	235,800	161,200	164,800	180,700	166,400	166,400	166,300	79,100	83,400
250	289,900	195,300	199,000	187,400	168,000	168,000	167,800	79,500	83,800
500	337,400	227,400	226,800	191,800	169,000	168,900	168,700	79,700	84,100

Indicates possible overestimation of flow due to questionable HEC-RAS model behavior - data should not be used without further evaluation

Table 5-3: Levee Breach Details for Existing Condition

Damage Reach ID	Damage Reach Descriptor	Index Point		Existing Condition Levee Breach Details								
		Model River Mile	Physical Location	Minimum Flood	RAS Breach Lat. Str. Number	Lat. Str. Breach Center Station (ft)	RAS u/s XS	Levee Crest Elevation	PFP (85%) Elevation	LFP (50%) Elevation	PNP (15%) Elevation	Source PFP/PNP Data
1	Upper Right Bank Floodplain	RM 21.3	Right Bank Lafayette Road	25-yr	21.59	3350	21.6	48.66	47.66	46.66	45.66	PFP & PNP 1.0/3.0 from crest per USACE, 2013
1A	Burlington	RM 18.1	Right Bank u/s BNSF bridge	25-yr	17.89	300	17.9	45.46	44.96	44.21	43.46	PFP & PNP 0.5/2.0 from crest per USACE, 2013
2	Lower Right Bank Floodplain	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.79	3000	13.8	37.35	36.95	35.75	34.55	PFP & PNP from 2011 Hydraulic Tech. Doc.
2A	West Mount Vernon	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.79	3000	13.8	37.35	36.95	35.75	34.55	PFP & PNP from 2011 Hydraulic Tech. Doc.
3	Fir Island	No Fk RM 8.3	Left bank	10-yr	828 (NF)	400	829	26.91	25.99	24.49	22.99	PFP & PNP from 2011 Hydraulic Tech. Doc.
4	Lower Left Bank Floodplain	So Fk RM 4.5	Left bank at Fisher Slough	5-yr	464 (SF)	730	465	15.9	15.8	15.15	14.5	PFP & PNP from 2011 Hydraulic Tech. Doc.
4A	Mount Vernon	RM 11.7	Left bank d/s of Mount Vernon Flood Wall	25-yr	12.39	3500	12.4	32.99	32.89	32.24	31.59	PFP & PNP from 2011 Hydraulic Tech. Doc.
5	River Bend	RM 16.8	Left bank immediately blw I-5 bridge.	25-yr	16.779*	250	16.78	45.08	44.01	41.96	39.91	PFP & PNP from 2011 Hydraulic Tech. Doc.*
5A	North Mount Vernon	RM 16.8	Left bank immediately blw I-5 bridge.	25-yr	16.779*	250	16.78	45.08	44.01	41.96	39.91	PFP & PNP from 2011 Hydraulic Tech. Doc.*
6	Nookachamps	RM 22.1	Left bank below Hwy 9	5-yr	22.26	5300	22.2	40	n/a	n/a	n/a	No breach. Overtopping natural high ground
6A	Clear Lake	RM 22.1	Left bank below Hwy 9	5-yr	22.26	5300	22.2	40	n/a	n/a	n/a	No breach. Overtopping natural high ground
7	La Conner	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.79	3000	13.8	37.35	36.95	35.75	34.55	PFP & PNP from 2011 Hydraulic Tech. Doc.
8	Sedro-Woolley	RM 23.3	Right bank at Sedro-Woolley WWTP	25-yr	n/a	n/a	23.2	n/a	n/a	n/a	n/a	No levee.

* Note: For breach in Lateral Structure 16.779, PFP and PNP elevations were taken from 2011 Hydraulic Tech. Doc. instead of measuring down from revised levee crest elevation.
This results in an error of -0.37 ft in breach trigger elevations; correct PFP and PNP elevations should be 44.38 ft and 40.28 ft respectively.
This error has a negligible (+0.04 ft) impact on simulated flood levels in damage reaches 5/5A.

Table 5-4: Summary of Existing Condition In-Channel With-Breach Simulation Results

Damage Reach ID	Damage Reach Descriptor	Index Point: Model River Mile	Index Point: Physical Location	Minimum Flood	RAS XS	Minimum Flood: Peak Flow (cfs)	Minimum Flood: Peak Stage (ft)	100-year: Peak Flow (cfs)	100-year: Peak Stage (ft)	500-year+2SD: Peak Flow (cfs)	500-year+2SD: Peak Stage (ft)
						Peak Flow and Stage at Index Point*					
1	Upper Right Bank Floodplain	RM 21.3	Right Bank Lafayette Road	25-yr	21.6	134,700	46.33	175,300	48.74	327,200	51.98
1A	Burlington	RM 18.1	Right Bank u/s BNSF bridge	25-yr	17.9	162,500	43.90	208,900	46.71	239,800	48.95
2	Lower Right Bank Floodplain	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.8	150,800	35.73	170,400	36.79	175,700	37.11
2A	West Mount Vernon	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.8	150,800	35.73	170,400	36.79	175,700	37.11
3	Fir Island	No Fk RM 8.3	Left bank	10-yr	829	67,400	21.71	94,800	24.24	97,000	24.46
4	Lower Left Bank Floodplain	So Fk RM 4.5	Left bank at Fisher Slough	5-yr	465	42,900	13.48	81,100	15.80	82,600	15.90
4A	Mount Vernon	RM 11.7	Left bank d/s of Mount Vernon Flood Wall	25-yr	12.4	150,500	32.80	168,800	33.76	173,400	34.07
5	River Bend	RM 16.8	Left bank immediately blw I-5 bridge.	25-yr	16.78	153,600	41.06	175,400	42.94	184,700	43.48
5A	North Mount Vernon	RM 16.8	Left bank immediately blw I-5 bridge.	25-yr	16.78	153,600	41.06	175,400	42.94	184,700	43.48
6	Nookachamps	RM 22.1	Left bank below Hwy 9	5-yr	22.2	99,100	41.96	161,300	49.91	339,400	52.92
6A	Clear Lake	RM 22.1	Left bank below Hwy 9	5-yr	22.2	99,100	41.96	161,300	49.91	339,400	52.92
7	La Conner	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.8	150,800	35.73	170,400	36.79	175,700	37.11
8	Sedro-Woolley	RM 23.3	Right bank at Sedro-Woolley WWTP	25-yr	23.2	169,700	50.63	235,900	53.61	501,000	61.76

Indicates maximum post-breach in-channel value. A higher in-channel stage or flow occurs prior to levee failure.

* Note: Peak flow and stage for index points associated with a levee failure location are with a levee breach at that location only.

Table 5-5: Flood Quantiles (cfs): No Breach, Existing Geometry, Additional Early Season Flood Regulation Storage.

Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)								
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	80,400	79,500	77,900	77,100	77,000	76,900	76,900	34,900	41,700
5	105,100	99,000	93,600	91,300	91,200	91,200	91,200	42,400	48,900
10	131,800	115,900	109,600	116,400	116,300	116,300	116,300	55,400	60,800
25	165,400	137,200	132,100	145,900	145,800	145,700	145,700	70,400	75,100
50	191,900	143,100	132,700	167,700	162,200	162,100	162,100	78,000	82,200
75	213,500	152,200	147,900	176,200	165,100	165,100	165,100	78,800	83,000
100	229,100	157,900	159,800	179,500	166,000	166,000	166,000	79,100	83,300
250	282,500	190,200	194,500	186,600	167,800	167,800	167,700	79,500	83,800
500	333,200	224,500	224,400	191,400	168,900	168,900	168,700	79,700	84,000

Indicates possible overestimation of flow due to questionable HEC-RAS model behavior - data should not be used without further evaluation

Table 5-6: Difference in No-Breach Flood Quantiles (cfs): With Additional Early Season Flood Regulation Storage less Existing Condition

Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)								
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	0	0	0	0	0	0	0	0	0
5	-100	0	0	0	-100	-100	-100	0	0
10	-1,500	-900	-1,700	-700	-800	-800	-700	-400	-400
25	-4,200	-2,900	-400	-3,300	-3,300	-3,300	-3,300	-1,800	-1,500
50	-5,600	-1,800	-2,500	-2,900	-1,100	-1,200	-1,200	-300	-300
75	-6,600	-4,300	-4,600	-1,400	-400	-400	-400	-100	-200
100	-6,700	-3,300	-5,000	-1,200	-400	-400	-300	0	-100
250	-7,400	-5,100	-4,500	-800	-200	-200	-100	0	0
500	-4,200	-2,900	-2,400	-400	-100	0	0	0	-100

Indicates uncertain quantile differences due to questionable model behavior

Table 5-7: Flood Quantiles (cfs): No Breach, Improved Levees, Existing Flood Control Regulation

Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)								
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	80,400	79,300	78,500	77,000	77,000	76,900	76,800	34,900	41,700
5	105,200	99,000	92,600	91,300	91,300	91,300	91,300	42,400	48,900
10	132,700	118,000	109,600	118,500	118,400	118,400	118,400	56,400	61,800
25	169,800	138,900	131,700	149,400	149,300	149,300	149,200	72,000	76,900
50	197,400	144,700	135,400	164,900	163,800	163,800	163,800	78,100	82,700
75	220,000	156,500	151,200	169,500	165,600	165,600	165,600	78,600	83,200
100	235,700	160,600	163,400	171,700	166,300	166,300	166,300	78,800	83,400
250	289,900	194,800	197,100	176,800	167,700	167,700	167,700	79,200	83,800
500	337,500	226,700	224,200	180,400	168,600	168,600	168,600	79,300	84,100

Indicates possible overestimation of flow due to questionable HEC-RAS model behavior - data should not be used without further evaluation

Table 5-8: Difference in No-Breach Flood Quantiles (cfs): With Improved Levees less Existing Condition

Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)								
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	0	-200	600	-100	0	0	-100	0	0
5	0	0	-1,000	0	0	0	0	0	0
10	-600	1,200	-1,700	1,400	1,300	1,300	1,400	600	600
25	200	-1,200	-800	200	200	300	200	-200	300
50	-100	-200	200	-5,700	500	500	500	-200	200
75	-100	0	-1,300	-8,100	100	100	100	-300	0
100	-100	-600	-1,400	-9,000	-100	-100	0	-300	0
250	0	-500	-1,900	-10,600	-300	-300	-100	-300	0
500	100	-700	-2,600	-11,400	-400	-300	-100	-400	0

Indicates uncertain quantile differences due to questionable model behavior

Table 5-9: Levee Breach Details for Improved Levee Condition

Damage Reach ID	Damage Reach Descriptor	Index Point		Levee Breach Details for Improved Levee Condition								
		Model River Mile	Physical Location	Minimum Flood	RAS Breach Lat. Str. Number	Lat. Str. Breach Center Station (ft)	RAS u/s XS	Levee Crest Elevation	PFP (85%) Elevation	LFP (50%) Elevation	PNP (15%) Elevation	Notes
1	Upper Right Bank Floodplain	RM 21.3	Right Bank Lafayette Road	25-yr	21.59	3350	21.6	48.66	47.66	46.66	45.66	No change
1A	Burlington	RM 18.1	Right Bank u/s BNSF bridge	50-yr	17.89	300	17.9	46.45	45.95	45.20	44.45	Raise levee crest and breach elevations.
2	Lower Right Bank Floodplain	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.79	3000	13.8	38.76	38.36	37.16	35.96	Raise levee crest and breach elevations.
2A	West Mount Vernon	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.79	3000	13.8	38.76	38.36	37.16	35.96	Raise levee crest and breach elevations.
3	Fir Island	No Fk RM 8.3	Left bank	10-yr	828 (NF)	400	829	26.91	25.99	24.49	22.99	No Change
4	Lower Left Bank Floodplain	So Fk RM 4.5	Left bank at Fisher Slough	25-yr	464 (SF)	730	465	18.55	18.45	17.80	17.15	Raise levee crest and breach elevations.
4A	Mount Vernon	RM 11.7	Left bank d/s of Mount Vernon Flood Wall	25-yr	12.39	3500	12.4	32.99	32.89	32.24	31.59	No change
5	River Bend	RM 16.8	Left bank immediately blw I-5 bridge.	50-yr	16.779*	250	16.78	45.08	44.01	43.21	42.40	Raise PNP (15%). No change in crest elev.
5A	North Mount Vernon	RM 16.8	Left bank immediately blw I-5 bridge.	50-yr	16.779*	250	16.78	45.08	44.01	43.21	42.40	Raise PNP (15%). No change in crest elev.
6	Nookachamps	RM 22.1	Left bank below Hwy 9	5-yr	22.26	5300	22.2	40.00	n/a	n/a	n/a	No change
6A	Clear Lake	RM 22.1	Left bank below Hwy 9	5-yr	22.26	5300	22.2	40.00	n/a	n/a	n/a	No change
7	La Conner	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.79	3000	13.8	38.76	38.36	37.16	35.96	Raise levee crest and breach elevations.
8	Sedro-Woolley	RM 23.3	Right bank at Sedro-Woolley WWTP	25-yr	n/a	n/a	23.2	n/a	n/a	n/a	n/a	No change

* Note: For breach in Lateral Structure 16.779, PFP elevation was taken from 2011 Hydraulic Tech. Doc. instead of measuring down from revised levee crest elevation. This results in an error of -0.37 ft in the PFP elevation and -0.18 ft in the LFP elevation; correct PFP and LFP elevations should be 44.38 ft and 43.39 ft respectively. This error has a negligible (less than +0.04 ft) impact on simulated flood levels in damage reaches 5/5A.

Indicates value changed from existing condition

Table 5-10: Summary of Improved Levee Condition In-Channel With-Breach Simulation Results

Damage Reach ID	Damage Reach Descriptor	Index Point: Model River Mile	Index Point: Physical Location	Minimum Flood	RAS XS	Peak Flow and Stage at Index Point*: Minimum Flood Peak Flow (cfs)	Minimum Flood Peak Stage (ft)	100-year Peak Flow (cfs)	100-year Peak Stage (ft)	500-year+2SD Peak Flow (cfs)	500-year+2SD Peak Stage (ft)
1	Upper Right Bank Floodplain	RM 21.3	Right Bank Lafayette Road	25-yr	21.6	131,700	46.33	175,000	48.74	324,200	51.98
1A	Burlington	RM 18.1	Right Bank u/s BNSF bridge	50-yr	17.9	185,100	45.47	209,400	46.75	239,700	48.94
2	Lower Right Bank Floodplain	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.8	151,400	36.40	170,400	36.80	175,700	37.11
2A	West Mount Vernon	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.8	151,400	36.40	170,400	36.80	175,700	37.11
3	Fir Island	No Fk RM 8.3	Left bank	10-yr	829	67,100	21.79	94,800	24.24	97,100	24.77
4	Lower Left Bank Floodplain	So Fk RM 4.5	Left bank at Fisher Slough	25-yr	465	73,100	15.80	82,100	15.80	82,600	15.89
4A	Mount Vernon	RM 11.7	Left bank d/s of Mount Vernon Flood Wall	25-yr	12.4	150,700	32.80	168,800	33.77	173,400	34.06
5	River Bend	RM 16.8	Left bank immediately blw I-5 bridge.	50-yr	16.78	173,100	42.61	181,800	42.90	186,200	43.48
5A	North Mount Vernon	RM 16.8	Left bank immediately blw I-5 bridge.	50-yr	16.78	173,100	42.61	181,800	42.90	186,200	43.48
6	Nookachamps	RM 22.1	Left bank below Hwy 9	5-yr	22.2	99,000	41.99	160,600	50.00	336,400	52.99
6A	Clear Lake	RM 22.1	Left bank below Hwy 9	5-yr	22.2	99,000	41.99	160,600	50.00	336,400	52.99
7	La Conner	RM 13.1	Right bank u/s of Division Street bridge.	25-yr	13.8	151,400	36.40	170,400	36.80	175,700	37.11
8	Sedro-Woolley	RM 23.3	Right bank at Sedro-Woolley WWTP	25-yr	23.2	169,800	50.64	235,700	53.63	501,200	61.77

(Highlighted) Indicates maximum post-breach in-channel value. A higher in-channel stage or flow occurs prior to levee failure.

* Note: Peak flow and stage for index points associated with a levee failure location are with a levee breach at that location only.

Table 5-11: Standard Deviation of Model Stage Error

Data Source and Location	Standard Deviation of Error (ft)
Variation of stage observations about the existing condition HEC-RAS model rating at the USGS gage site Skagit River near Mount Vernon (including 7 November 2006 measurement).	1.2*
Variation of stage observations about the existing condition HEC-RAS model rating at the USGS gage site Skagit River near Mount Vernon (excluding 7 November 2006 measurement).	0.7
Variation of modeled water surface profiles about observed high water mark data for floods of October 2003, November 1995 and November 2006 (from Table 5-12).	1.2*

* Equal values are coincidental

Table 5-12: HEC-RAS Model Calibration and Validation Errors

Source	River	Location (River Mile)	High Water Mark (feet NAVD88)	2013 Simulated (feet NAVD88)	Difference (feet)
21 October 2003 - calibration event					
Skagit County	Mainstem Skagit	22.78	48.9	47.6	-1.30
Skagit County	Mainstem Skagit	21.6	44.5	45	0.50
Skagit County	Mainstem Skagit	19.48	43.5	43.5	0.00
Skagit County	Mainstem Skagit	17.07	40.4	41.3	0.90
USGS Gage	Mainstem Skagit	17.04	40	41.2	1.20
Skagit County	Mainstem Skagit	15.89	39	39	0.00
Skagit County	Mainstem Skagit	13.03	34	34.5	0.50
Skagit County	Mainstem Skagit	12.18	32	32.2	0.20
Skagit County	North Fork Skagit	8.09	25	24.6	-0.40
Skagit County	North Fork Skagit	4.42	15.5	12.7	-2.80
Skagit County	South Fork Skagit	5.8	19.7	19.7	0.00
Skagit County	South Fork Skagit	3.52	14.1	11.1	-3.00
29 November 1995 - validation event					
USACE	Mainstem Skagit	24.7	54.6	54.1	-0.5
USACE	Mainstem Skagit	22.4	50	46.2	-3.8
Leonard Halverson	Mainstem Skagit	22.3	45.7	46.2	0.5
Leonard Halverson	Mainstem Skagit	21.93	45.1	45.3	0.2
Leonard Halverson	Mainstem Skagit	21.6	45.2	45.2	0
Leonard Halverson	Mainstem Skagit	18.57	43.8	43.3	-0.5
Leonard Halverson	Mainstem Skagit	17.9	44.6	42.9	-1.7
Photograph (Chuck Bennett, DD#12)	Mainstem Skagit	17.54	41.7	42.1	0.4
Leonard Halverson	Mainstem Skagit	17.53	43	42.1	-0.9
Leonard Halverson	Mainstem Skagit	17.08	41	41.5	0.5
USGS Gage	Mainstem Skagit	17.04	41.1	41.3	0.2
7 November 2006 - validation event					
USGS Gage	Mainstem Skagit	22.3	46	44.8	-1.2
Skagit County	Mainstem Skagit	22.29	43.6	45.2	1.6
Skagit County	Mainstem Skagit	21.4	43.2	43.2	0
Skagit County	Mainstem Skagit	20.9	42.6	42.8	0.1
Skagit County	Mainstem Skagit	18.77	40.9	41.2	0.4
Skagit County	Mainstem Skagit	18.31	40.5	40.9	0.5
Skagit County	Mainstem Skagit	17.79	41.5	40.4	-1.1
Skagit County	Mainstem Skagit	17.12	38.4	39.5	1.1
USGS Gage	Mainstem Skagit	17.04	37.6	39.2	1.6
Skagit County	Mainstem Skagit	15.85	36.5	37.2	0.7
Skagit County	Mainstem Skagit	14.8	35.4	35.7	0.3
Skagit County	Mainstem Skagit	14.59	35.4	35.7	0.3
Skagit County	Mainstem Skagit	13.05	32.5	33.2	0.7
Skagit County	Mainstem Skagit	12.96	32	32.2	0.2
Skagit County	Mainstem Skagit	12.65	31.1	31.5	0.3
Skagit County	Mainstem Skagit	12.09	30	30.6	0.7
Skagit County	South Fork Skagit	4.59	16.5	16.7	0.2
			No. Observations		**40**
			Mean		**-0.08**
			Standard Deviation		**1.2**

Table 5-13: Standard Deviation and Coefficient of Variation of Manning's n

Average *n*	**Standard Deviation**	**Coefficient of Variation**
0.02	0.003	0.15
0.03	0.008	0.27
0.04	0.012	0.30
0.06	0.023	0.38
0.10	0.05	0.50

Source: Adapted from Figure 5.4, EM 1110-2-1619

Table 6-1: Setback Levee Reach Descriptions and Project Elements Evaluated

Setback Reach Descriptions		
Code	Name	Reach Description
NF	North Fork	North Fork from Skagit Bay to the Forks
SF	South Fork	South Fork from Skagit Bay to the Forks
LMS	Lower Mainstem	From the Forks to the Mount Vernon WWTP
MMS	Mid Mainstem	From Division St. to I-5
UMS	Upper Mainstem	From the BNSF Bridge to the upper end of DD12/Lafayette Rd
Project Element Descriptions		
Code	Name	Description
SB	Standard Setback	Approximately 1,000 foot setback.
MxSB	Maximum Setback Levee	A setback variant of approximately double the width for the preferred configuration (around 2,000 feet wide).
WMVBP	West Mt Vernon Bypass	Construction of a high flow bypass west of West Mount Vernon.
WMVLF	West Mt. Vernon Landfill Removal	Removal of the de-commissioned landfill, which forms a partial barrier to flow downstream of the Division Street Bridge.
DIVST	Division Street Bridge Center Pier Modification	Removal of the dolphin on the center pivot pier of the existing bridge in order to increase bridge conveyance.
3BR	Three Bridge Corridor	Implementation of the 3-Bridge Corridor plan, including 500 foot setbacks and extension/replacement of the existing bridges.
R/I	Raise/Improve Existing Levees	Improve existing levees by reducing geotechnical failure risk and raising crest elevations where necessary.

Table 6-2: Setback Levee Configurations Evaluated

Configuration No.	NF	SF	LMS	MMS	UMS	WMV BP	WMV LF	DIVST	3BR
1	SB								
2	SB	SB							
3	SB	SB	SB						
4	MxSB	MxSB	MxSB						
5	MxSB	MxSB	MxSB			Y	Y	Y	
6	MxSB	MxSB	MxSB	SB		Y	Y	Y	
7	MxSB	MxSB	MxSB	SB		Y	Y	Y	Y
8	MxSB		MxSB	SB	R/I		Y	Y	Y
9	SB		SB	SB	R/I		Y	Y	
Preferred	SB	SB	SB	R/I	R/I				

Note: See Table 6-1 for description of column header codes

Table 6-3: 100-yr Peak Flow and WSEL: Preferred Setback Levee Configuration

Location	Model RM	Preferred Setback Levee Configuration				Baseline Condition	
		Without BNSF Bridge Debris		With BNSF Bridge Debris		With BNSF Bridge Debris	
		100-yr Peak Flow (cfs)	100-yr WSEL (ft)	100-yr Peak Flow (cfs)	100-yr WSEL (ft)	100-yr Peak Flow (cfs)	100-yr WSEL (ft)
Upstream from Great Northern bridge	24.4	237,200	53.66	232,800	53.82	235,800	53.66
Upstream end of DD12 levee*	21.5	157,200	49.92	148,700	50.45	163,300	49.78
Upstream from BNSF bridge	18.2	191,100	48.21	172,400	49.23	171,700	48.06
Below I-5 bridge	16.9	191,100	45.44	172,400	43.74	166,200	43.65
Above Division Street bridge	13.9	191,100	38.74	172,400	37.40	166,200	37.89
Below Mt. Vernon Flood Wall	12.3	191,100	35.00	172,400	33.82	166,200	34.96

* Note: Flows reported at this location exclude flow through the model's left bank Nookachamps storage areas

Table 6-4: 100-yr Peak Flow and WSEL: Joe Leary Slough Flood Bypass Alternative

Location	Model RM	Joe Leary Slough Flood Bypass				Baseline Condition	
		Wide Variant with BNSF Bridge Debris		Narrow Variant with BNSF Bridge Debris		With BNSF Bridge Debris	
		100-yr Peak Flow (cfs)	100-yr WSEL (ft)	100-yr Peak Flow (cfs)	100-yr WSEL (ft)	100-yr Peak Flow (cfs)	100-yr WSEL (ft)
Upstream from Great Northern bridge	24.4	235,900	53.61	235,900	53.61	235,700	53.66
Upstream from flood bypass intake*	21.5	195,200	46.50	192,900	46.71	163,400	49.78
Joe Leary Slough Flood Bypass	n/a	87,400	n/a	83,800	n/a	n/a	n/a
Upstream from BNSF bridge	18.2	149,700	44.51	151,800	44.74	171,700	48.06
Below I-5 bridge	16.9	149,700	42.02	151,800	42.24	166,300	43.65
Above Division Street bridge	13.9	149,700	36.58	151,800	36.76	166,300	37.89
Below Mt. Vernon Flood Wall	12.3	149,600	33.78	151,700	33.95	166,300	34.96

* Note: Flows reported at this location exclude flow through the model's left bank Nookachamps storage areas

Table 6-5: Flood Quantiles (cfs): No Breach, Joe Leary Slough Flood Bypass, Wide Variant

Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)										Joe Leary Bypass
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 22.28) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5A)	RM 13.3 (XS 13.8) (5)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)	
2	80,400	79,500	80,500	77,000	77,000	76,900	76,900	76,900	34,900	41,800	0
5	105,200	99,100	105,300	91,400	91,400	91,400	91,400	91,400	42,500	49,000	0
10	132,800	118,900	132,800	118,100	118,000	118,000	118,000	118,000	56,200	61,600	0
25	169,800	139,500	169,700	145,500	145,500	145,400	145,400	145,200	69,000	74,300	63,500
50	197,000	156,400	197,200	147,600	147,200	147,000	147,000	146,600	69,700	74,600	75,900
75	220,000	172,000	220,400	148,200	148,100	147,700	147,700	147,400	70,500	75,700	82,500
100	235,700	181,600	236,100	149,800	149,400	149,400	149,400	149,300	72,400	77,000	87,400
250	289,900	212,100	290,300	163,800	163,200	163,200	163,200	163,200	78,000	82,500	117,000
500	337,500	236,400	337,800	171,600	166,300	166,300	166,300	166,300	78,800	83,400	134,200

Index Point location for Damage Reaches 1 and 5 for Joe Leary Bypass are different from those assumed for Existing Condition and Improved Levee Condition

Table 6-6: Difference in No-Breach Flood Quantiles (cfs): Joe Leary Slough Flood Bypass less Improved Levee Condition

Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)									
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 22.28) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5A)	RM 13.3 (XS 13.8) (5)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	0	200	n/a	0	0	n/a	0	100	0	100
5	0	100	n/a	100	100	n/a	100	100	100	100
10	100	900	n/a	-400	-400	n/a	-400	-400	-200	-200
25	0	600	n/a	-3,900	-3,800	n/a	-3,900	-4,000	-3,000	-2,600
50	-400	11,700	n/a	-17,300	-16,600	n/a	-16,800	-17,200	-8,400	-8,100
75	0	15,500	n/a	-21,300	-17,500	n/a	-17,900	-18,200	-8,100	-7,500
100	0	21,000	n/a	-21,900	-16,900	n/a	-16,900	-17,000	-6,400	-6,400
250	0	17,300	n/a	-13,000	-4,500	n/a	-4,500	-4,500	-1,200	-1,300
500	0	9,700	n/a	-8,800	-2,300	n/a	-2,300	-2,300	-500	-700

Index Point location for Damage Reaches 1 and 5 for Joe Leary Bypass are different from those assumed for Existing Condition and Improved Levee Condition

Table 6-7: Levee Breach Details for Joe Leary Slough Flood Bypass Alternative

Damage Reach ID	Damage Reach Descriptor	Index Point		Levee Breach Detail with Joe Leary Slough Flood Bypass										
		Model River Mile	Physical Location	Breach Floods	RAS Breach Lat. Str. Number	Lat. Str. Breach Center Station (ft)	RAS u/s XS	Levee Crest Elevation	PFP (85%) Elevation	LFP (50%) Elevation	PNP (15%) Elevation	Breach Trigger Elev. by Breach Flood (ft., PNP or	Notes
1	Upper Right Bank Floodplain	RM 22.269	Right Bank Sterling Dam	100yr, 250yr, 500++	22.269	6190	22.27	49.96	49.96	49.96	49.96	46.8, 46.8, 46.8	Revised index point and breach location. Breach elev. = 100-yr WSEL
1A	Burlington	RM 18.1	Right Bank u/s BNSF bridge	100yr, 250yr, 500++	17.89	300	17.9	46.45	45.95	45.20	44.45	44.2, LFP, LFP	Breach elev. lowered below PNP for 100-yr event
2	Lower Right Bank Floodplain	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.79	3000	13.8	38.76	38.36	37.16	35.96	PNP, LFP, LFP	100-yr fails at PNP, 250 & 500++ fail at LFP
2A	West Mount Vernon	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.79	3000	13.8	38.76	38.36	37.16	35.96	PNP, LFP, LFP	100-yr fails at PNP, 250 & 500++ fail at LFP
3	Fir Island	No Fk RM 8.3	Left bank	10yr, 100yr, 500++	828 (NF)	400	829	26.91	25.99	24.49	22.99	PNP, LFP, LFP	No change from improved levee condition
4	Lower Left Bank Floodplain	So Fk RM 4.5	Left bank at Fisher Slough	25yr, 100yr, 500++	464 (SF)	730	465	18.55	18.45	17.80	17.15	PNP, LFP, LFP	No change from improved levee condition
4A	Mount Vernon	RM 11.7	Left bank d/s of Mount Vernon Flood Wall	25yr, 100yr, 500++	12.39	3500	12.4	32.99	32.89	32.24	31.59	PNP, LFP, LFP	No change from improved levee condition
5	River Bend	RM 13.3	Left bank approx 4300 ft u/s Division St	25yr, 100yr, 500++	13.78	1800	13.8	38.87	38.27	36.67	35.07	PNP, PNP, LFP	New index point and breach location. 100-yr fails at PNP
5A	North Mount Vernon	RM 16.8	Left bank immediately blw I-5 bridge.	100yr, 250yr, 500++	16.779*	250	16.78	45.18	44.01	43.21	42.40	41.7, LFP, LFP	Breach elev. lowered below PNP for 100-yr event
6	Nookachamps	RM 22.1	Left bank below Hwy 9	5yr, 100yr, 500++	22.26	5300	22.2	40.00	n/a	n/a	n/a	n/a	No change from improved levee condition
6A	Clear Lake	RM 22.1	Left bank below Hwy 9	5yr, 100yr, 500++	22.26	5300	22.2	40.00	n/a	n/a	n/a	n/a	No change from improved levee condition
7	La Conner	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.79	3000	13.8	38.76	38.36	37.16	35.96	PNP, LFP, LFP	100-yr fails at PNP, 250 & 500++ fail at LFP
8	Sedro-Woolley	RM 23.3	Right bank at Sedro-Woolley WWTP	25yr, 100yr, 500++	n/a	n/a	23.2	n/a	n/a	n/a	n/a	n/a	No change from improved levee condition

* Note: For breach in Lateral Structure 16.779, PFP elevation was taken from 2011 Hydraulic Tech. Doc. instead of measuring down from revised levee crest elevation. This results in an error of -0.37 ft in the PFP elevation and -0.18 ft in the LFP elevation; correct PFP and LFP elevations should be 44.38 ft and 43.39 ft respectively. This error has a negligible (less than +0.04 ft) impact on simulated flood levels in damage reaches 5/5A.

"500++" indicates 500-year + 2 std. dev. flood

Table 6-8: Summary of Joe Leary Slough Flood Bypass In-Channel With-Breach Simulation Results

Damage Reach ID	Damage Reach Descriptor	Index Point: Model River Mile	Index Point: Physical Location	Minimum, Intermediate, and Maximum Floods	RAS u/s XS	Peak Flow and Stage at Index Point*: Minimum Flood Peak Flow (cfs)	Minimum Flood Peak Stage (ft)	Intermediate Flood Peak Flow (cfs)	Intermediate Flood Peak Stage (ft)	500-year+2SD Flood Peak Flow (cfs)	500-year+2SD Flood Peak Stage (ft)
1	Upper Right Bank Floodplain	RM 22.269	Right Bank Sterling Dam	100yr, 250yr, 500++	22.27	236,100	47.94	290,300	48.91	501,600	53.10
1A	Burlington	RM 18.1	Right Bank u/s BNSF bridge	100yr, 250yr, 500++	17.9	166,100	44.34	188,300	45.47	221,700	48.47
2	Lower Right Bank Floodplain	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.8	151,600	35.57	167,000	36.63	174,100	37.02
2A	West Mount Vernon	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.8	151,600	35.57	167,000	36.63	174,100	37.02
3	Fir Island	No Fk RM 8.3	Left bank	10yr, 100yr, 500++	829	67,300	21.74	86,100	23.28	96,200	24.39
4	Lower Left Bank Floodplain	So Fk RM 4.5	Left bank at Fisher Slough	25yr, 100yr, 500++	465	71,600	15.50	77,000	15.30	82,000	15.84
4A	Mount Vernon	RM 11.7	Left bank d/s of Division Street bridge.	25yr, 100yr, 500++	12.4	145,100	33.08	150,900	33.50	171,700	33.96
5	River Bend	RM 13.3	Left bank approx 4300 ft u/s Division St.	25yr, 100yr, 500++	13.8	147,000	35.65	148,500	36.43	169,200	38.06
5A	North Mount Vernon	RM 16.8	Left bank immediately blw I-5 bridge.	100yr, 250yr, 500++	16.78	151,400	41.15	169,400	42.56	181,300	43.40
6	Nookachamps	RM 22.1	Left bank below Hwy 9	5yr, 100yr, 500++	22.2	99,100	41.95	181,600	47.18	346,800	52.20
6A	Clear Lake	RM 22.1	Left bank below Hwy 9	5yr, 100yr, 500++	22.2	99,100	41.95	181,600	47.18	346,800	52.20
7	La Conner	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.8	151,600	35.57	167,000	36.63	174,100	37.02
8	Sedro-Woolley	RM 23.3	Right bank at Sedro-Woolley WWTP	25yr, 100yr, 500++	23.2	105,200	45.95	235,700	53.63	501,200	61.77

Indicates maximum post-breach in-channel value. A higher in-channel stage or flow occurs prior to levee failure.

* Note: Peak flow and stage for index points associated with a levee failure location are with a levee breach at that location only.

"500++" indicates 500-year + 2 std. dev. flood

Table 6-9: 100-Year Peak Flow and WSEL: Swinomish Flood Bypass Alternative

Location	Model RM	Swinomish Flood Bypass: All Variants with BNSF Bridge Debris 100-yr Peak Flow (cfs)	All Variants with BNSF Bridge Debris 100-yr WSEL (ft)	All Variants without BNSF Bridge Debris 100-yr Peak Flow (cfs)	All Variants without BNSF Bridge Debris 100-yr WSEL (ft)	Baseline Condition With BNSF Bridge Debris 100-yr Peak Flow (cfs)	Baseline Condition With BNSF Bridge Debris 100-yr WSEL (ft)
Upstream from Great Northern bridge	24.4	235,700	53.63	235,700	53.63	235,700	53.66
Upstream from BNSF bridge	18.2	199,300	47.58	211,600	46.35	171,700	48.06
Below I-5 bridge	16.9	199,300	40.85	211,500	41.54	166,300	43.65
Swinomish Flood Bypass	n/a	63,500	n/a	69,300	n/a	n/a	n/a
Above Division Street bridge	13.9	137,200	35.36	142,300	35.98	166,300	37.89
Below Mt. Vernon Flood Wall	12.3	136,900	32.65	142,300	33.23	166,300	34.96

Table 6-10: Flood Quantiles (cfs): No Breach, Swinomish Flood Bypass, Wide Variant

Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach) RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)	Bypass
2	80,400	79,300	78,500	77,000	76,900	76,900	76,800	34,900	41,600	0
5	105,200	99,000	92,600	91,300	91,300	91,200	91,200	42,500	48,700	0
10	132,700	117,900	109,800	118,400	118,300	118,300	118,300	56,600	61,600	0
25	169,800	138,900	131,600	161,300	161,300	133,900	133,300	63,300	68,000	37,200
50	197,200	146,500	146,500	187,100	187,000	135,800	135,500	63,600	68,800	57,700
75	220,000	158,200	155,500	196,000	195,900	136,600	135,900	64,500	69,300	61,900
100	235,700	166,000	160,800	199,300	199,200	137,200	136,900	65,500	70,200	63,500
250	289,900	190,900	174,600	204,800	204,700	138,700	138,700	66,900	71,600	66,100
500	337,500	217,800	187,300	216,100	215,000	144,200	144,200	69,600	74,200	70,600

Table 6-11: Difference in No-Breach Flood Quantiles (cfs): Swinomish Flood Bypass, Wide Variant less Improved Levee Condition

Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach) RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	0	0	0	0	-100	0	0	0	-100
5	0	0	0	0	0	-100	-100	100	-200
10	0	-100	200	-100	-100	-100	-100	200	-200
25	0	0	-100	11,900	12,000	-15,400	-15,900	-8,700	-8,900
50	-200	1,800	11,100	22,200	23,200	-28,000	-28,300	-14,500	-13,900
75	0	1,700	4,300	26,500	30,300	-29,000	-29,700	-14,100	-13,900
100	0	5,400	-2,600	27,600	32,900	-29,100	-29,400	-13,300	-13,200
250	0	-3,900	-22,500	28,000	37,000	-29,000	-29,000	-12,300	-12,200
500	0	-8,900	-36,900	35,700	46,400	-24,400	-24,400	-9,700	-9,900

Table 6-12: Levee Breach Details for Swinomish Flood Bypass Alternative

Damage Reach ID	Damage Reach Descriptor	Index Point		Levee Breach Detail with Swinomish Bypass									
		Model River Mile	Physical Location	Breach Floods	RAS Breach Lat. Str. Number	Lat. Str. Breach Center Station (ft)	RAS u/s XS	Levee Crest Elevation	PFP (85%) Elevation	LFP (50%) Elevation	PNP (15%) Elevation	Breach Trigger Elev. by Breach Flood (ft., PNP or LFP)	Notes
1	Upper Right Bank Floodplain	RM 21.3	Right Bank Lafayette Road	50yr, 100yr, 500++	21.59	3350	21.6	48.66	47.66	46.66	45.66	PNP, LFP, LFP	
1A	Burlington	RM 18.1	Right Bank u/s BNSF bridge	100yr, 250yr, 500++	17.89	300	17.9	50.28	49.78	49.03	48.28	46.6, LFP, LFP	Revised crest elevation, PNP & PFP. Breach elev. lowered below PNP for 100-yr event.
2	Lower Right Bank Floodplain	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.79	3000	13.8	38.76	38.36	37.16	35.96	34.86, 34.86, LFP	Breach elev. lowered below PNP for 100-yr & 250-yr events
2A	West Mount Vernon	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.79	3000	13.8	38.76	38.36	37.16	35.96	34.86, 34.86, LFP	Breach elev. lowered below PNP for 100-yr & 250-yr events
3	Fir Island	No Fk RM 8.3	Left bank	10yr, 100yr, 500++	828 (NF)	400	829	26.91	25.99	24.49	22.99	PNP, LFP, LFP	No change from improved levee condition
4	Lower Left Bank Floodplain	So Fk RM 4.5	Left bank at Fisher Slough	50yr, 100yr, 500++	464 (SF)	730	465	18.55	18.45	17.80	17.15	PNP, PNP, LFP	50-yr & 100-yr fail at PNP, 500++ at LFP
4A	Mount Vernon	RM 11.7	Left bank d/s of Mount Vernon Flood Wall	100yr, 250yr, 500++	12.39	3500	12.4	32.99	32.89	32.24	31.59	PNP, PNP, LFP	100-yr & 250-yr fail at PNP, 500++ at LFP
5	River Bend	RM 16.8	Left bank immediately blw I-5 bridge.	100yr, 250yr, 500++	16.779*	250	16.78	45.08	44.01	43.21	42.4	40.6, 40.6, LFP	Breach elev. lowered below PNP for 100-yr & 250-yr events
5A	North Mount Vernon	RM 16.8	Left bank immediately blw I-5 bridge.	100yr, 250yr, 500++	16.779*	250	16.78	45.08	44.01	43.21	42.4	40.6, 40.6, LFP	Breach elev. lowered below PNP for 100-yr & 250-yr events
6	Nookachamps	RM 22.1	Left bank below Hwy 9	5yr, 100yr, 500++	22.26	5300	22.2	40.00	n/a	n/a	n/a	n/a	No change from improved levee condition
6A	Clear Lake	RM 22.1	Left bank below Hwy 9	5yr, 100yr, 500++	22.26	5300	22.2	40.00	n/a	n/a	n/a	n/a	No change from improved levee condition
7	La Conner	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.79	3000	13.8	38.76	38.36	37.16	35.96	34.86, 34.86, LFP	Breach elev. lowered below PNP for 100-yr & 250-yr events
8	Sedro-Woolley	RM 23.3	Right bank at Sedro-Woolley WWTP	25yr, 100yr, 500++	n/a	n/a	23.2	n/a	n/a	n/a	n/a	n/a	No change from improved levee condition

* Note: For breach in Lateral Structure 16.779, PFP elevation was taken from 2011 Hydraulic Tech. Doc. instead of measuring down from revised levee crest elevation.
This results in an error of -0.37 ft in the PFP elevation and -0.18 ft in the LFP elevation; correct PFP and LFP elevations should be 44.38 ft and 43.39 ft respectively.
This error has a negligible (less than +0.04 ft) impact on simulated flood levels in damage reaches 5/5A.

"500++" indicates 500-year + 2 std. dev. flood

Table 6-13: Summary of Swinomish Flood Bypass in-Channel With-Breach Simulation Results

Damage Reach ID	Damage Reach Descriptor	Index Point		Minimum, Intermediate, and Maximum Floods	RAS u/s XS	Peak Flow and Stage at Index Point*					
						Minimum Flood		Intermediate Flood		500-year+2SD Flood	
		Model River Mile	Physical Location			Peak Flow (cfs)	Peak Stage (ft)	Peak Flow (cfs)	Peak Stage (ft)	Peak Flow (cfs)	Peak Stage (ft)
1	Upper Right Bank Floodplain	RM 21.3	Right Bank Lafayette Road	50yr, 100yr, 500++	21.6	160,300	46.84	184,300	48.12	279,500	53.20
1A	Burlington	RM 18.1	Right Bank u/s BNSF bridge	100yr, 250yr, 500++	17.9	232,200	47.05	250,100	49.42	286,400	50.97
2	Lower Right Bank Floodplain	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.8	139,100	34.68	142,700	34.94	167,300	36.65
2A	West Mount Vernon	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.8	139,100	34.68	142,700	34.94	167,300	36.65
3	Fir Island	No Fk RM 8.3	Left bank	10yr, 100yr, 500++	829	67,400	22.12	78,000	22.79	93,900	24.45
4	Lower Left Bank Floodplain	So Fk RM 4.5	Left bank at Fisher Slough	50yr, 100yr, 500++	465	62,700	15.13	66,400	14.89	79,500	15.68
4A	Mount Vernon	RM 11.7	Left bank d/s of Division Street bridge.	100yr, 250yr, 500++	12.4	138,400	32.65	141,600	32.79	165,700	33.69
5	River Bend	RM 16.8	Left bank immediately blw I-5 bridge.	100yr, 250yr, 500++	16.78	204,400	40.08	213,700	40.53	249,400	42.00
5A	North Mount Vernon	RM 16.8	Left bank immediately blw I-5 bridge.	100yr, 250yr, 500++	16.78	204,400	40.08	213,700	40.53	249,400	42.00
6	Nookachamps	RM 22.1	Left bank below Hwy 9	5yr, 100yr, 500++	22.2	99,000	41.99	166,000	49.80	322,900	54.05
6A	Clear Lake	RM 22.1	Left bank below Hwy 9	5yr, 100yr, 500++	22.2	99,000	41.99	166,000	49.80	322,900	54.05
7	La Conner	RM 13.1	Right bank u/s of Division Street bridge.	100yr, 250yr, 500++	13.8	139,100	34.68	142,700	34.94	167,300	36.65
8	Sedro-Woolley	RM 23.3	Right bank at Sedro-Woolley WWTP	25yr, 100yr, 500++	23.2	169,800	50.64	235,700	53.63	501,200	61.77

(yellow) Indicates maximum post-breach in-channel value. A higher in-channel stage or flow occurs prior to levee failure.

* Note: Peak flow and stage for index points associated with a levee failure location are with a levee breach at that location only.

"500++" indicates 500-year + 2 std. dev. flood

FIGURES

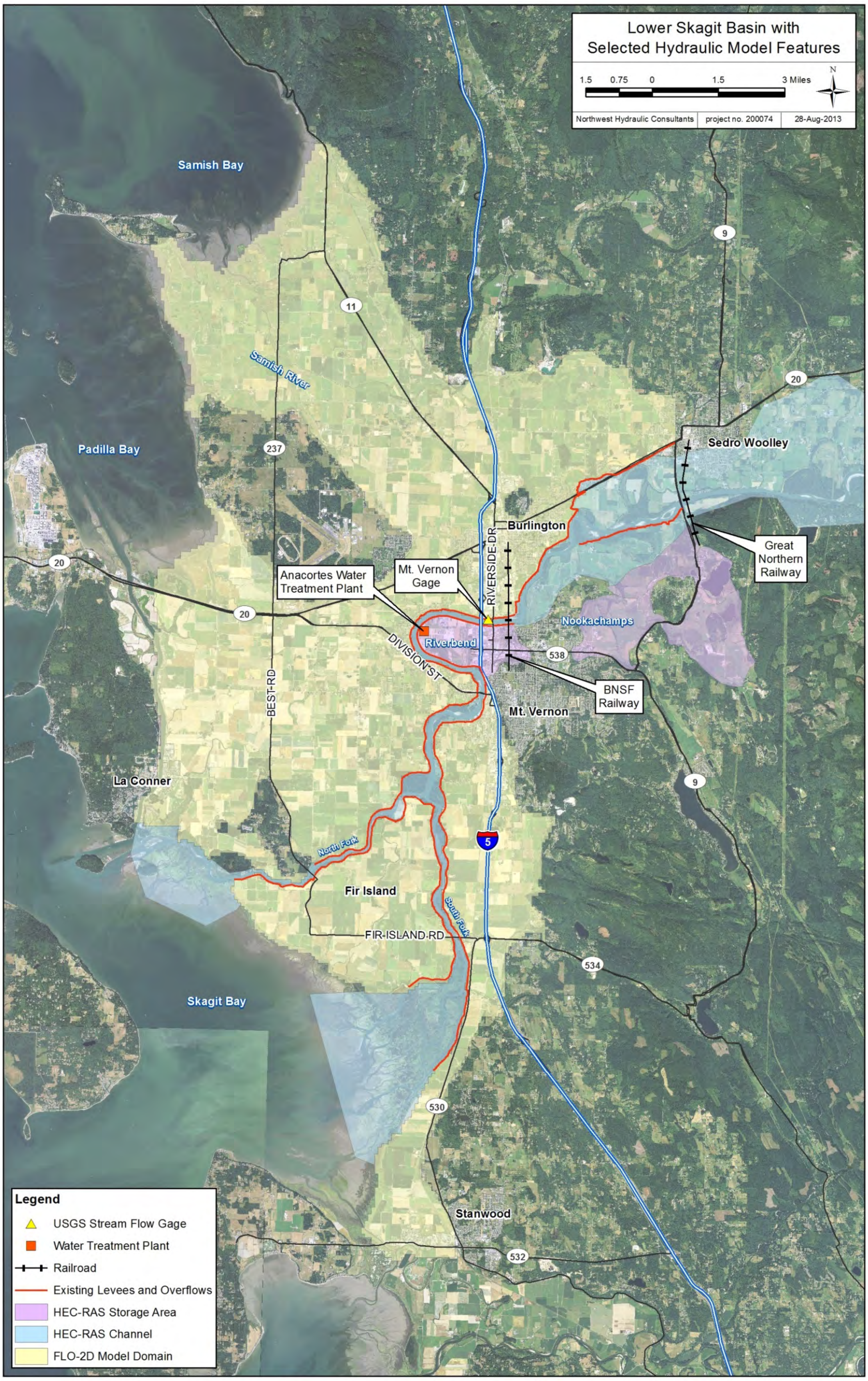


Figure 1- 1: Lower Skagit Basin with Selected Hydraulic Model Features.

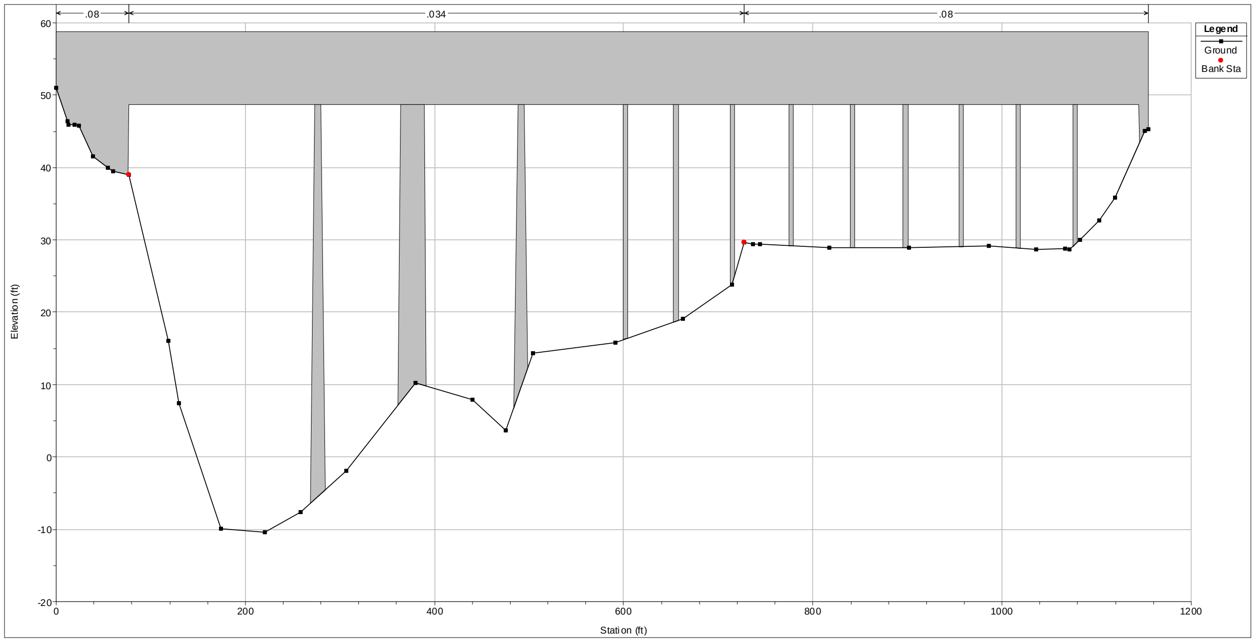


Figure 2-1: Previously assumed BNSF bridge geometry.

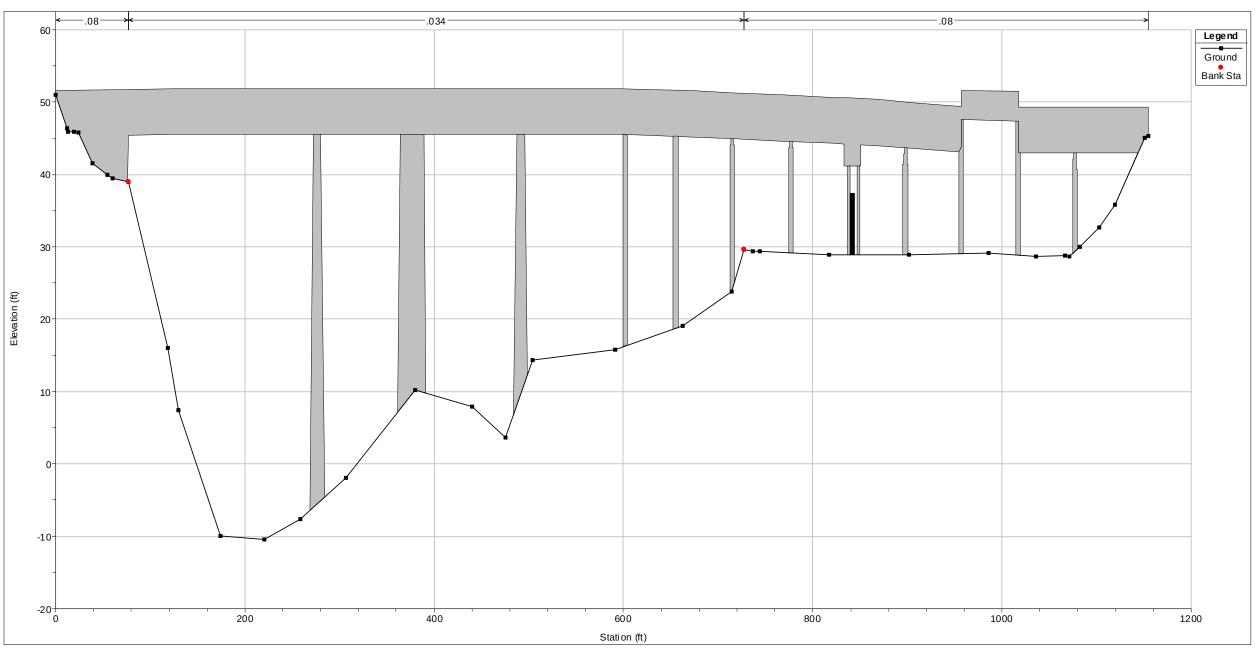


Figure 2-2: BNSF bridge geometry from November 2012 survey.

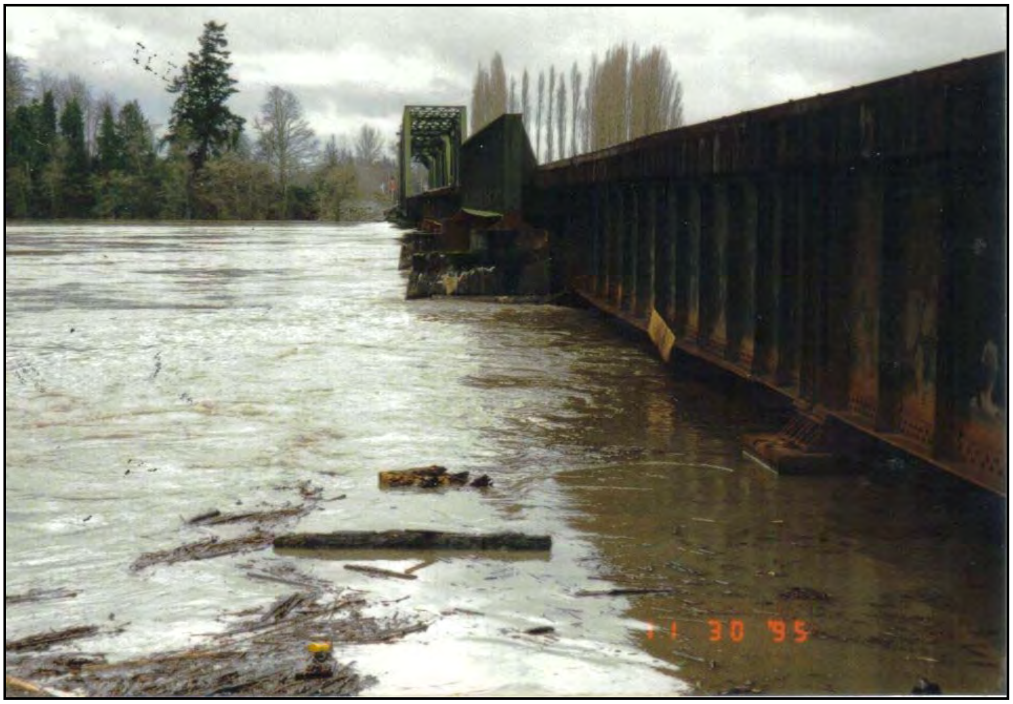


Figure 2-3: BNSF Bridge during November 1995 flood

(photo by Chuck Bennett, Dike District 12, courtesy of www.skagitriverhistory.com)

Figure 2-4: Close up of BNSF Bridge Pier during November 1995 flood

(photo by Chuck Bennett, Dike District 12, courtesy of www.skagitriverhistory.com)

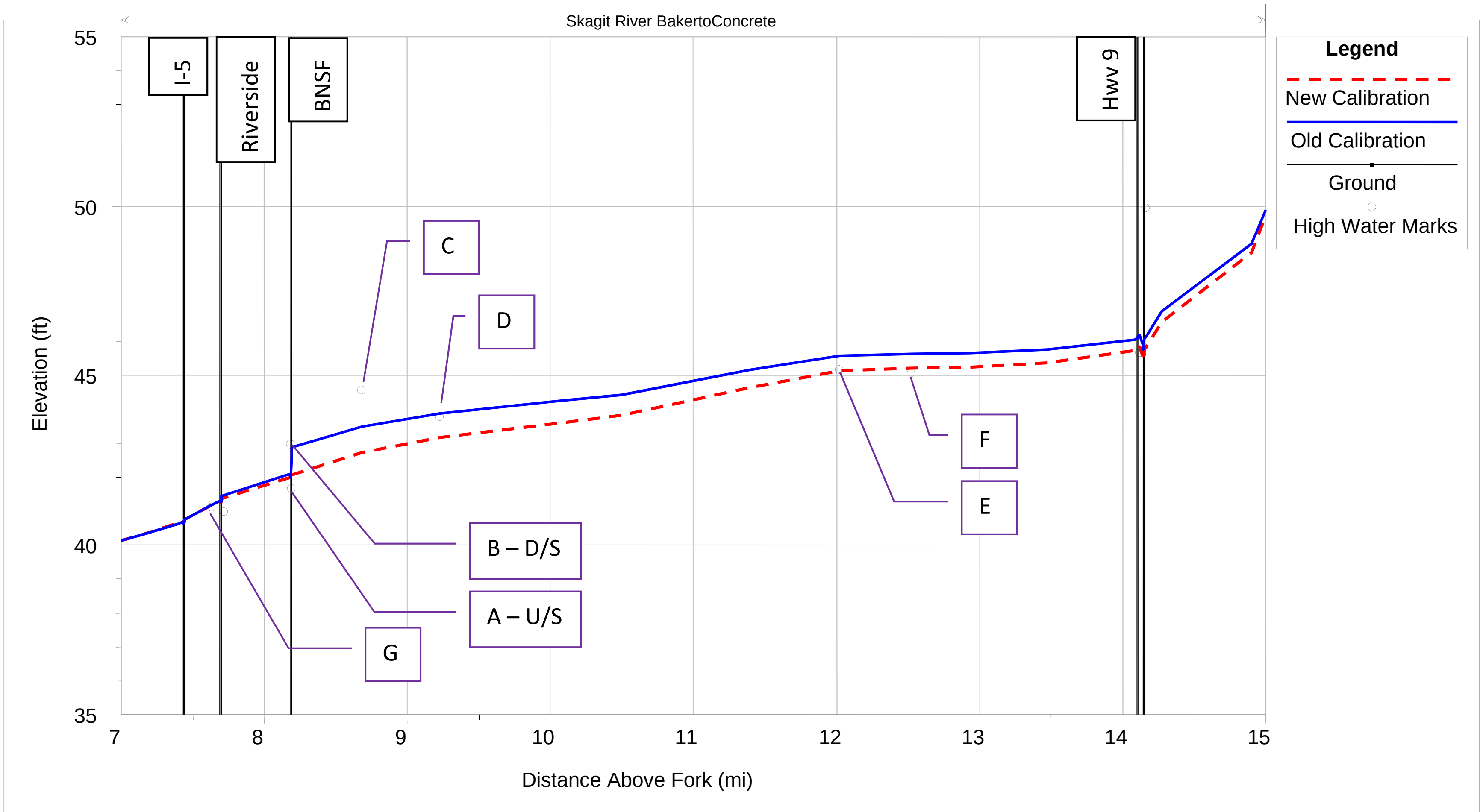


Figure 2-5: 1995 Event Simulated Water Surface Profiles and High Water Marks.

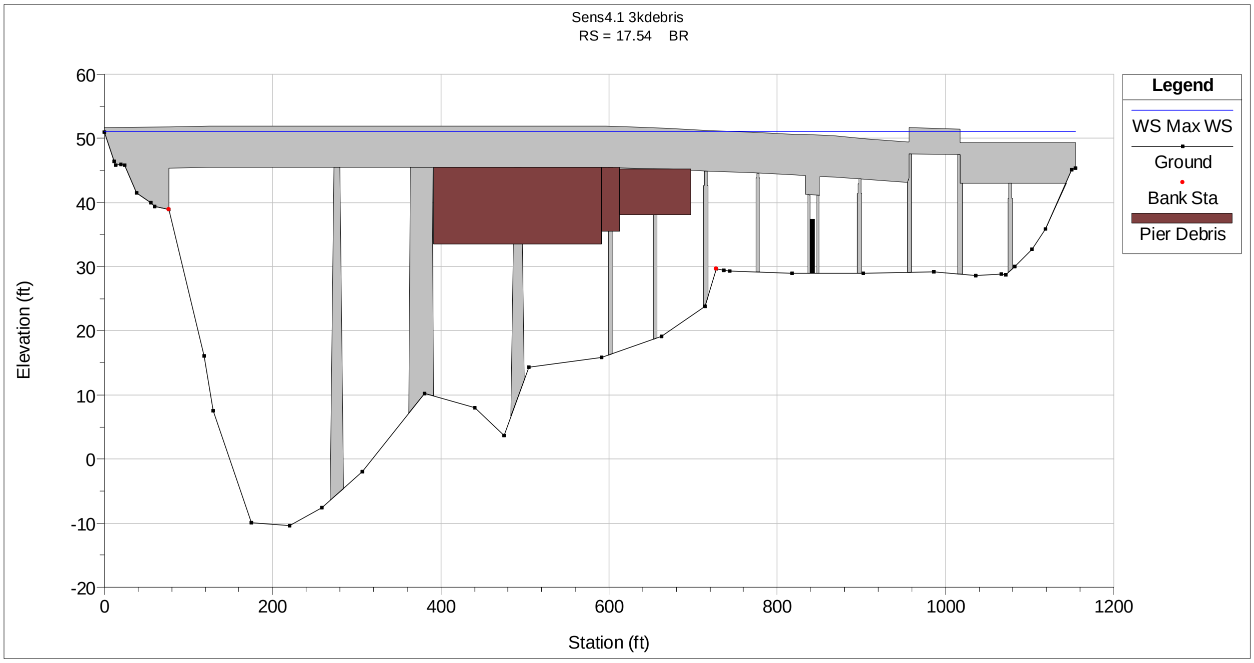


Figure 2-6: BNSF Bridge – 3,000 sq. ft. of debris.

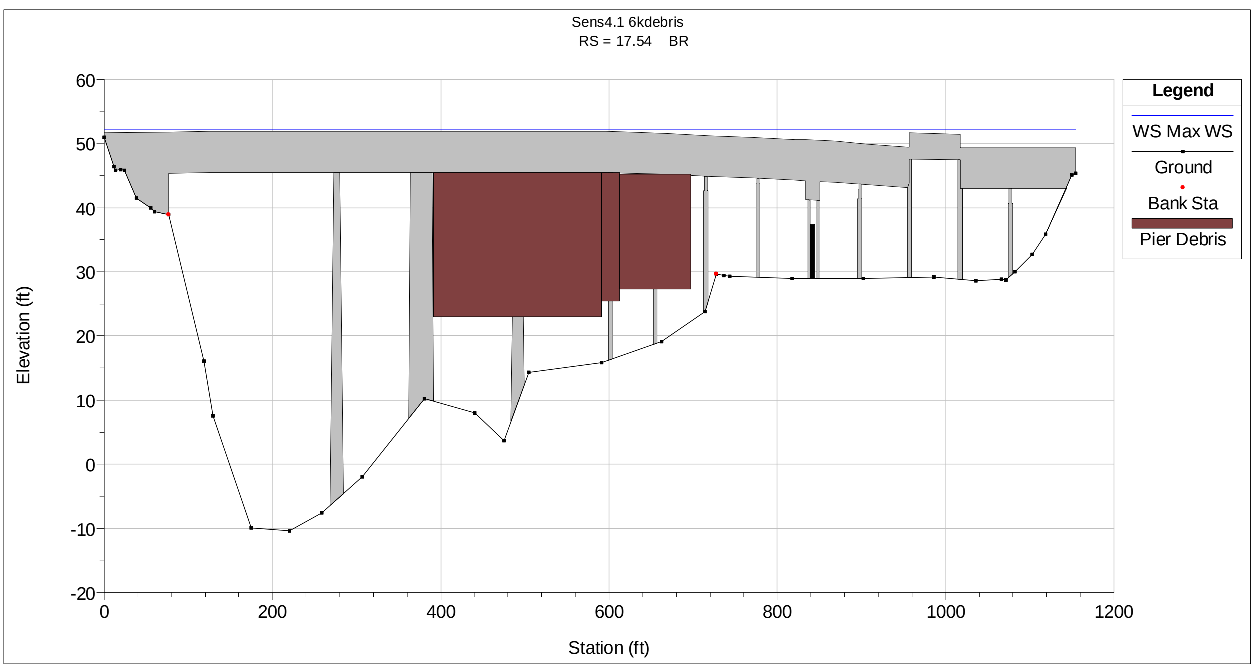


Figure 2-7: BNSF Bridge – 6,000 sq. ft. of debris.

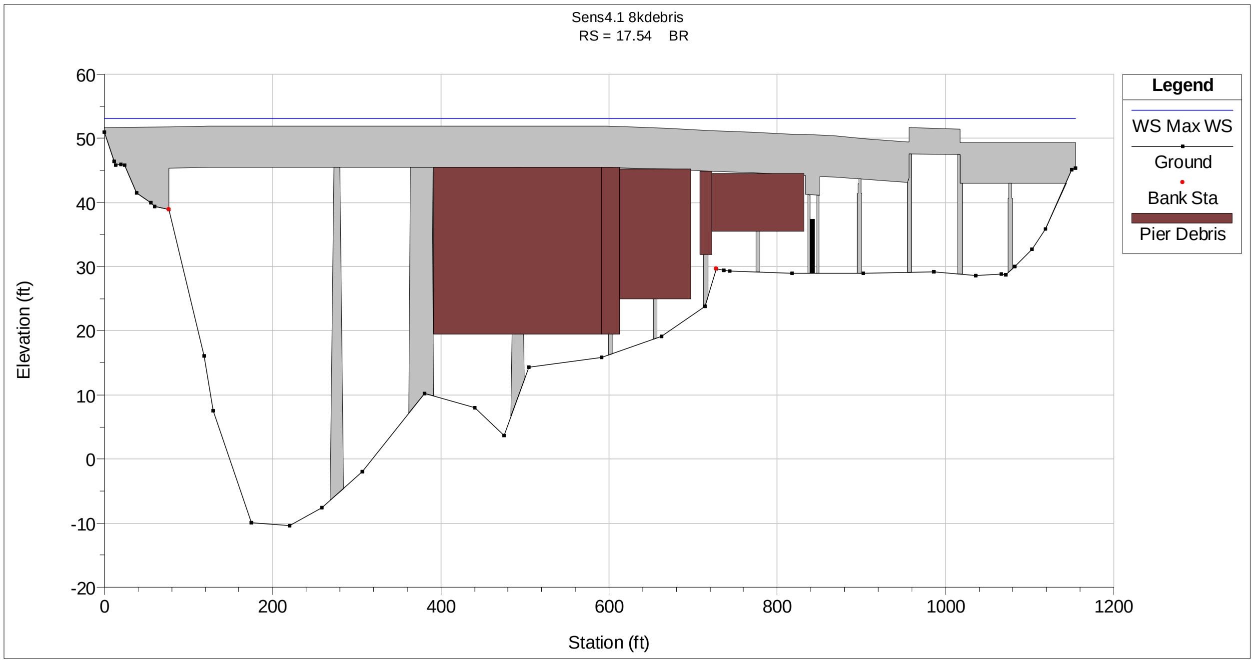


Figure 2-8: BNSF Bridge – 8,000 sq. ft. of debris.

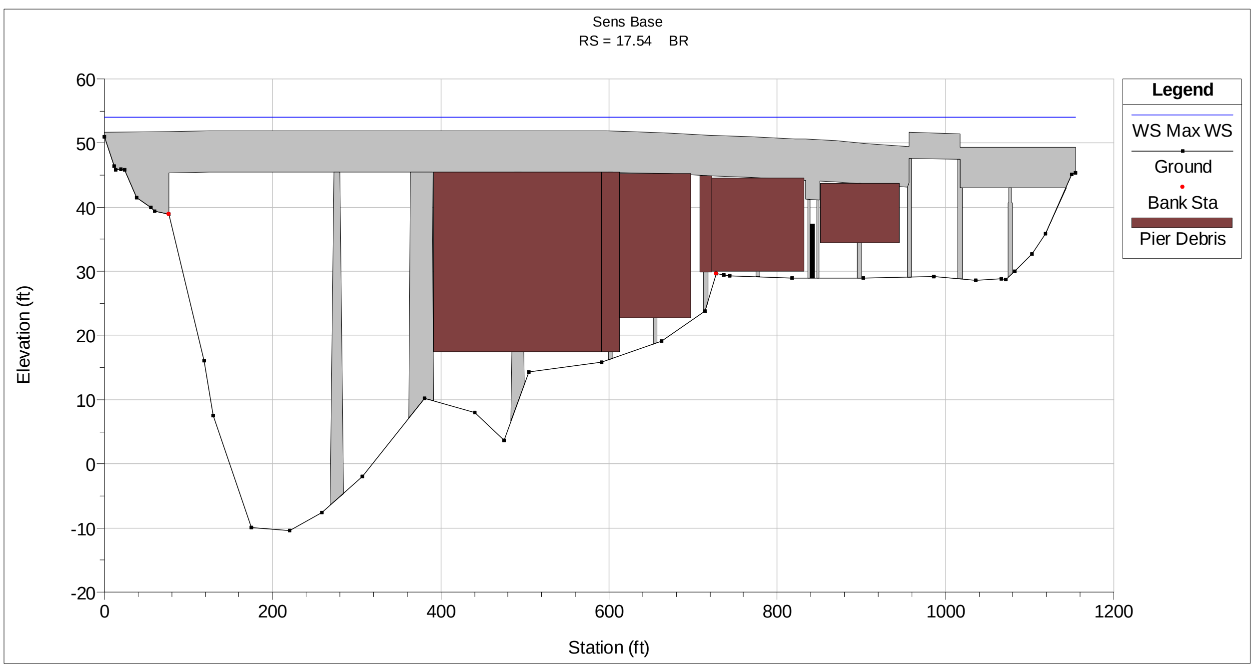


Figure 2-9: BNSF Bridge – 10,000 sq. ft. of debris (base scenario)

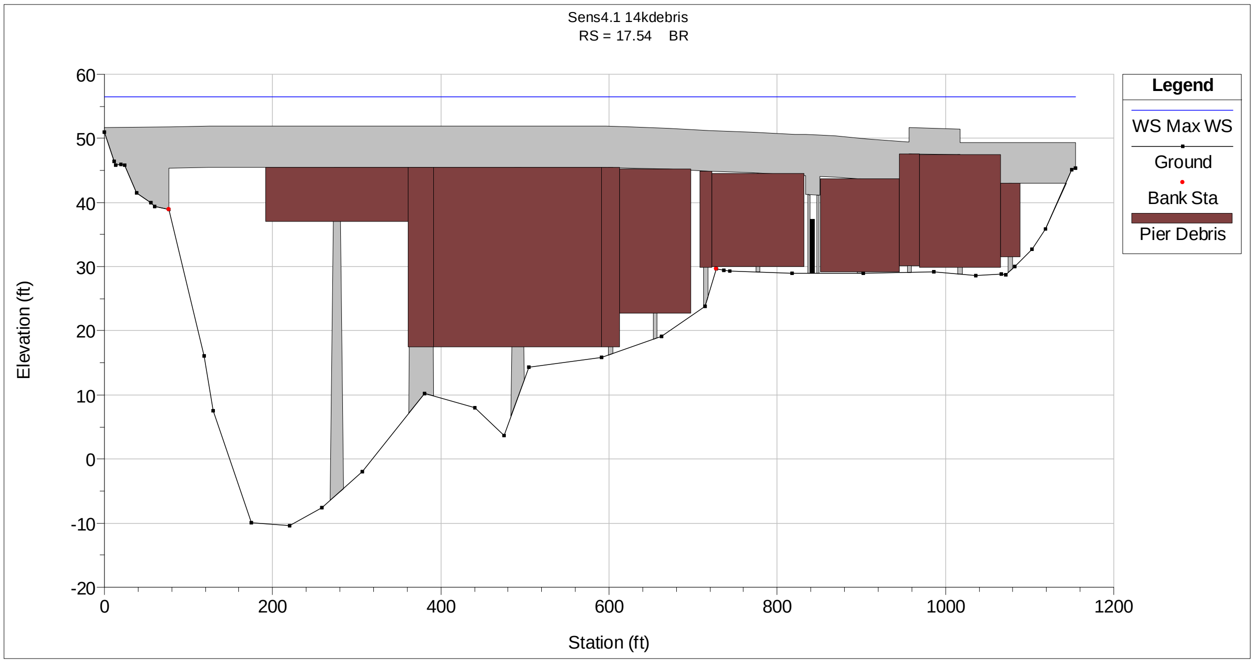


Figure 2-10: BNSF Bridge – 14,000 sq. ft. of debris.

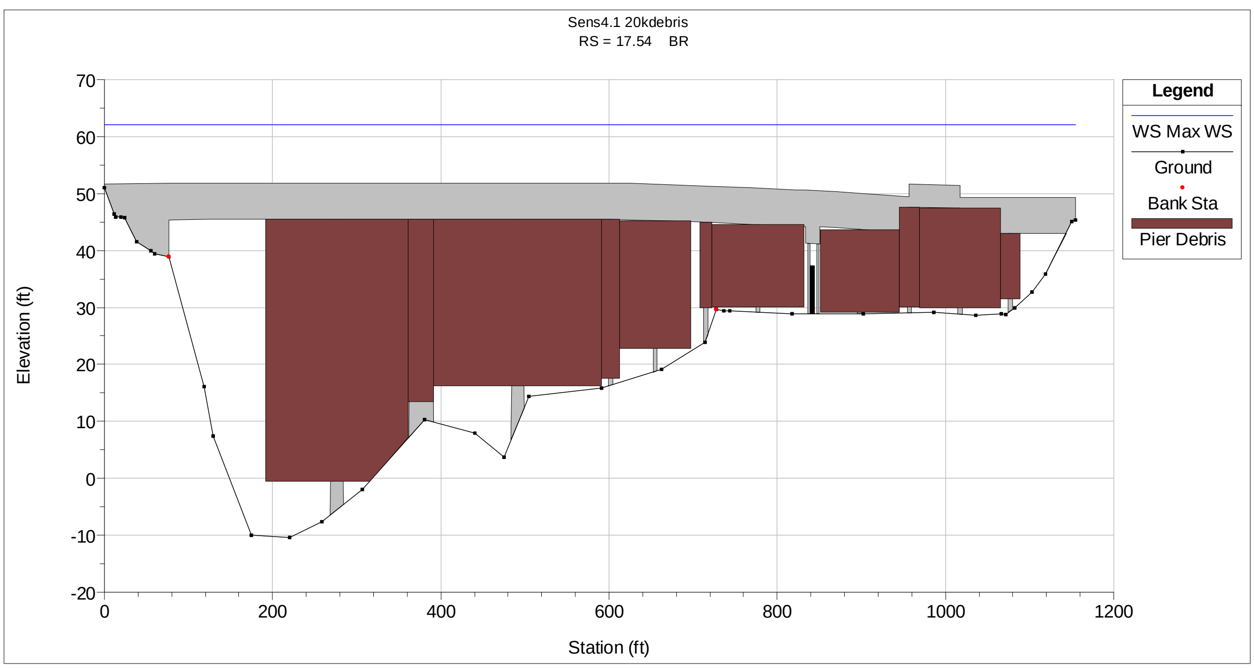


Figure 2-11: BNSF Bridge – 20,000 sq. ft. debris.

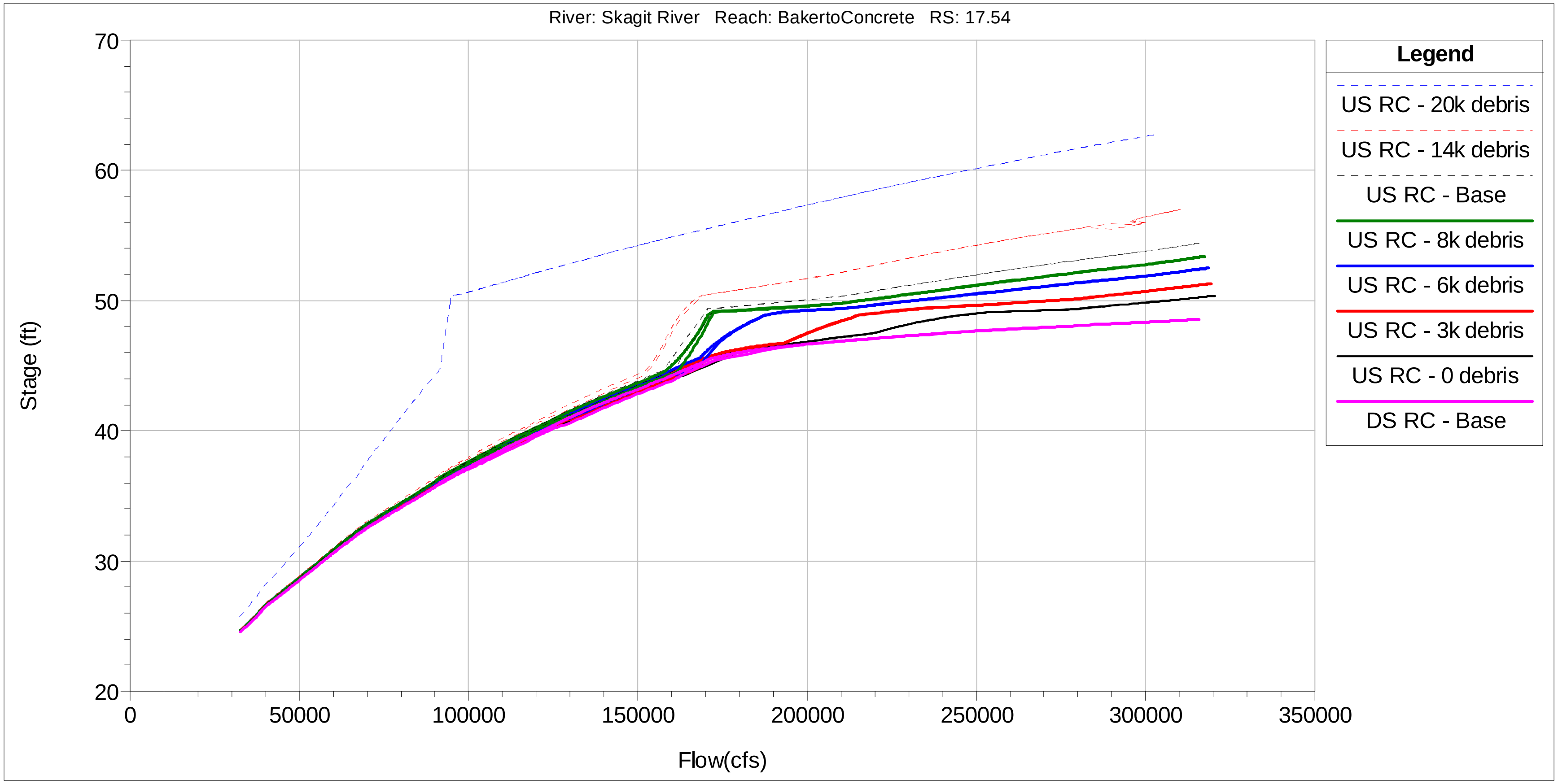


Figure 2-12: BNSF Bridge Rating Curves – Debris Sensitivity.

Notes: Base scenario has 10,000 sq ft of debris. Simulations assume no scour.

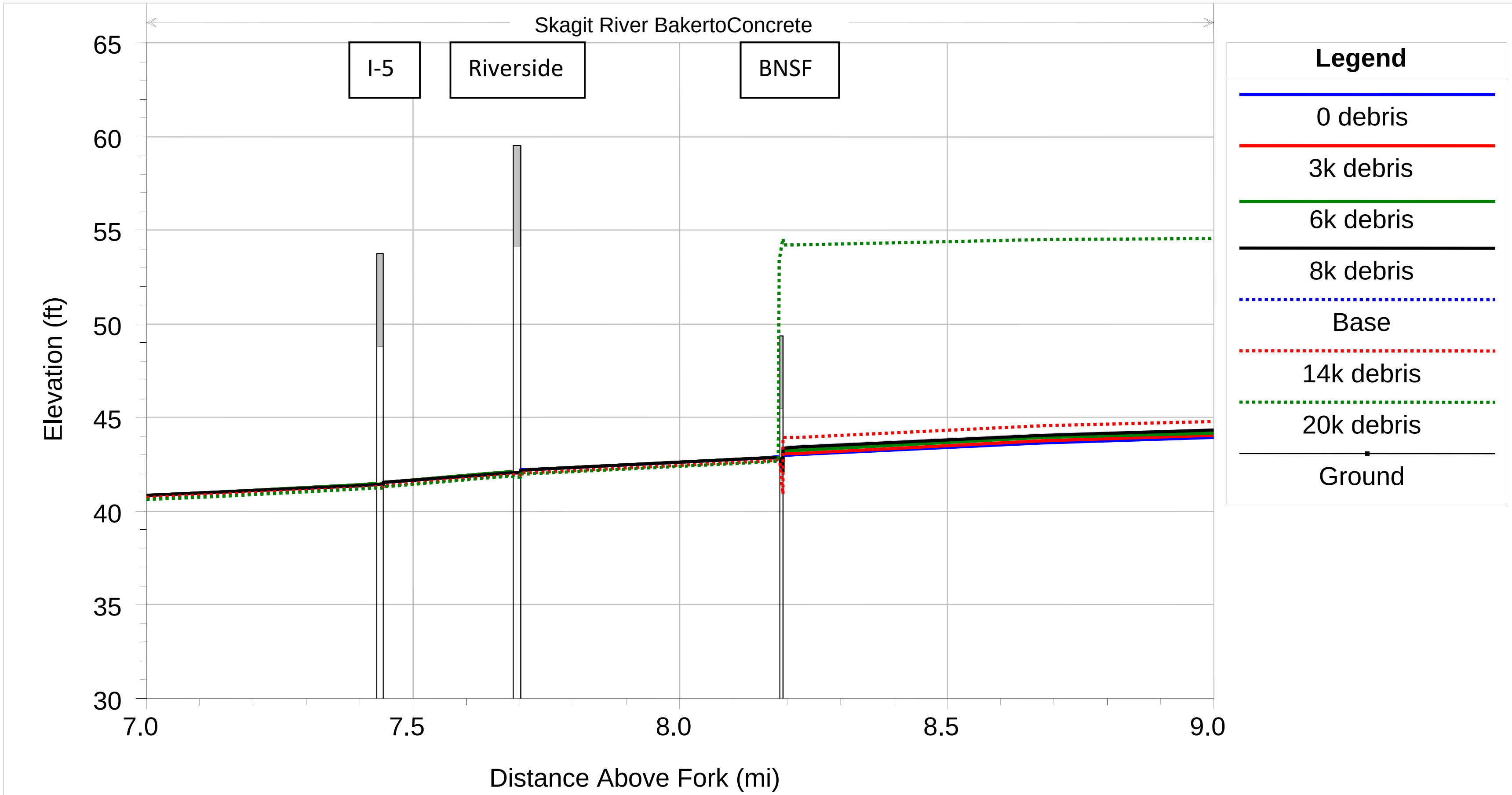


Figure 2-13: 150,000 cfs Water Surface Profiles – Debris Sensitivity.

Notes: Base scenario has 10,000 sq ft of debris. Simulations assume no scour.

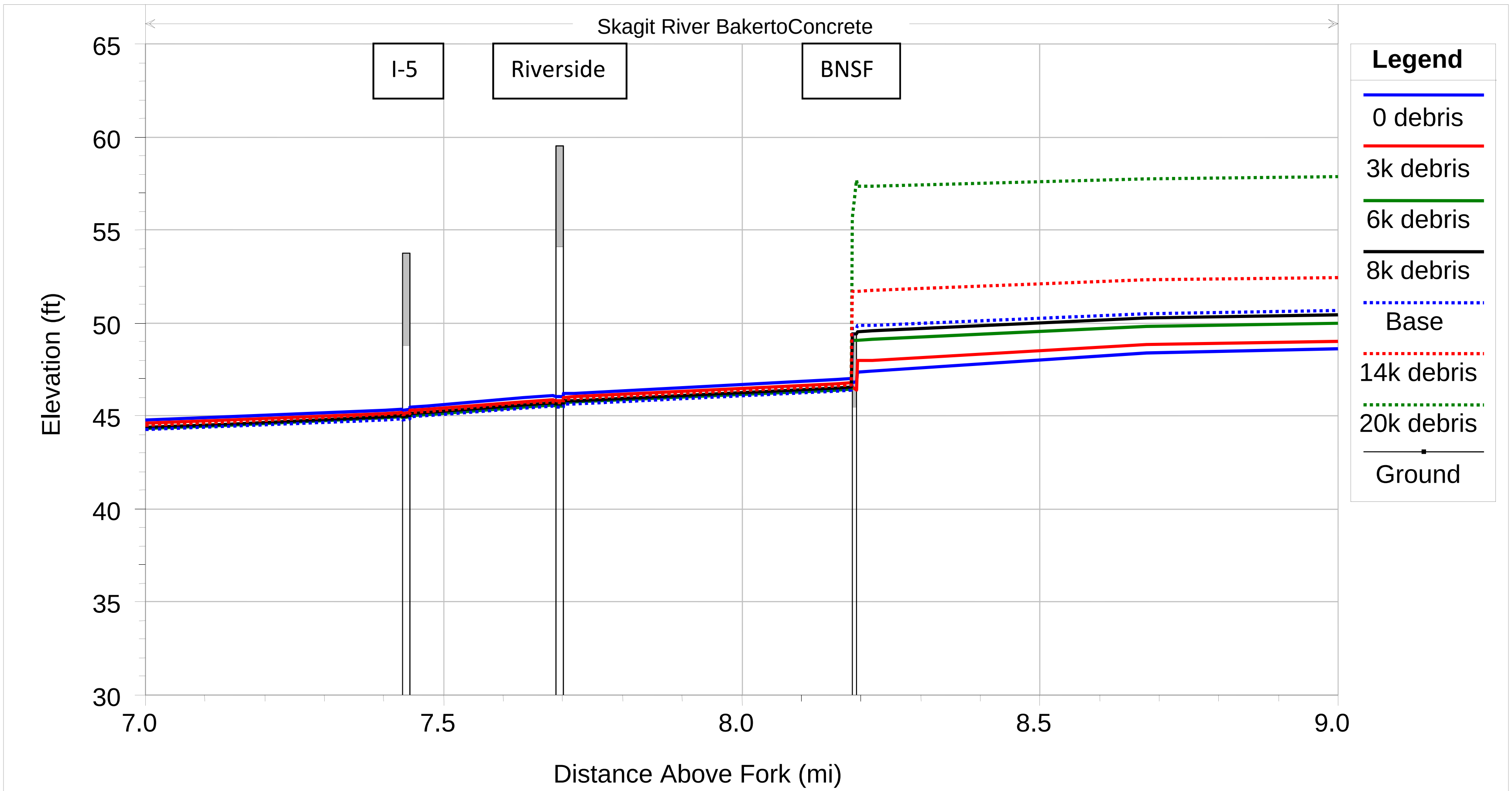


Figure 2-14: 200,000 cfs Water Surface Profiles – Debris Sensitivity.

Notes: Base scenario has 10,000 sq ft of debris. Simulations assume no scour.

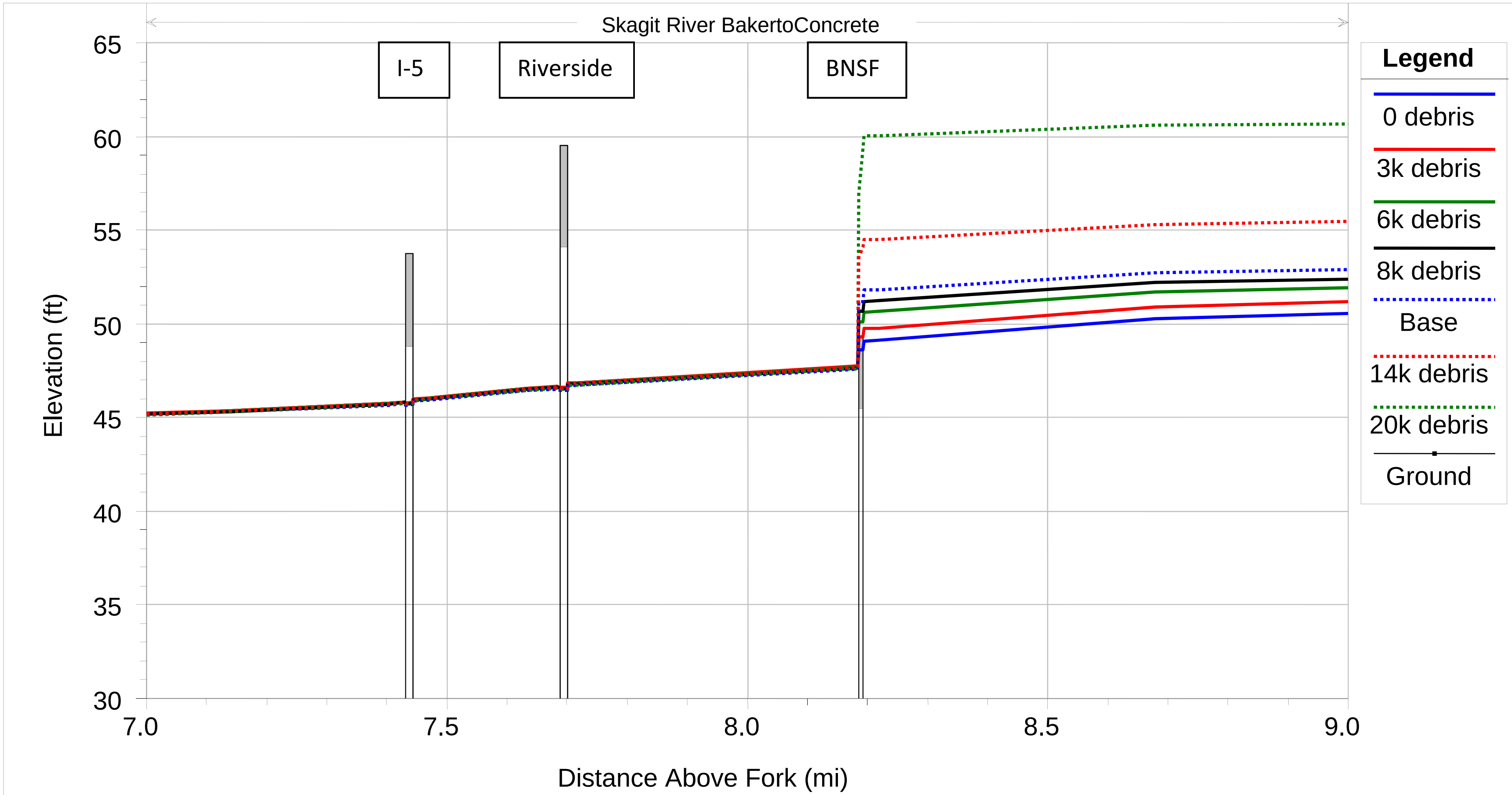


Figure 2-15: 250,000 cfs Water Surface Profiles – Debris Sensitivity.

Notes: Base scenario has 10,000 sq ft of debris. Simulations assume no scour.

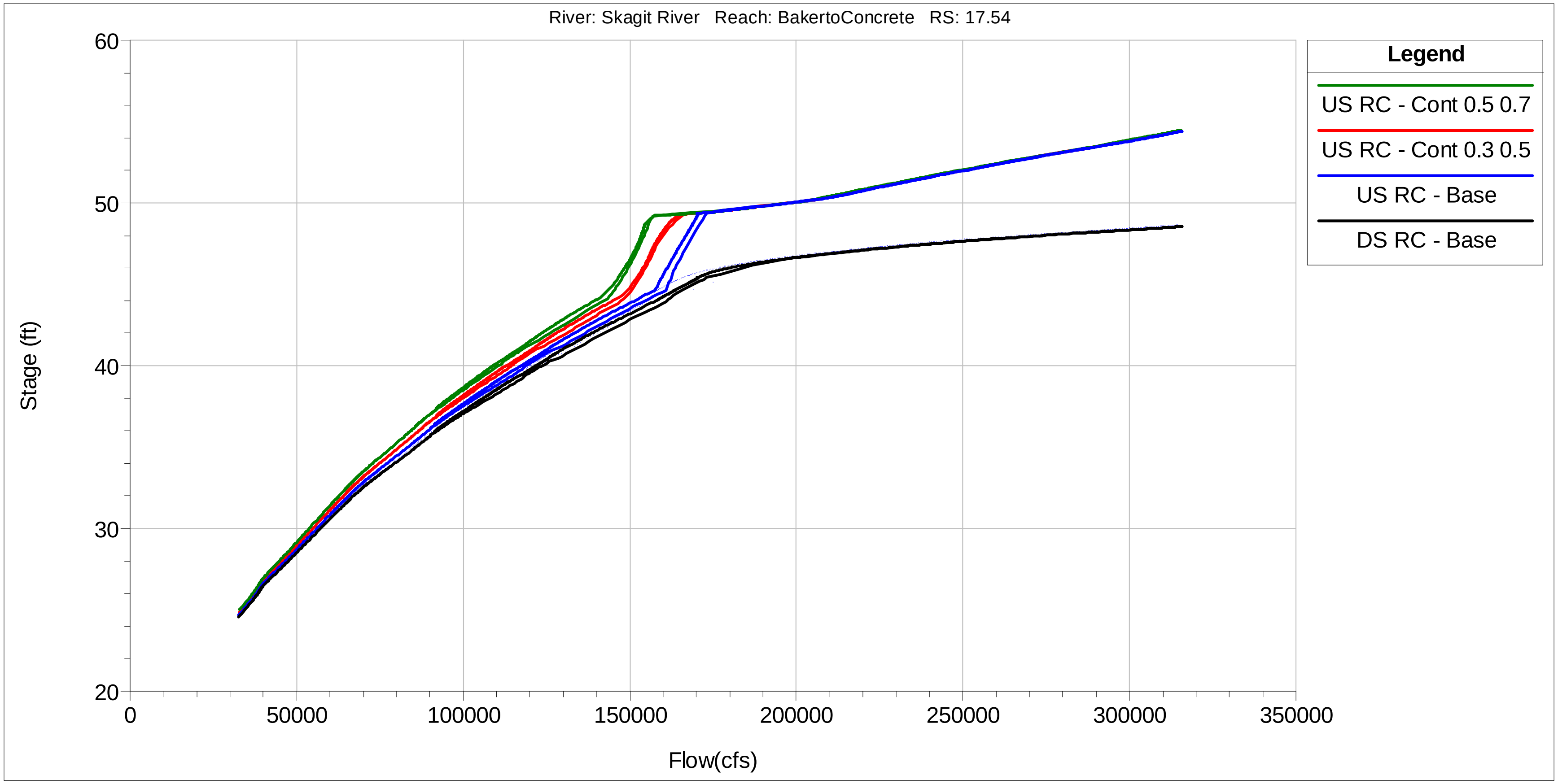


Figure 2-16: BNSF Bridge Rating Curves – Contraction/Expansion Coefficient Sensitivity.

Notes: Base scenario with 0.1/0.3 contraction/expansion coefficients. Simulations assume no scour.

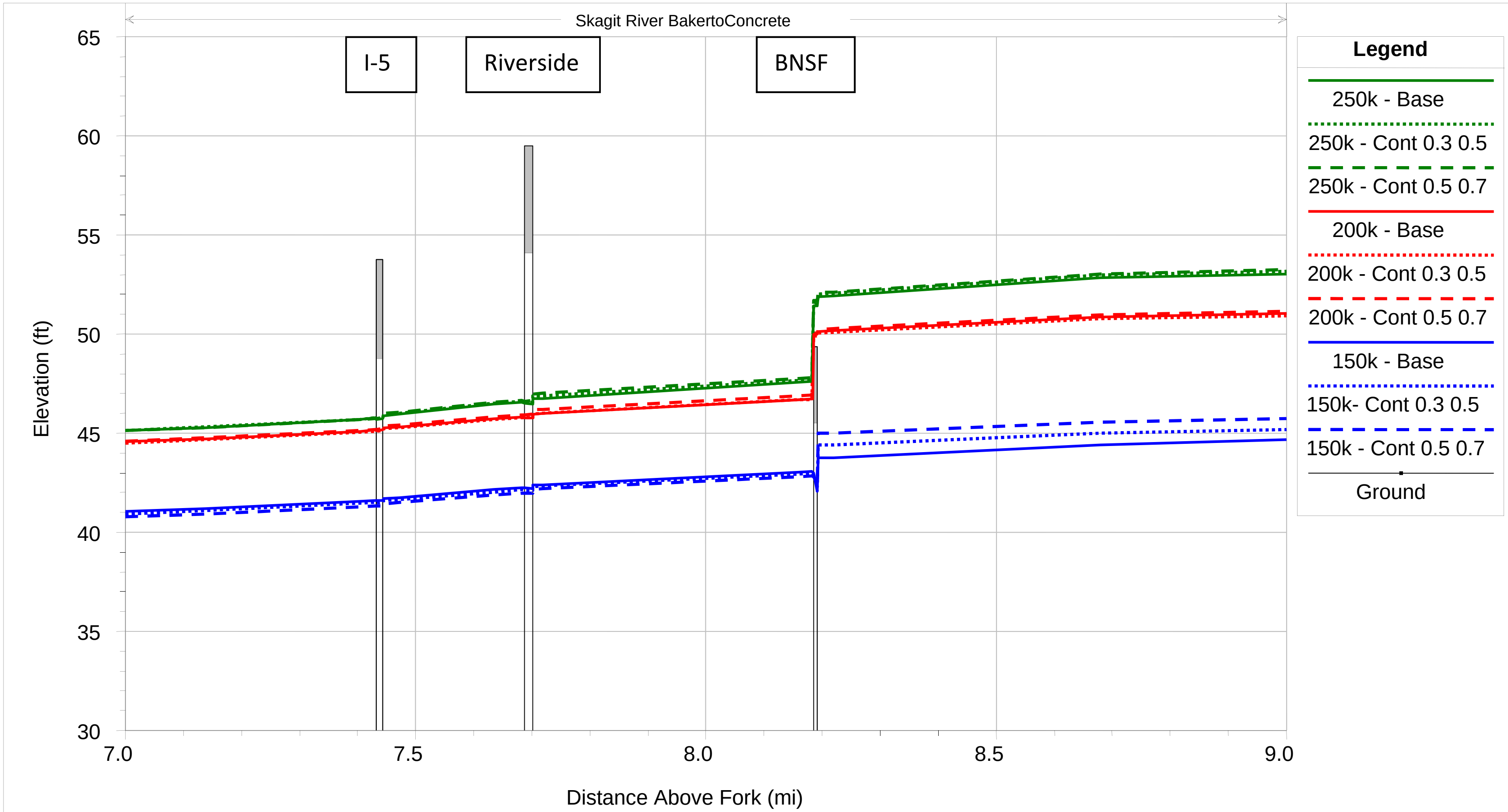


Figure 2-17: 150,000 cfs, 200,000 cfs and 250,000 cfs Water Surface Profiles – Contraction/Expansion Coefficient Sensitivity.

Notes: Base scenario with 0.1/0.3 contraction/expansion coefficients. Simulations assume no scour.

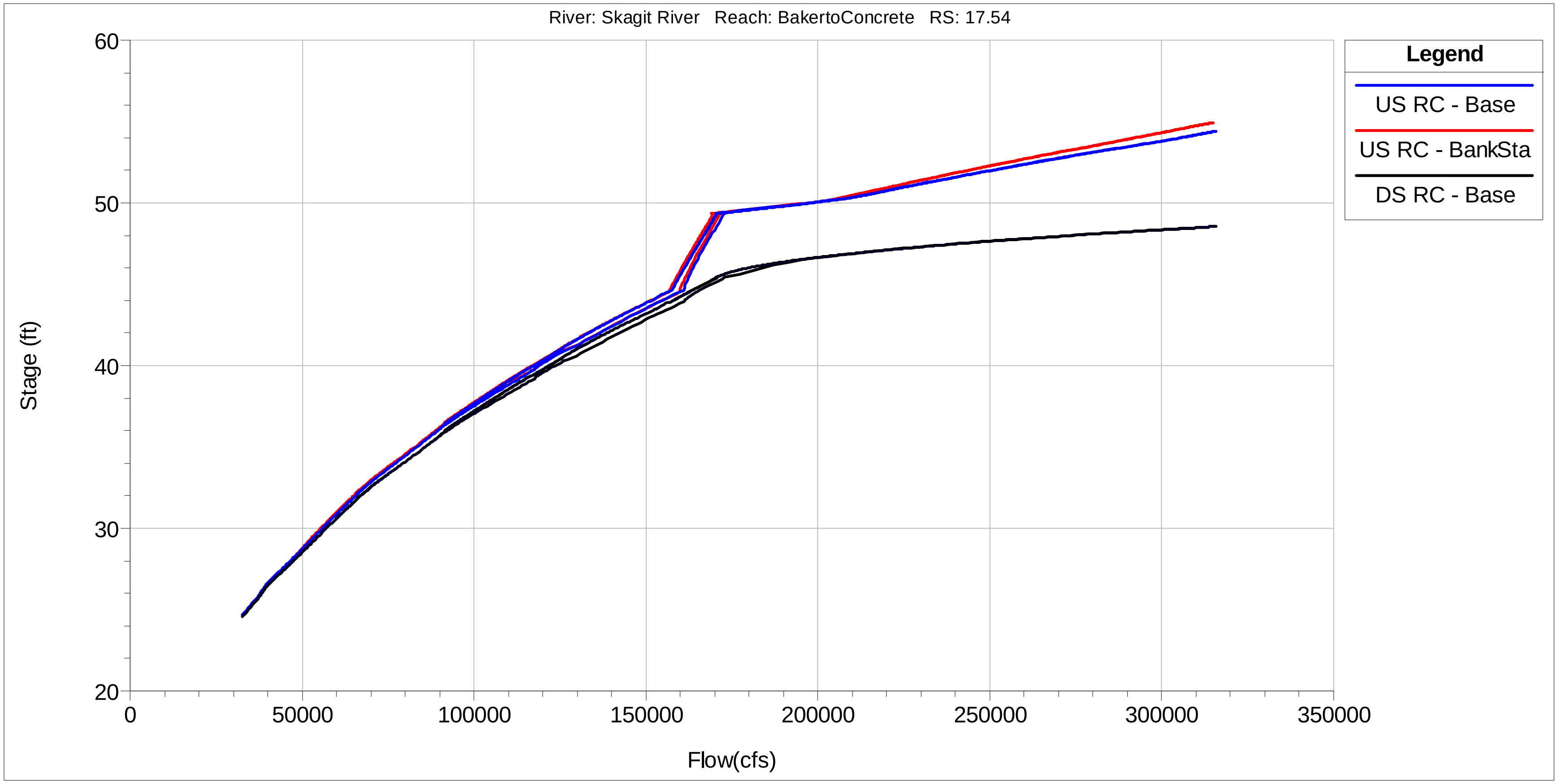


Figure 2-18: BNSF Bridge Rating Curves – Bank Station Sensitivity.

Notes: Base scenario with right bank station at edge of low flow channel. Simulations assume no scour.

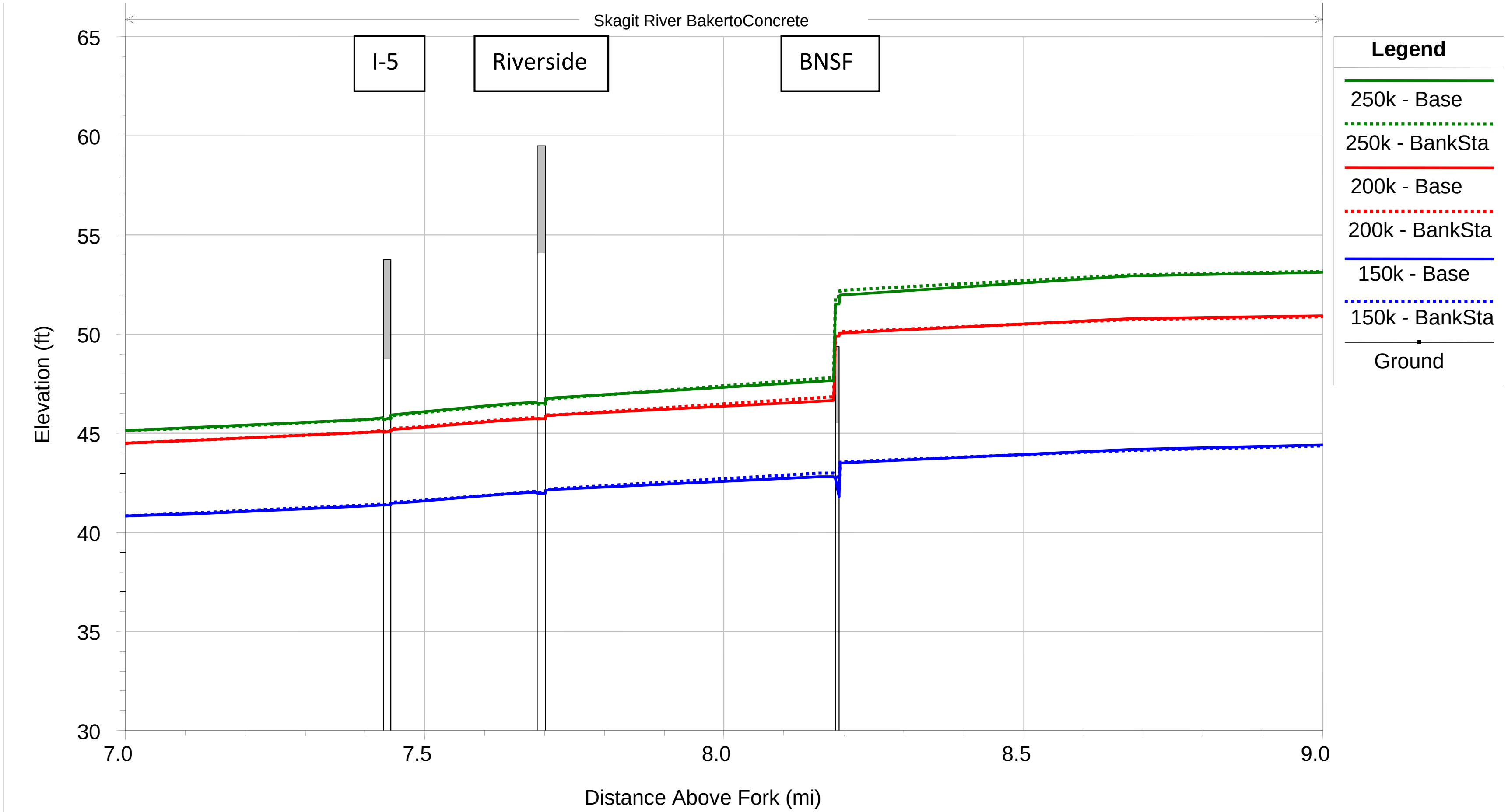


Figure 2-19: 150,000 cfs, 200,000 cfs and 250,000 cfs Water Surface Profiles – Bank Station Sensitivity.

Notes: Base scenario with right bank station at edge of low flow channel. Simulations assume no scour.

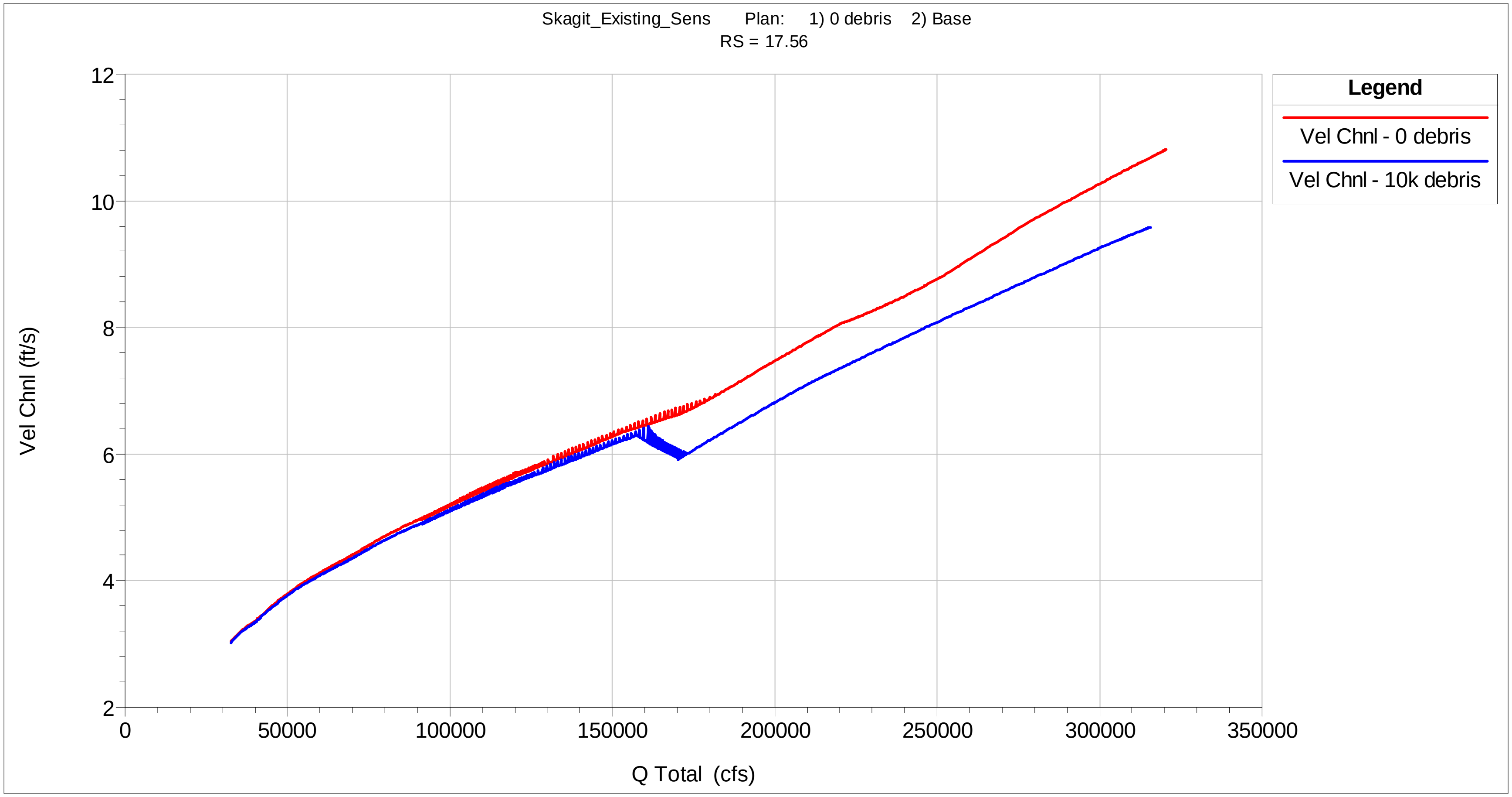


Figure 2-20: Channel Approach Velocity

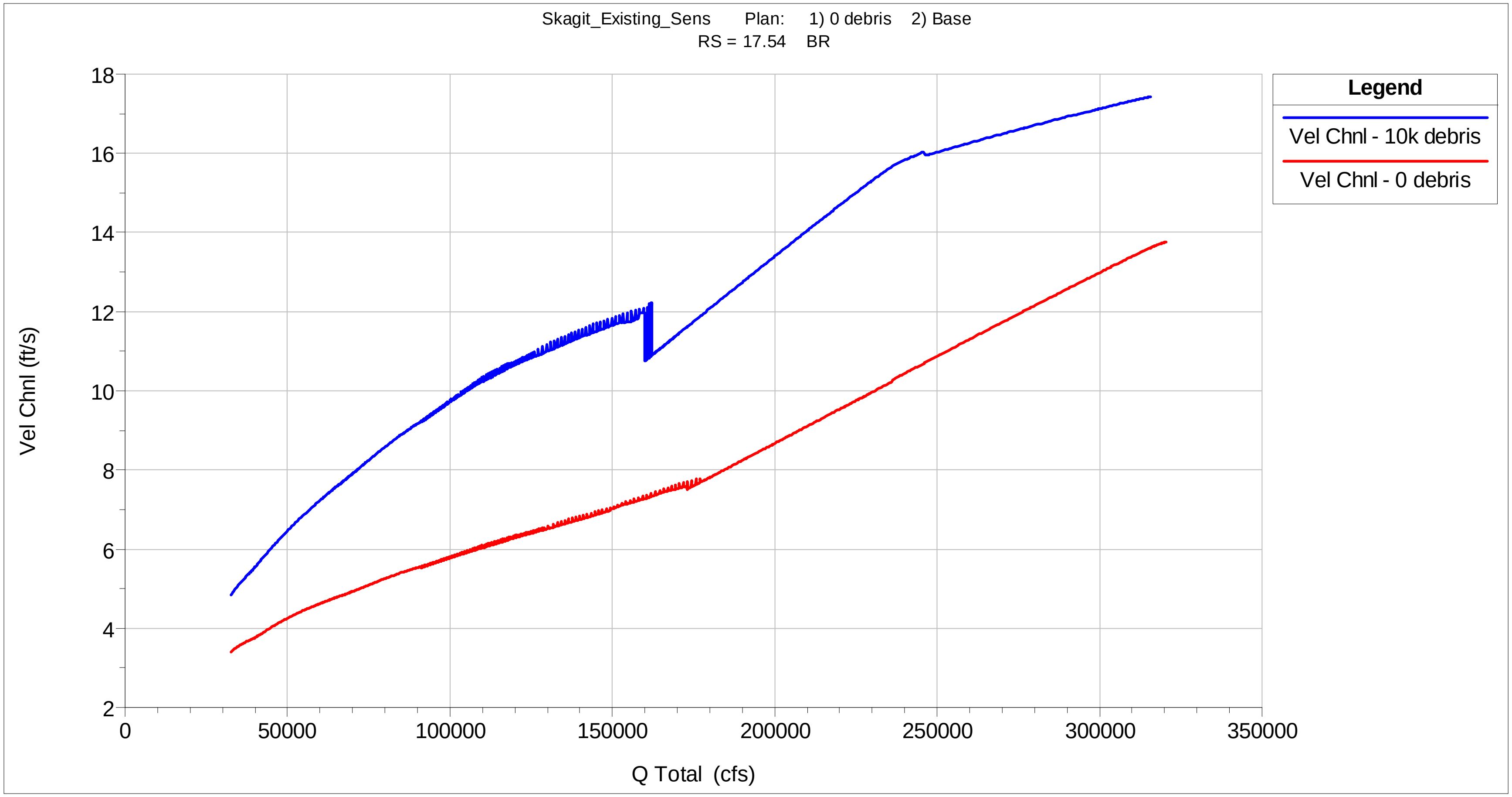


Figure 2-21: Bridge Opening Channel Velocity

Figure 2-22: BNSF Bridge at low flow (1993 – source and exact date unknown)

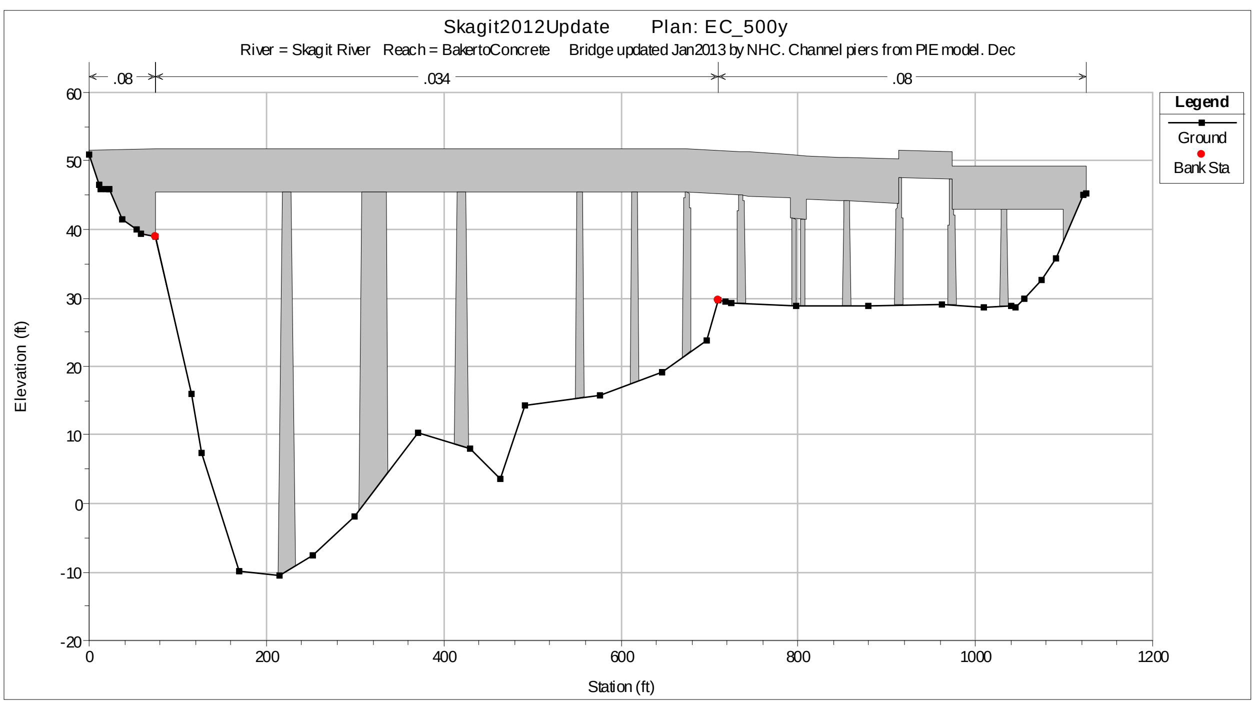


Figure 2-23: Final BNSF bridge geometry after January 2013 refinements

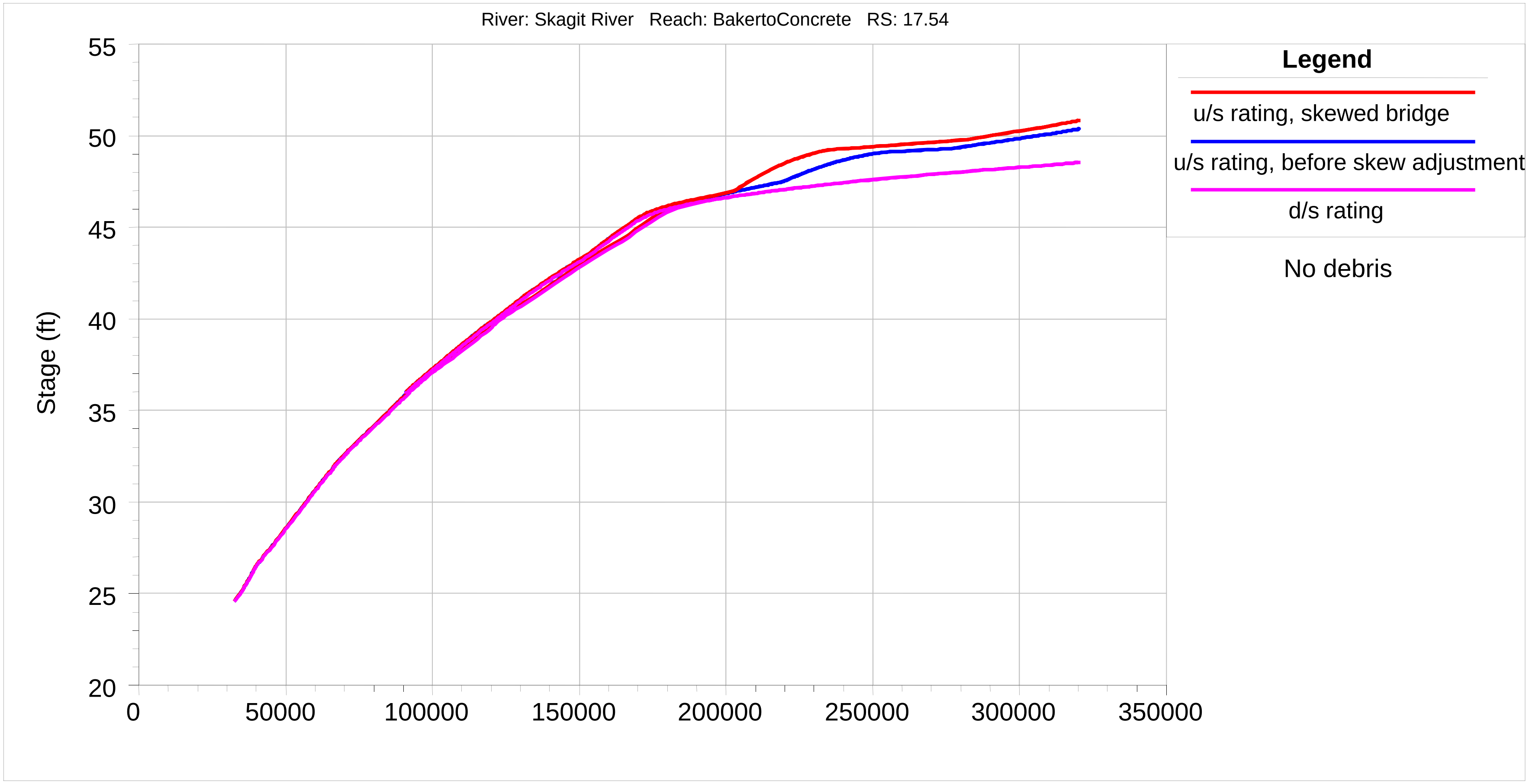


Figure 2-24: BNSF Bridge Rating Curves – With and Without Skew Adjustment; No Debris.

Note: Simulations assume no scour.

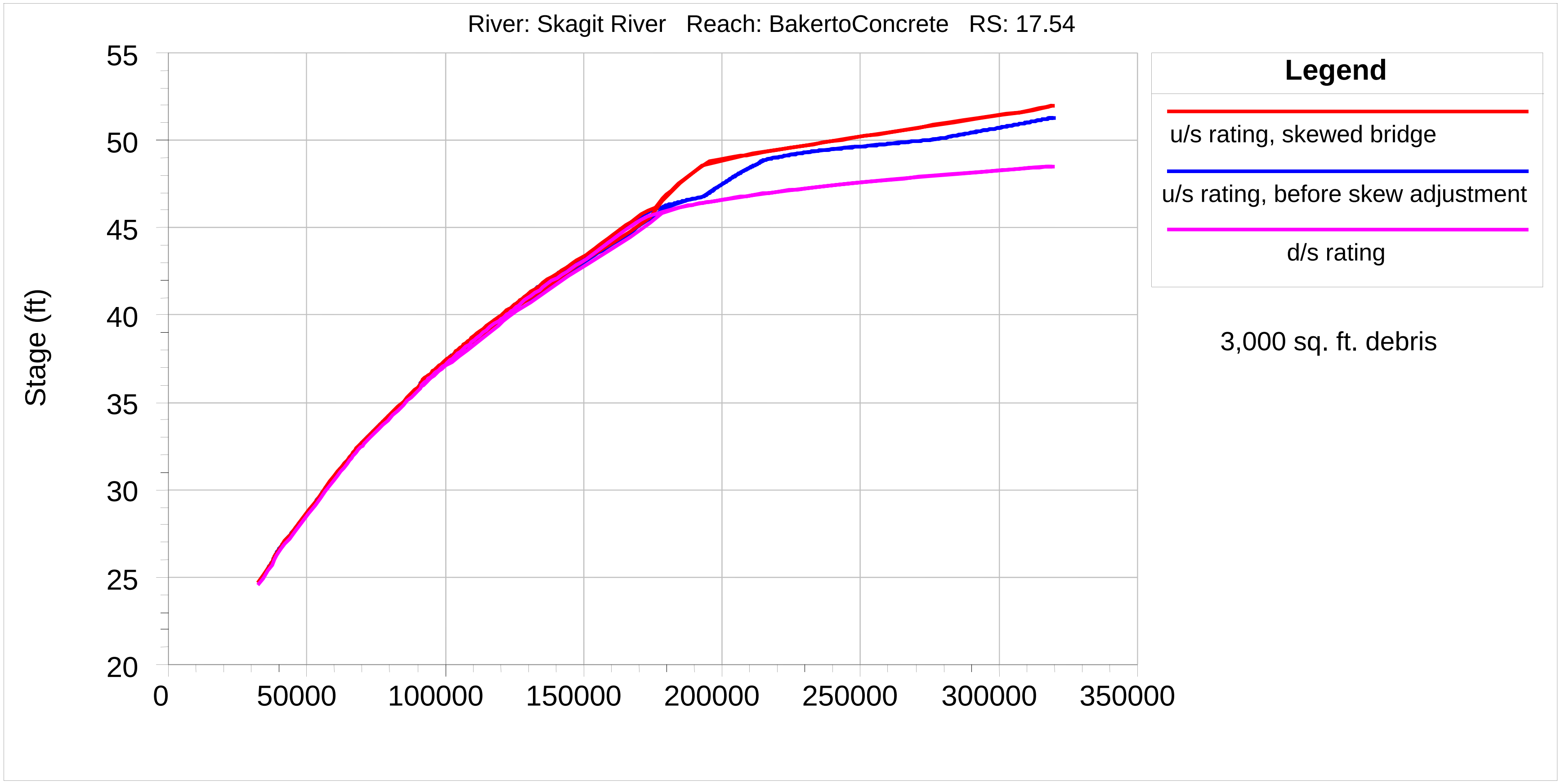


Figure 2-25: BNSF Bridge Rating Curves – With and Without Skew Adjustment; 3,000 sq. ft. of Debris.

Note: Simulations assume no scour.

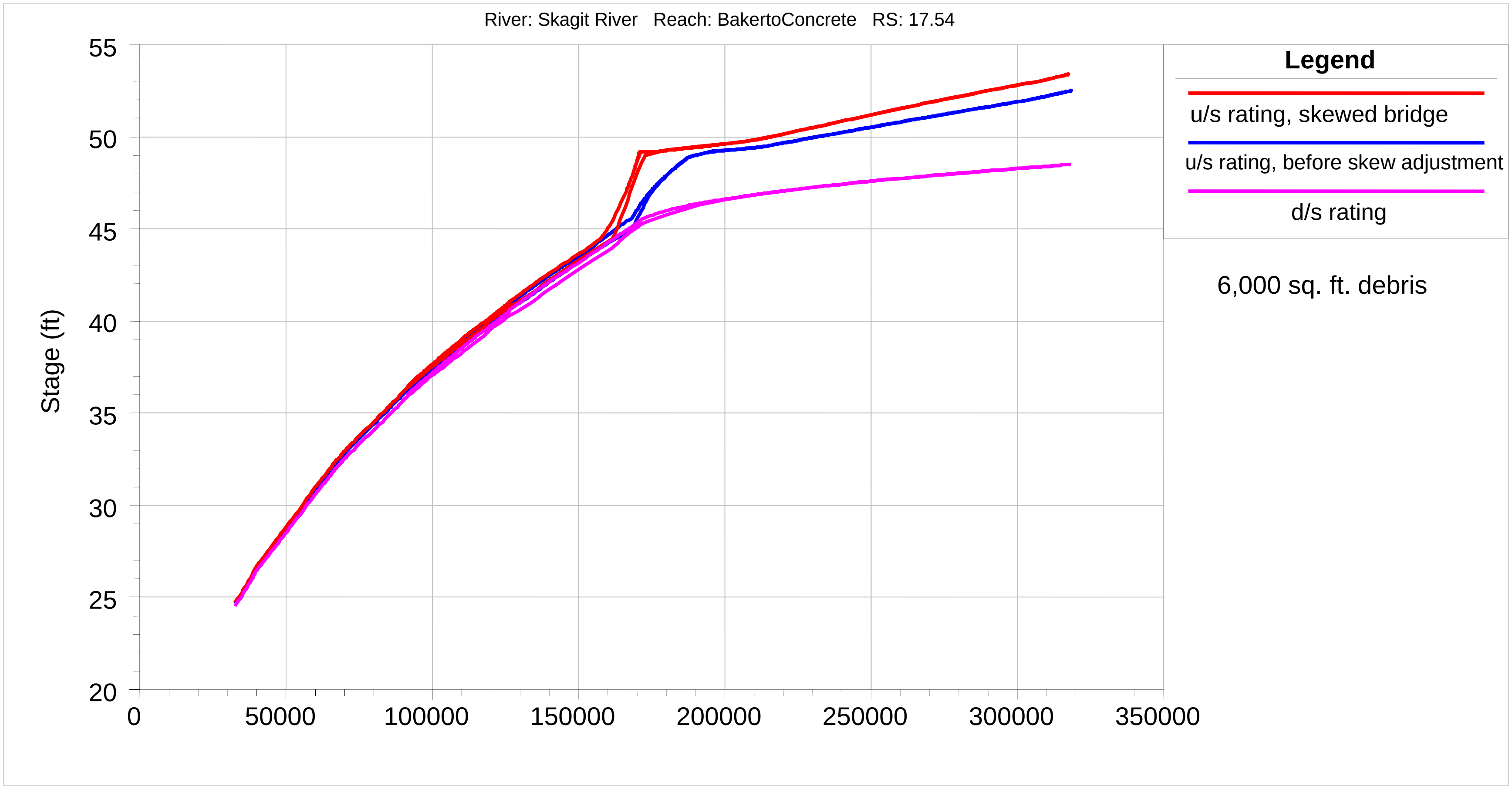


Figure 2-26: BNSF Bridge Rating Curves – With and Without Skew Adjustment; 6,000 sq. ft. of Debris.

Note: Simulations assume no scour.

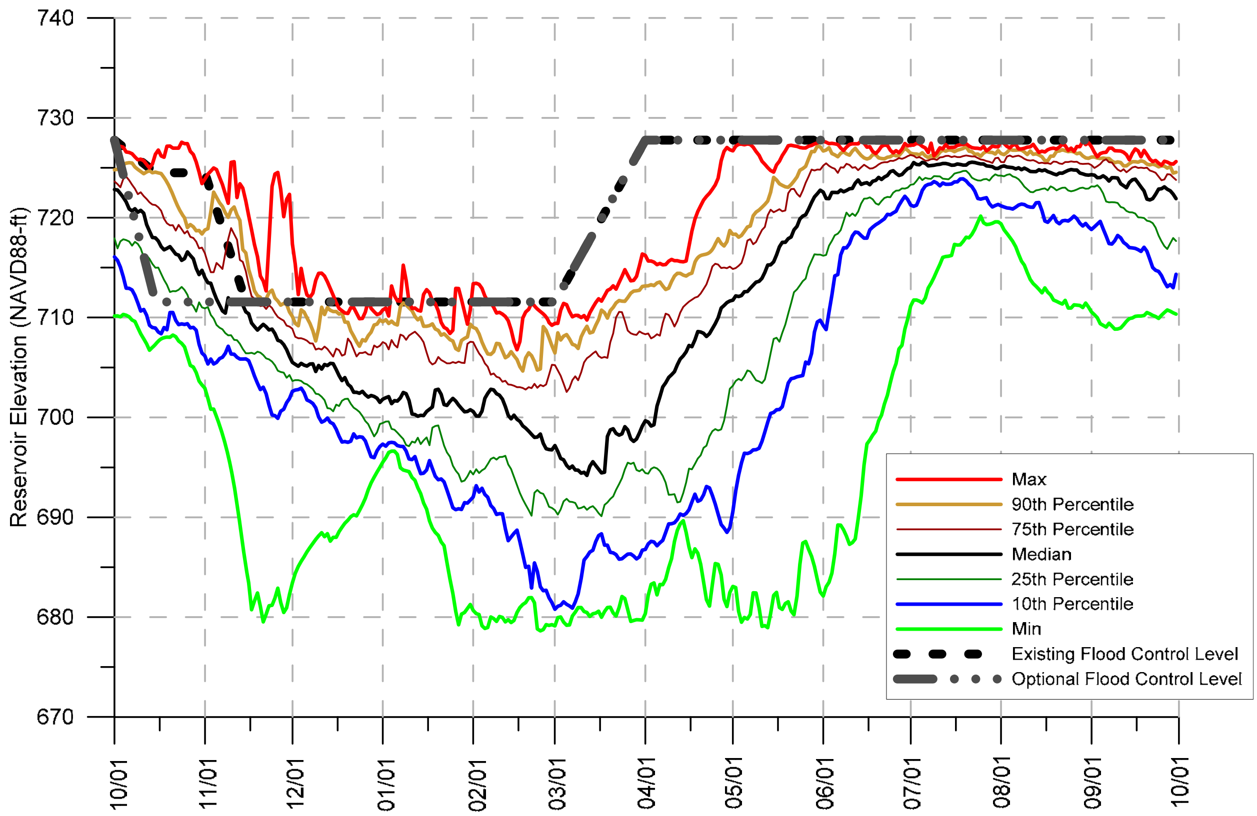


Figure 3-1: Upper Baker Reservoir Elevation Summary Hydrographs (Water Years 1984 to 2003).

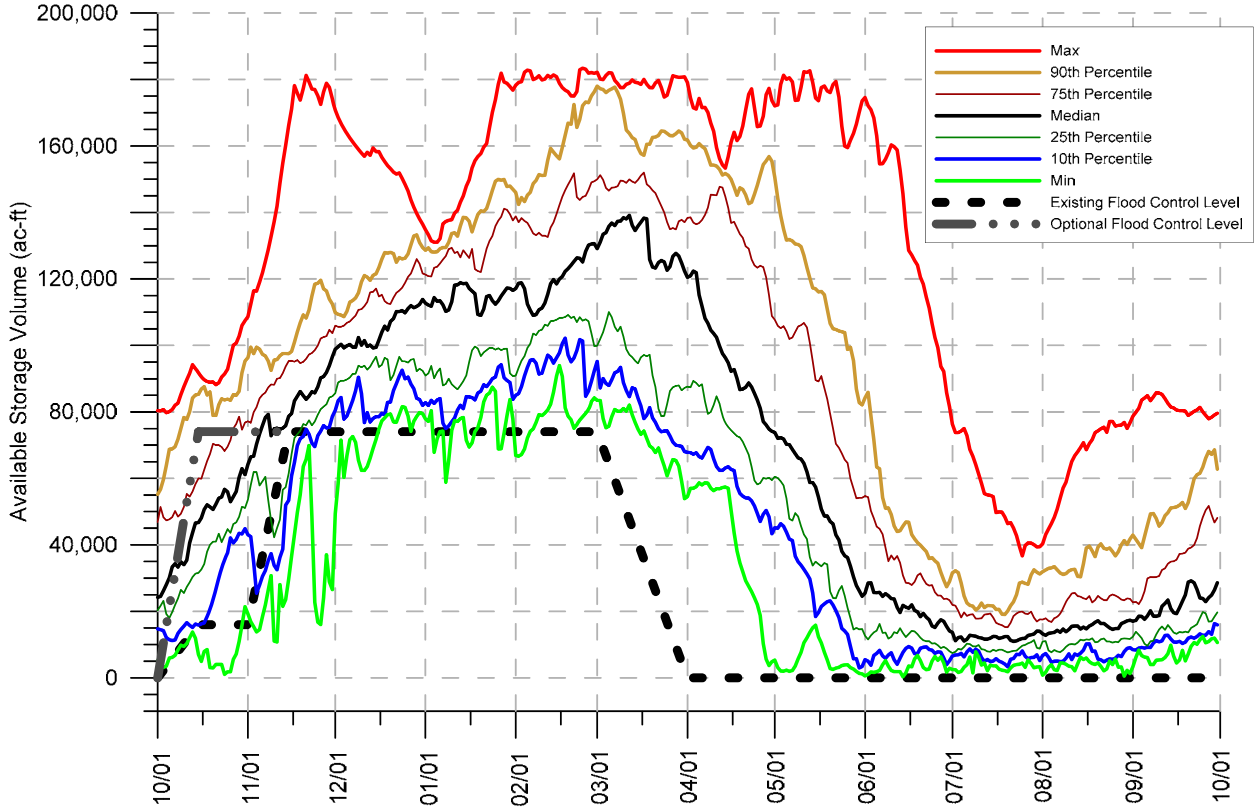


Figure 3-2: Upper Baker Reservoir Storage Volume Summary Hydrographs (Water Years 1984 to 2003).

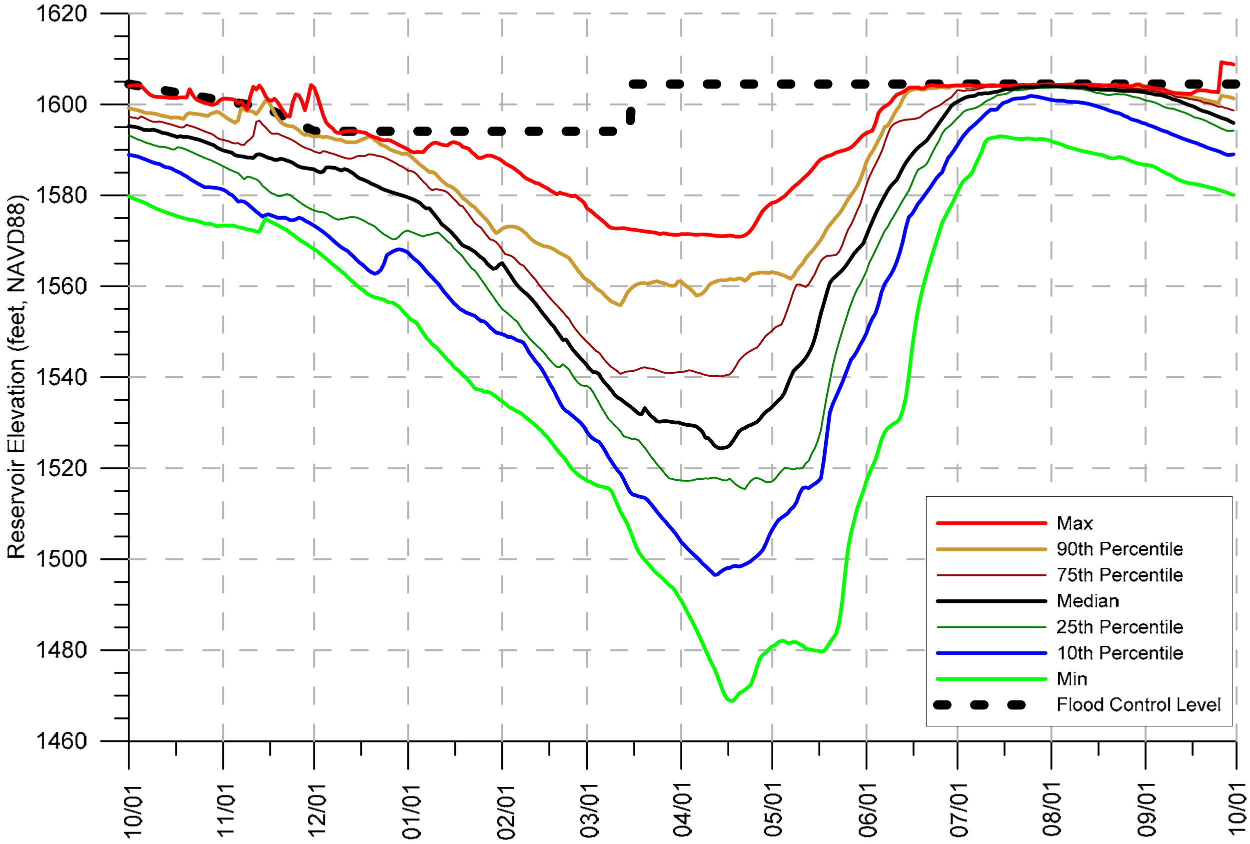


Figure 3-3: Ross Reservoir Elevation Summary Hydrographs (Water Years 1990 to 2009).

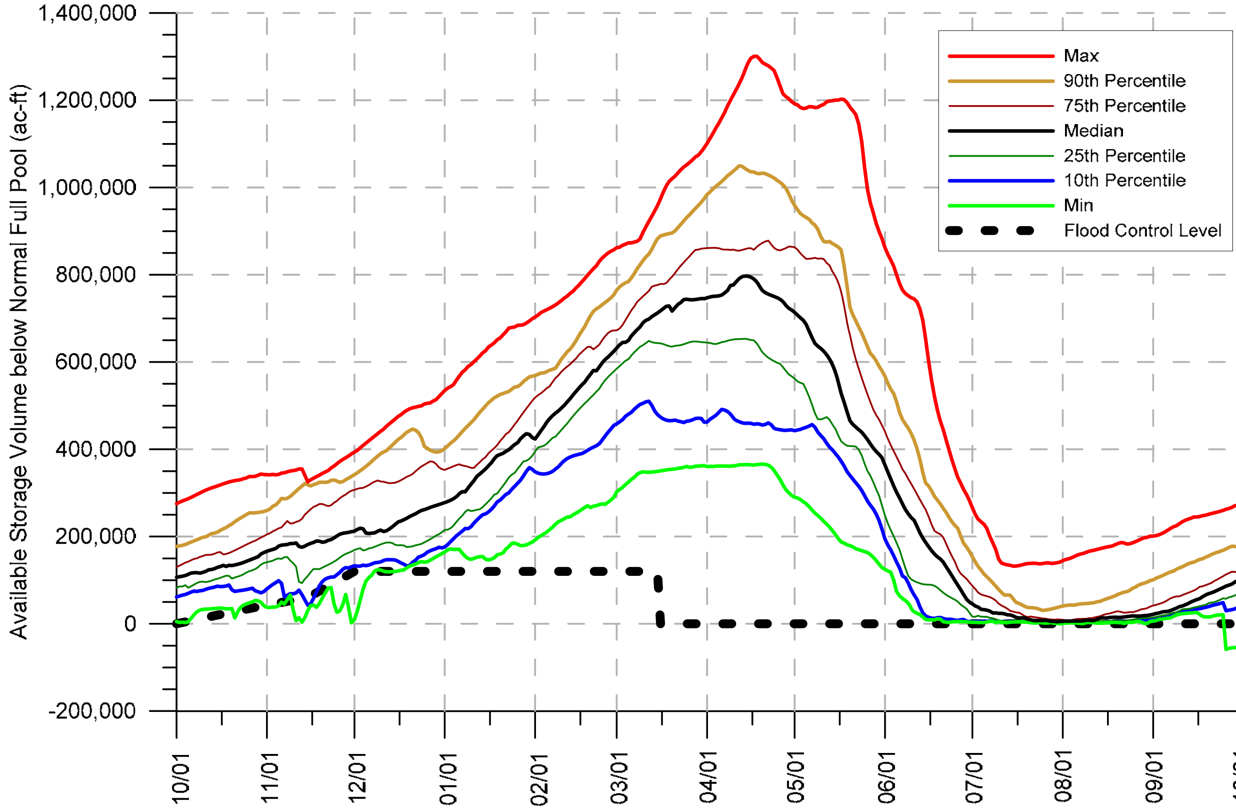


Figure 3-4: Ross Reservoir Storage Volume Summary Hydrographs (Water Years 1990 to 2009).

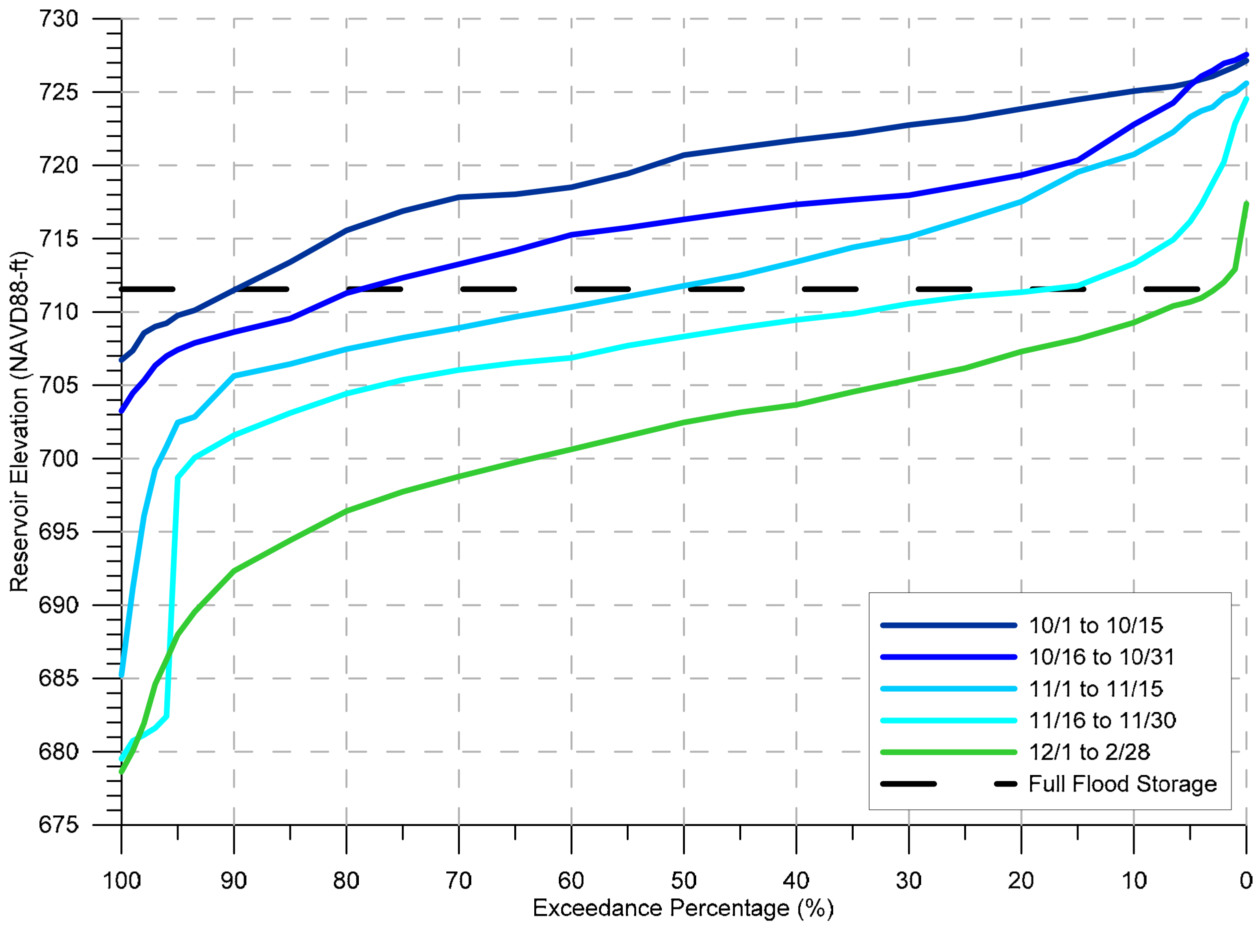


Figure 3-5: Upper Baker Reservoir Elevation Duration Curves (Water Years 1984 to 2003).

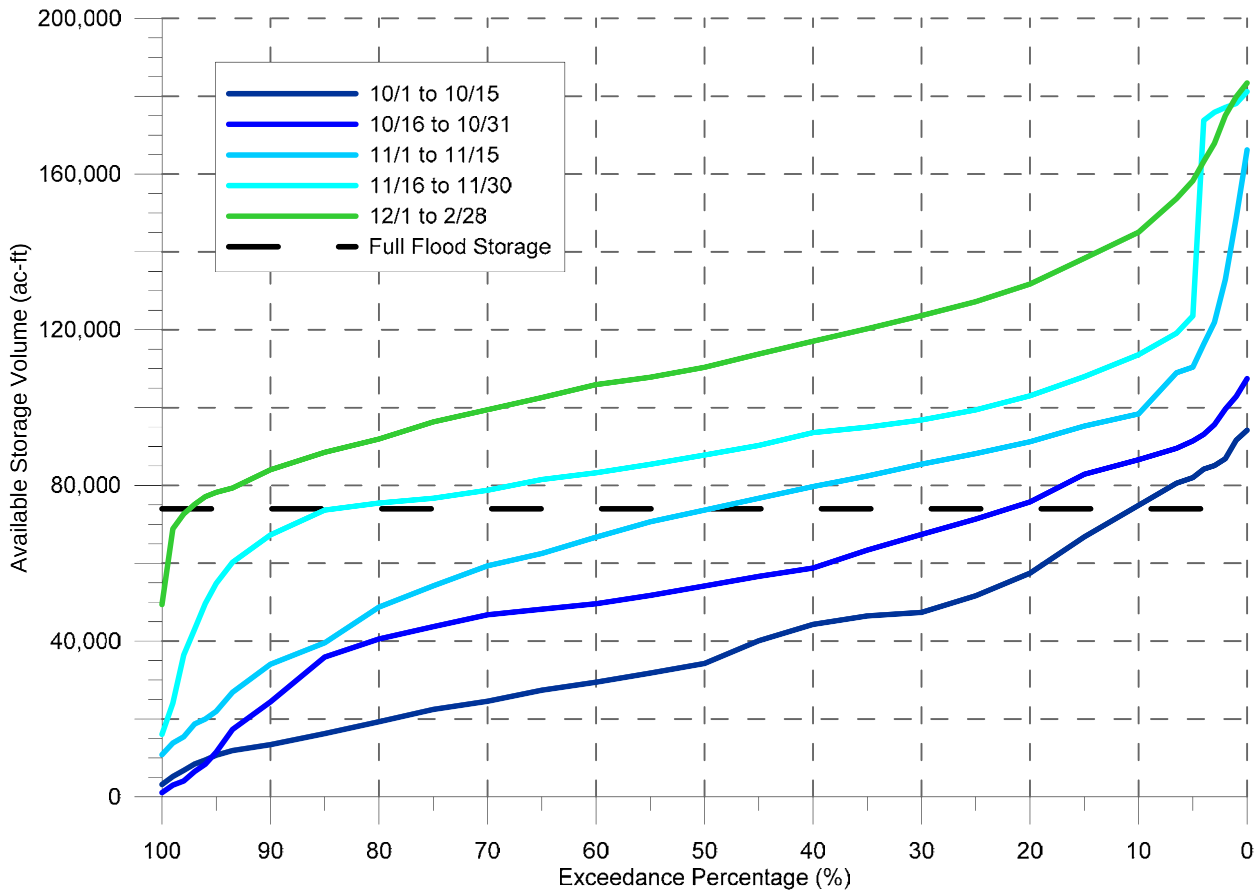


Figure 3-6: Upper Baker Reservoir Storage Volume Duration Curves (Water Years 1984 to 2003).

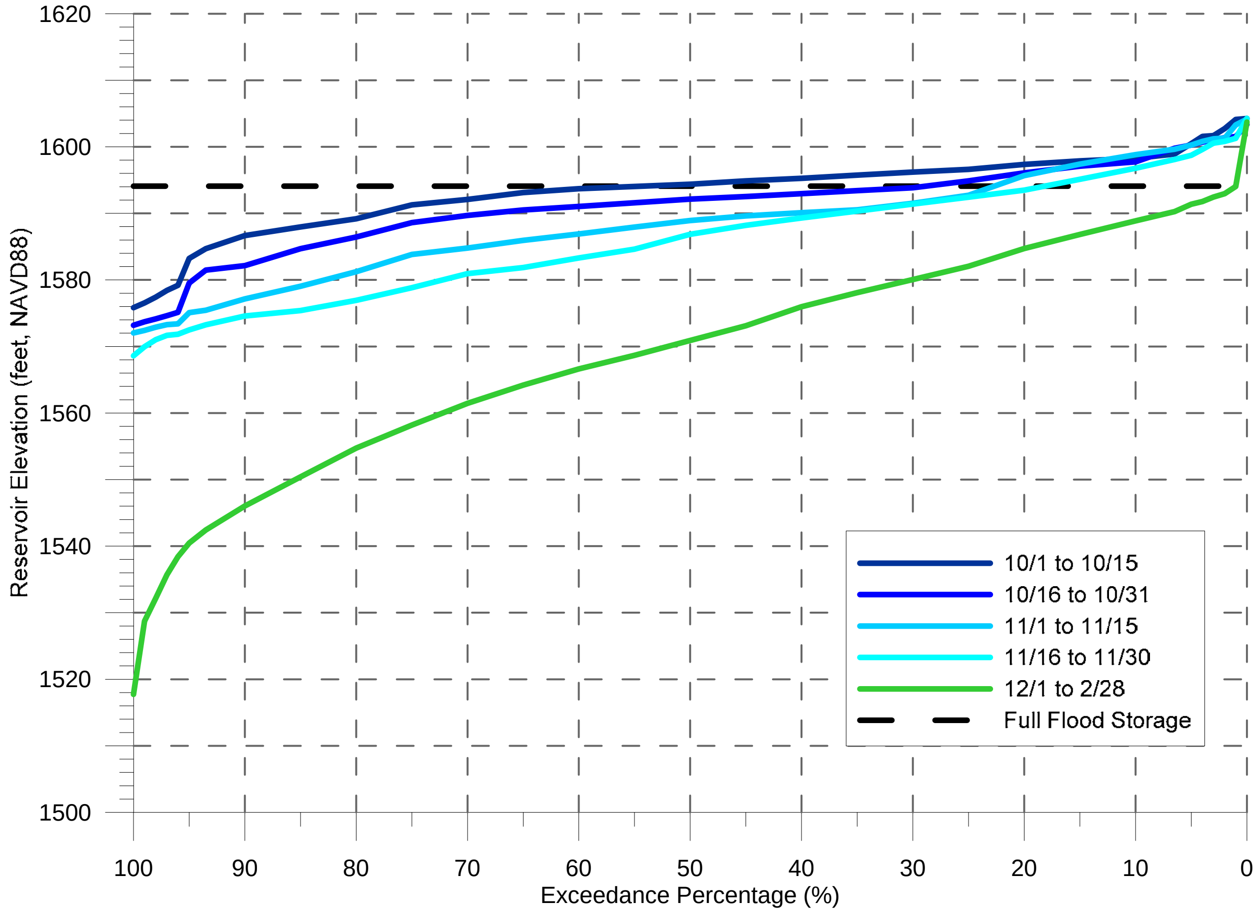


Figure 3-7: Ross Reservoir Elevation Duration Curves (Water Years 1990 to 2009).

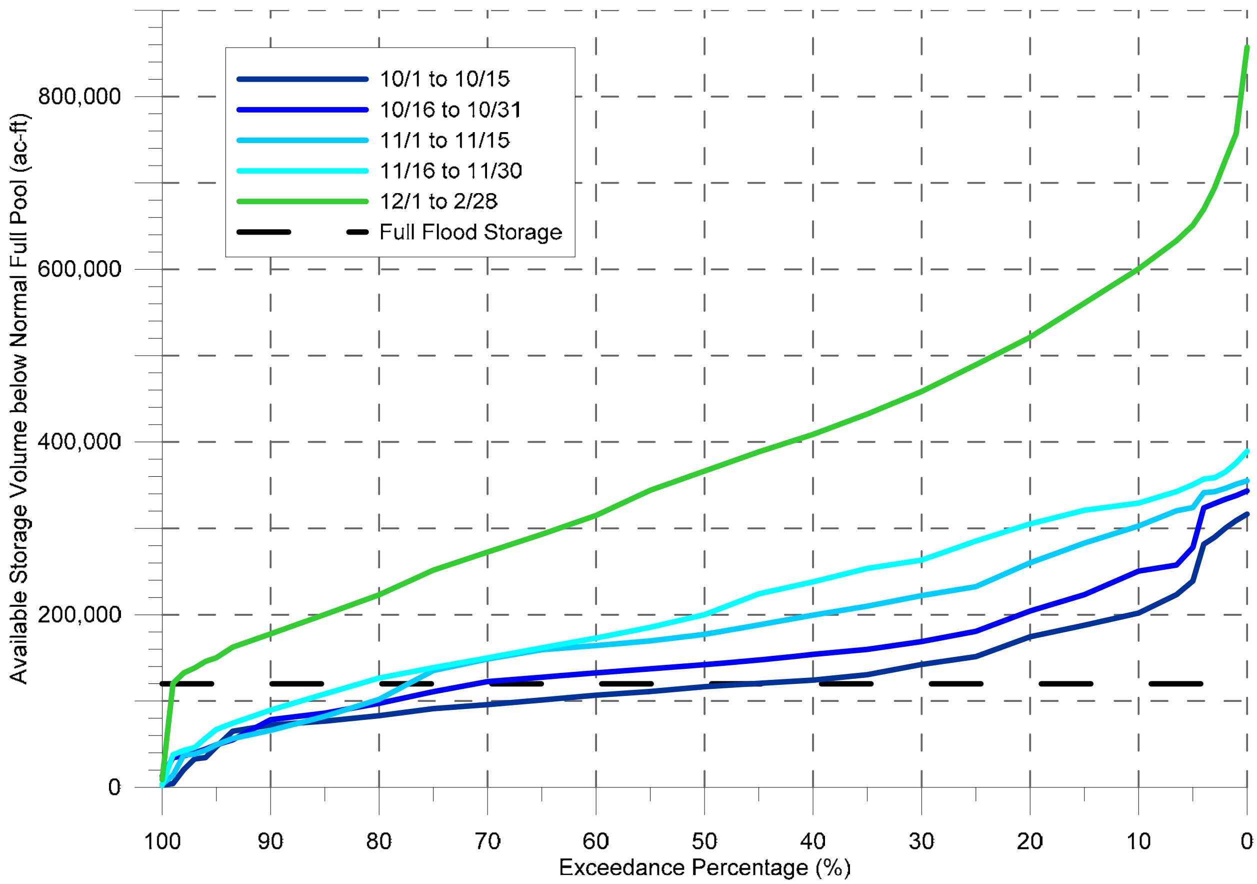


Figure 3-8: Ross Reservoir Storage Volume Duration Curves (Water Years 1990 to 2009).

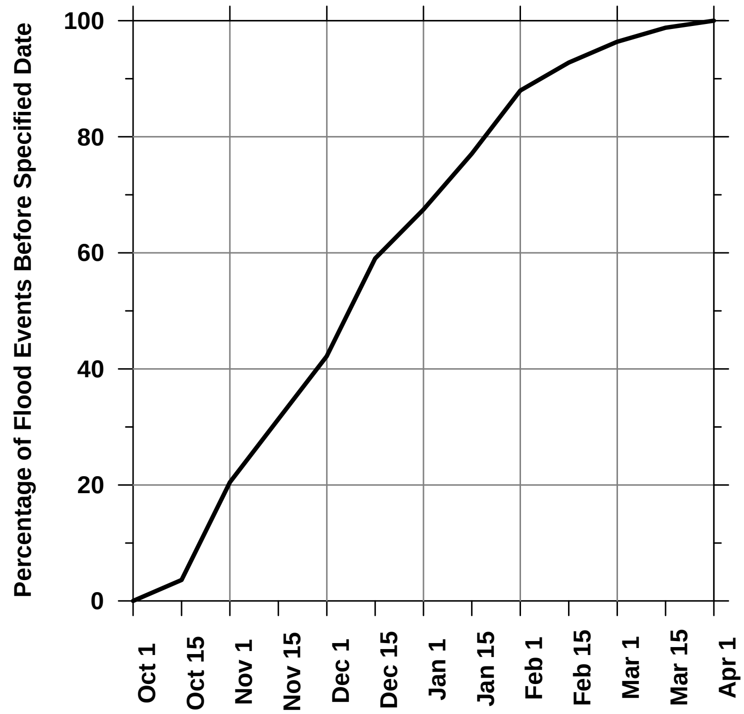


Figure 3-9: Cumulative seasonal distribution of winter floods

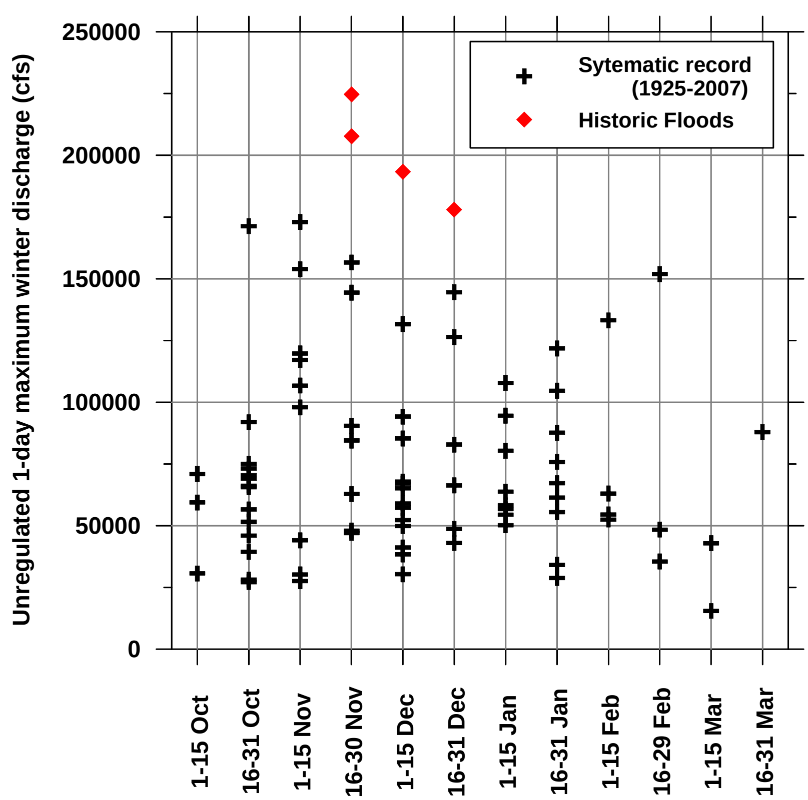


Figure 3-10: Magnitude and seasonal distribution of winter floods

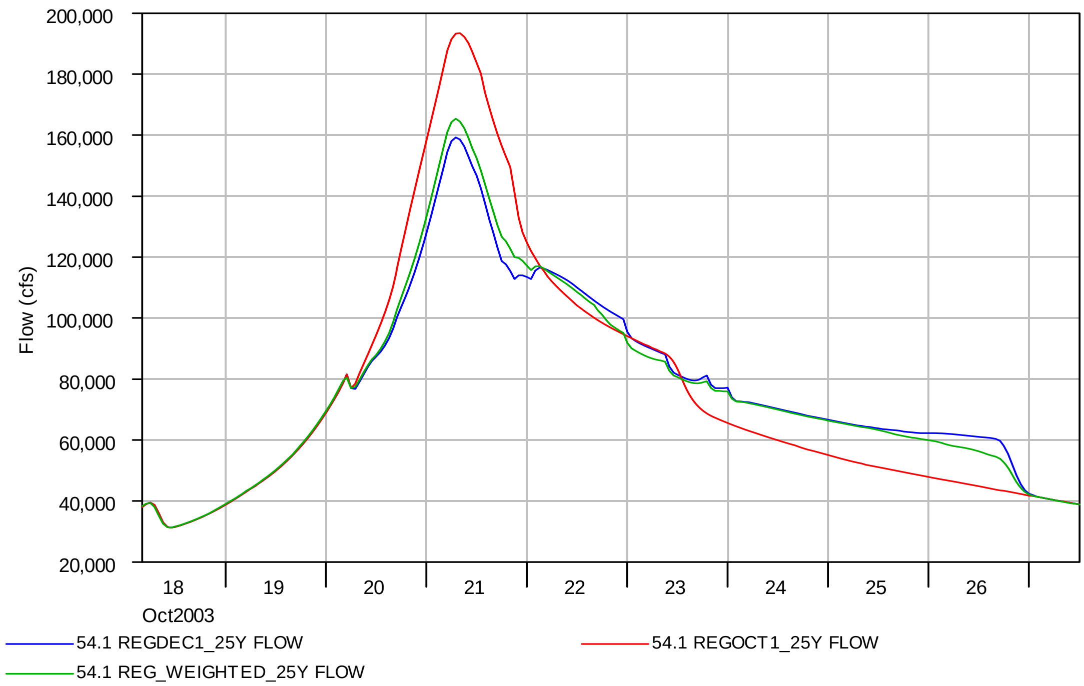


Figure 3-11: Existing regulation 1 December, 1 October and weighted 25-year hydrographs, Skagit River near Concrete

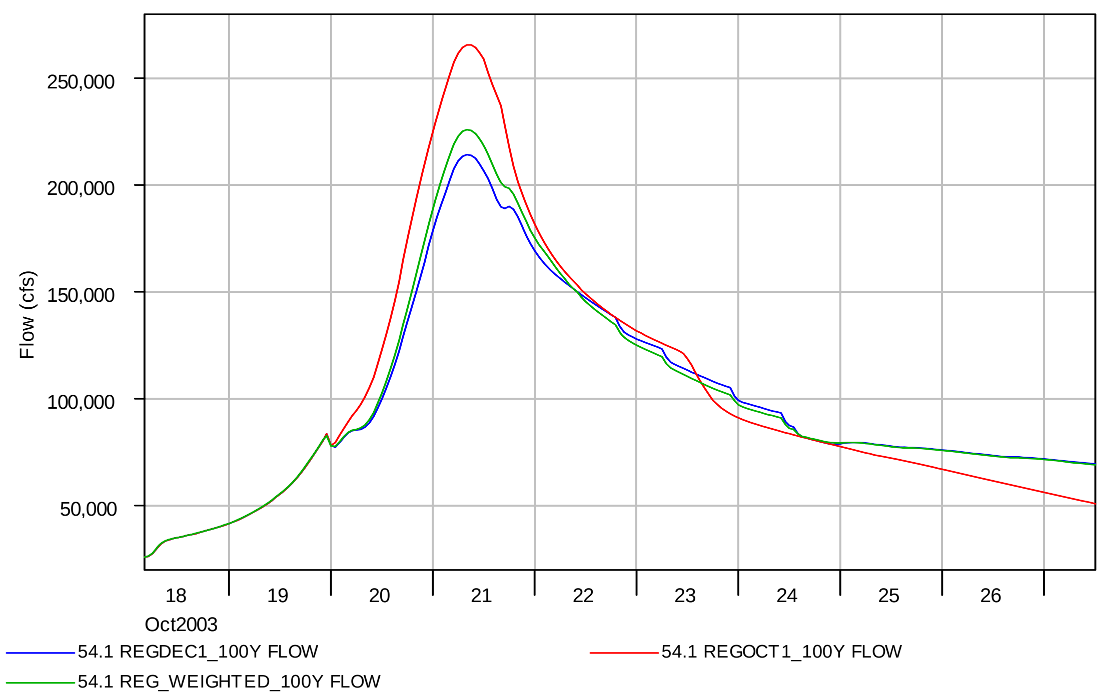


Figure 3-12: Existing regulation 1 December, 1 October and weighted 100-year hydrographs, Skagit River near Concrete, under existing conditions

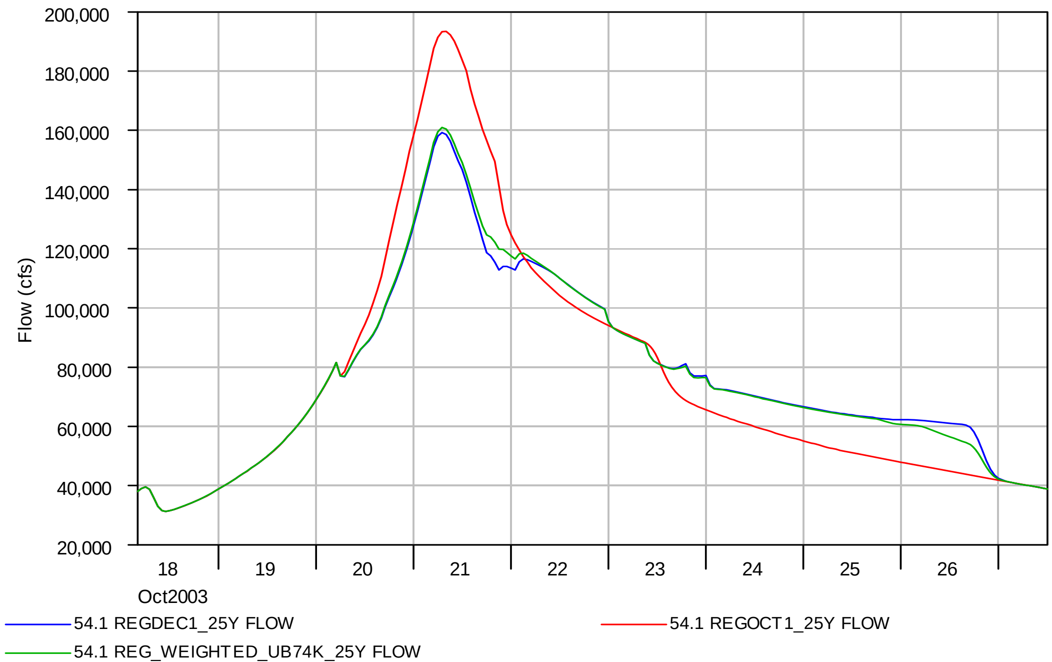


Figure 3-13: Optional regulation 1 December, 1 October and weighted 25-year hydrographs, Skagit River near Concrete

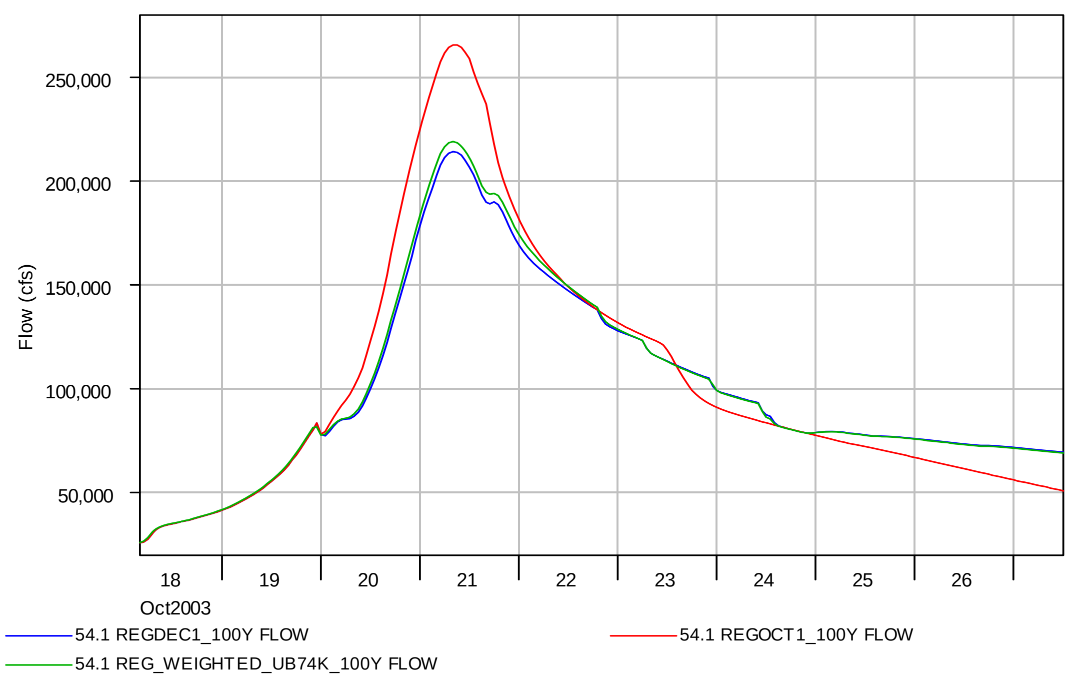


Figure 3-14: Optional regulation 1 December, 1 October and weighted 100-year hydrographs, Skagit River near Concrete

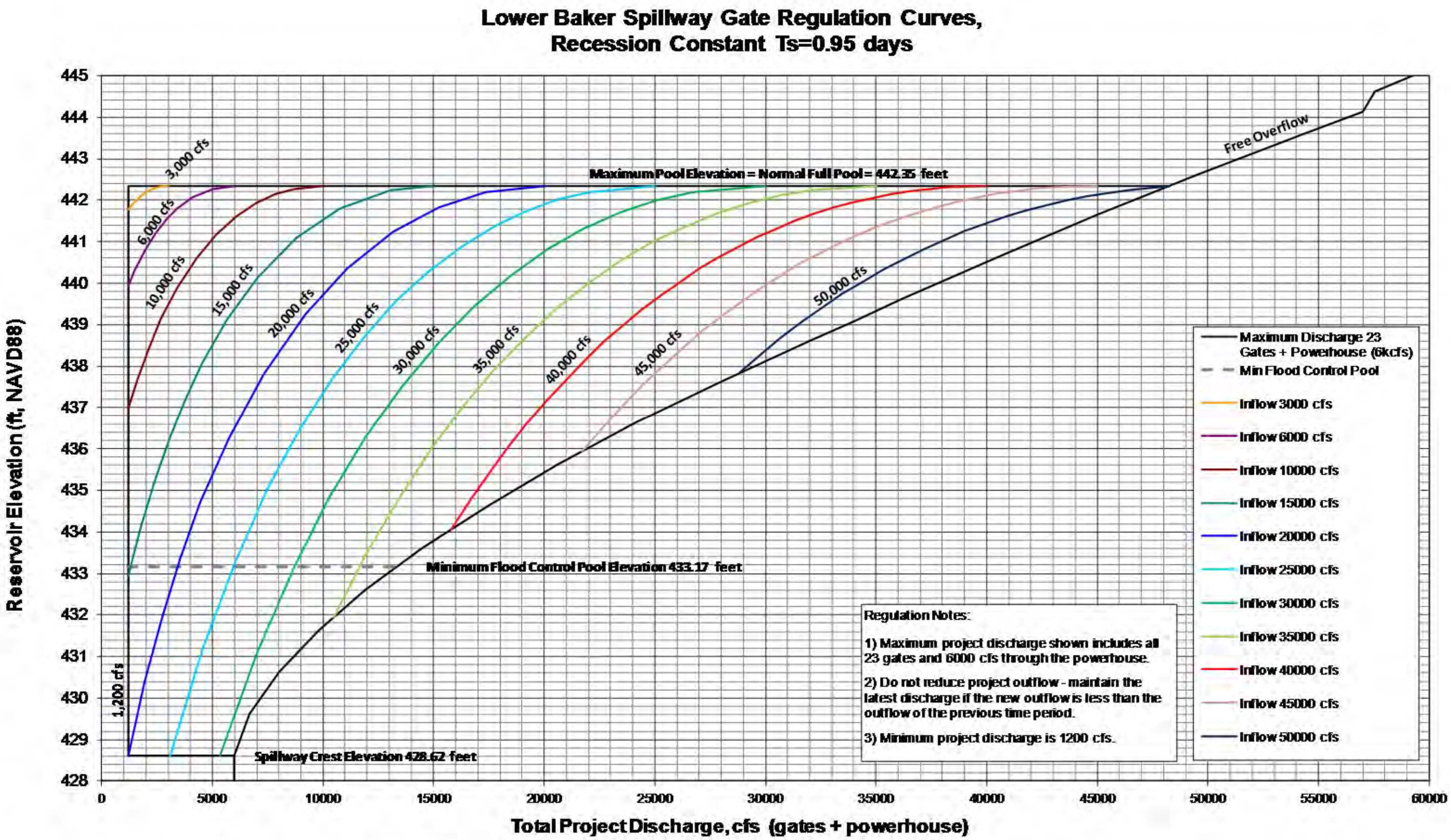


Figure 4-1: Lower Baker Dam Conceptual Spillway Gate Regulation Schedule, Ts = 0.95 days

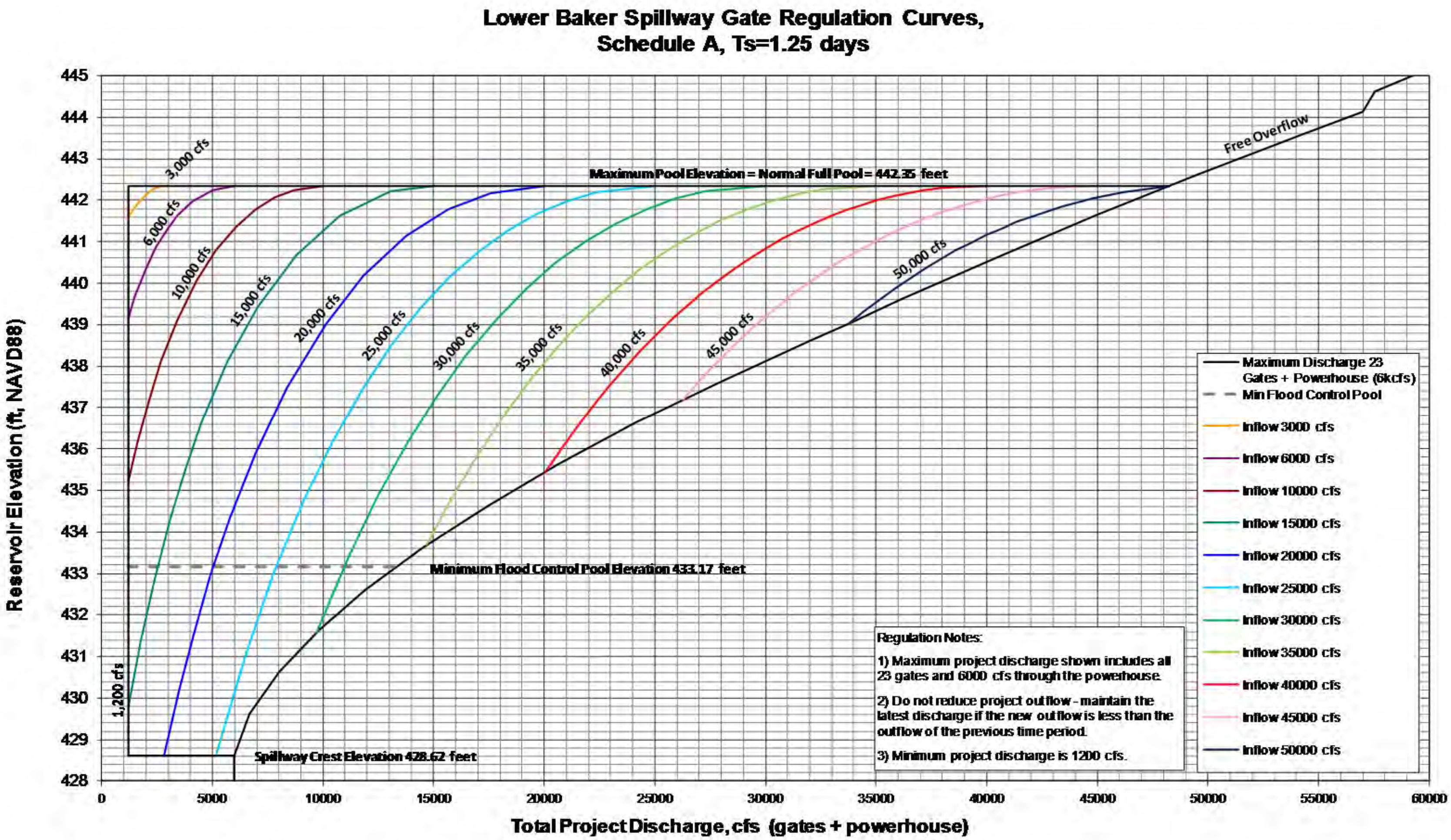


Figure 4-2: Lower Baker Dam Conceptual Spillway Gate Regulation Schedule, Ts = 1.25 days

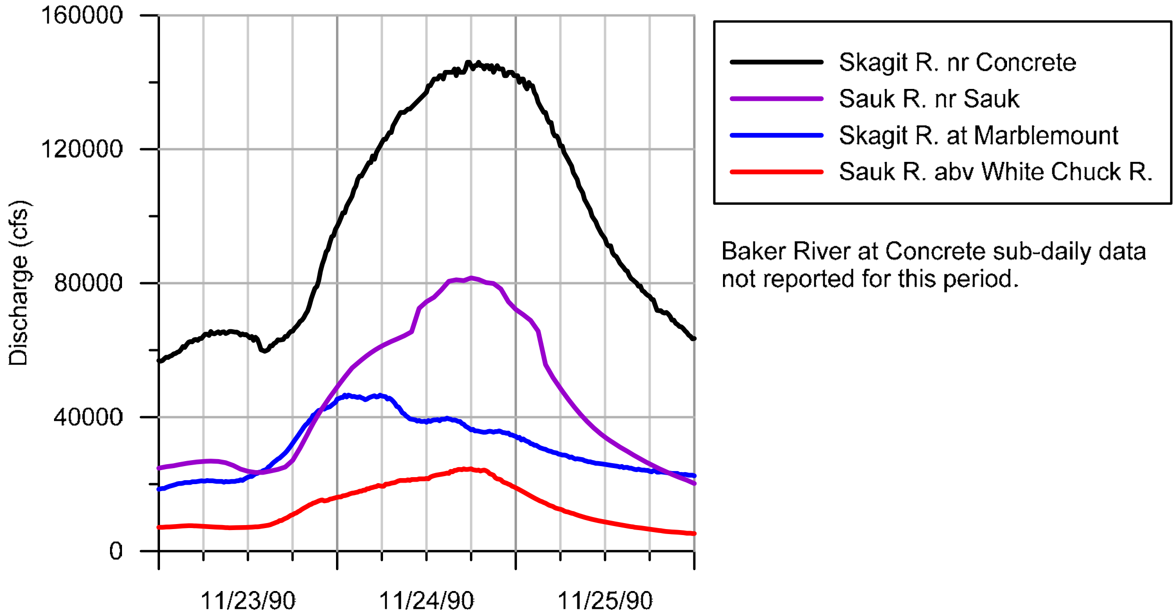


Figure 4-3: Flood Hydrographs, November 1990

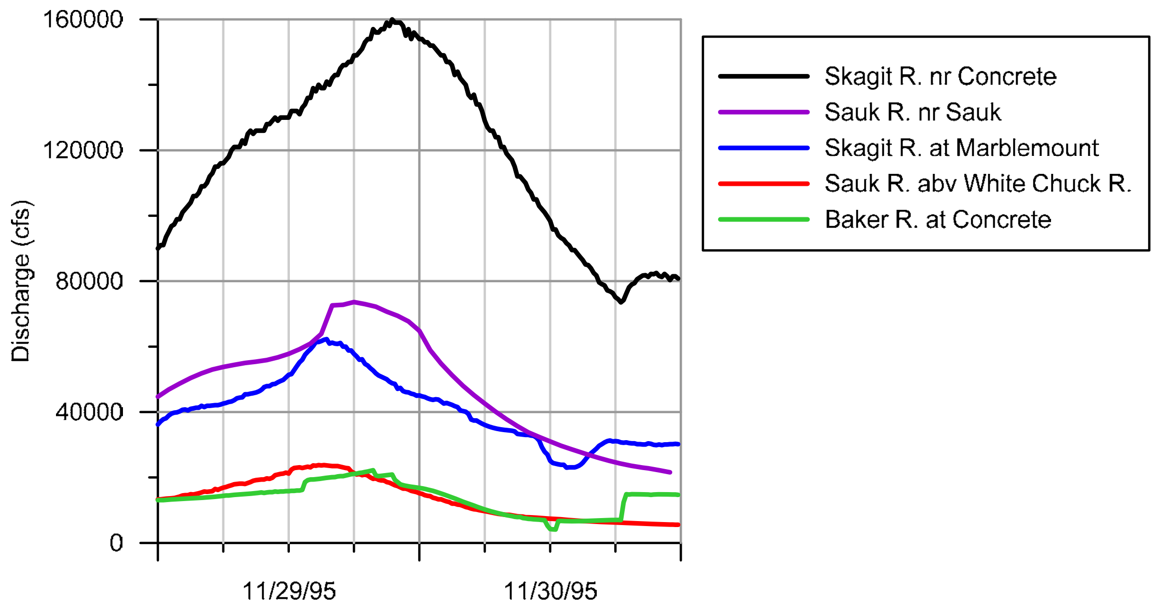


Figure 4-4: Flood Hydrographs, November 1995

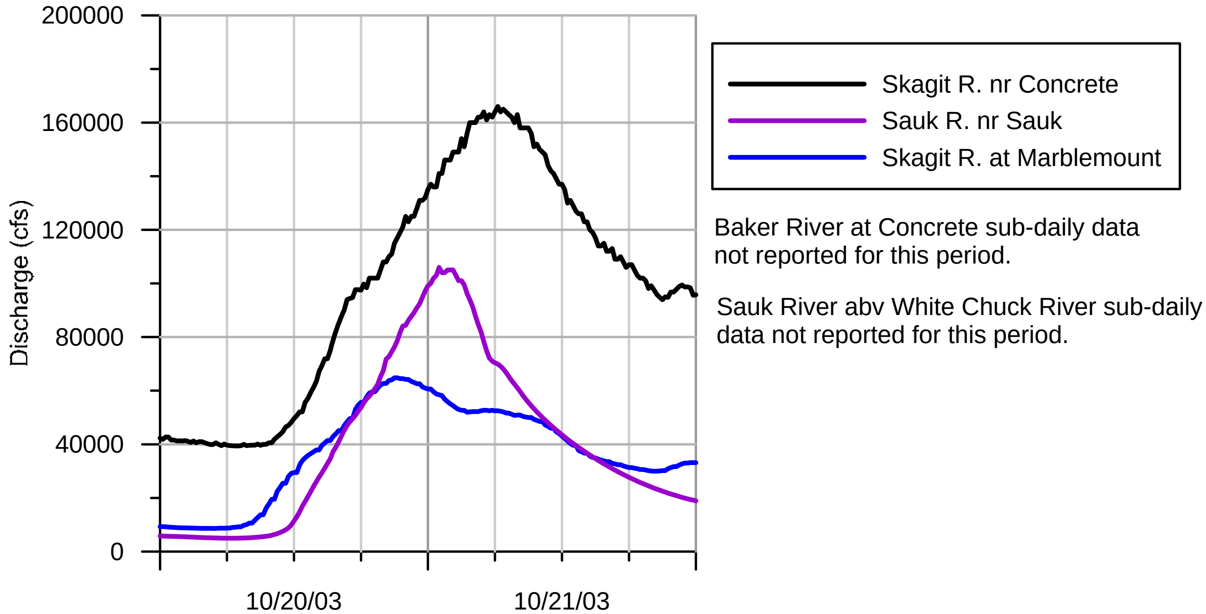


Figure 4-5: Flood Hydrographs, October 2003

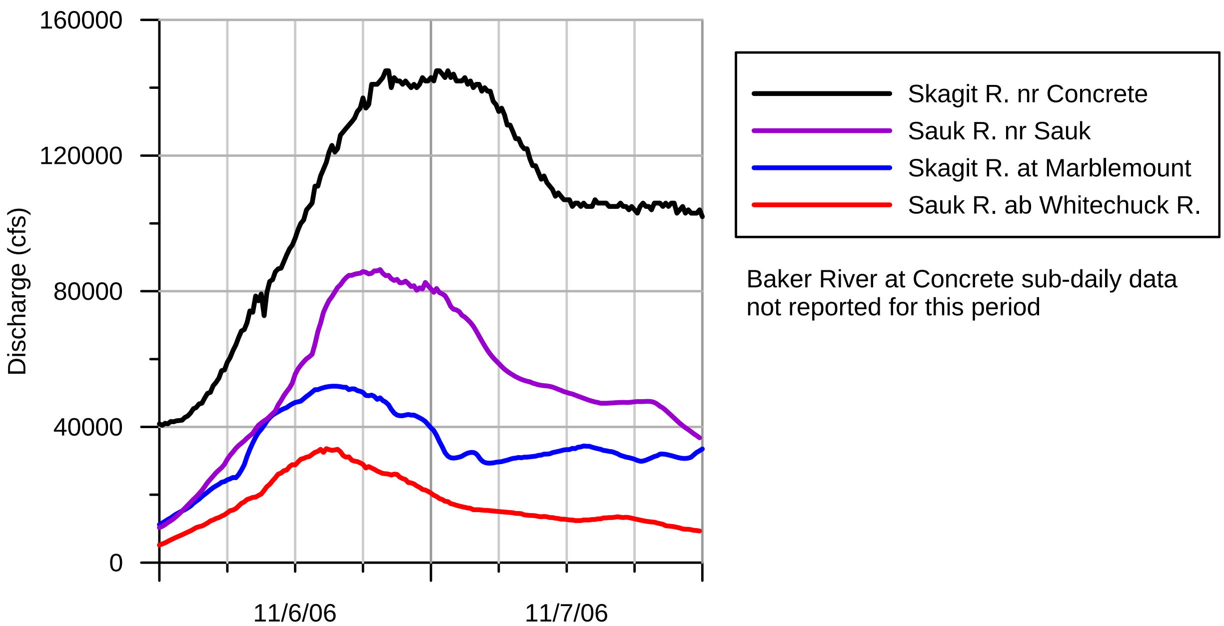


Figure 4-6: Flood Hydrographs, November 2006

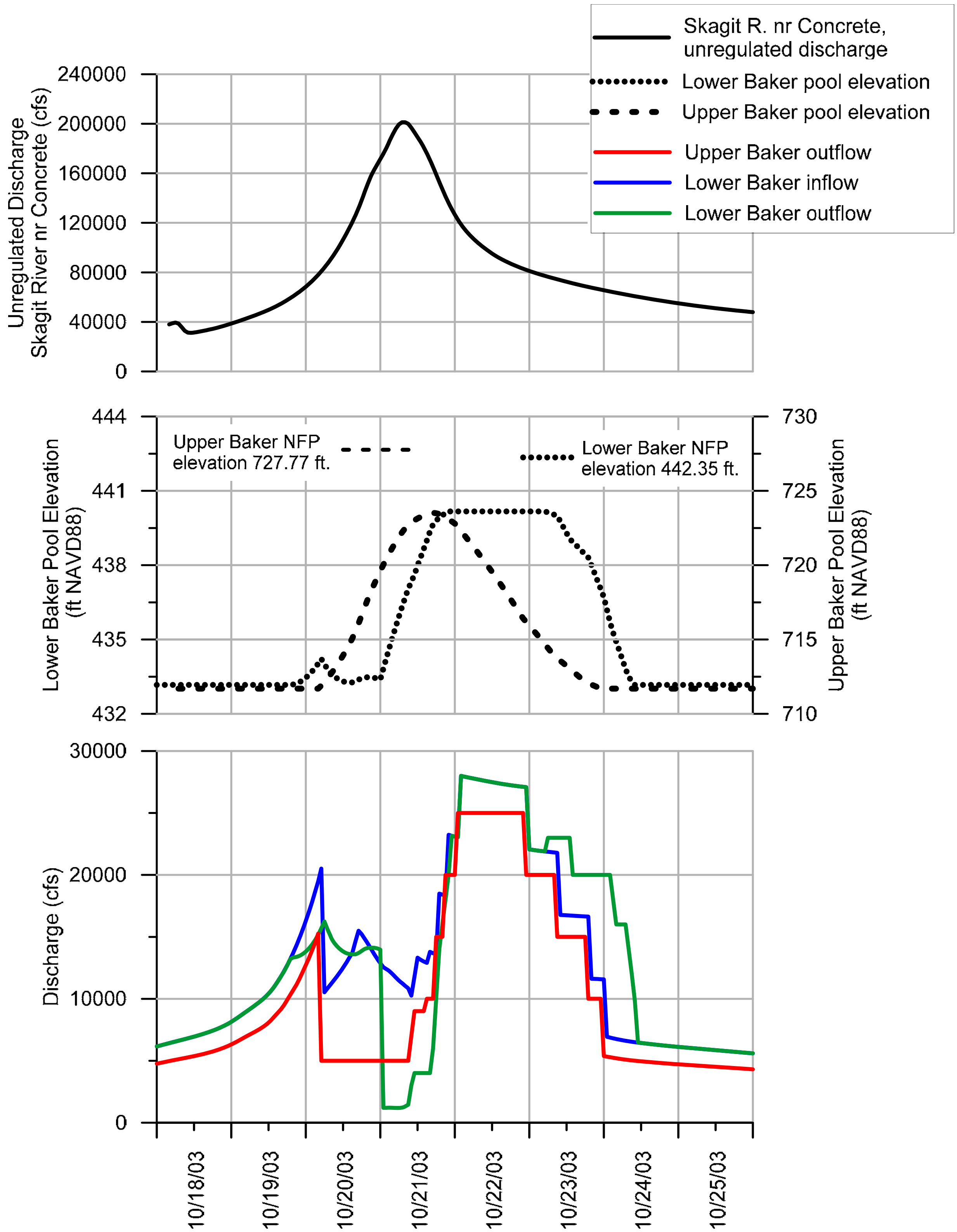


Figure 4-7: Spreadsheet model reservoir routing results for 25-year event occurring on 1 December with 20,000 acre-ft of Lower Baker reservoir flood control storage.

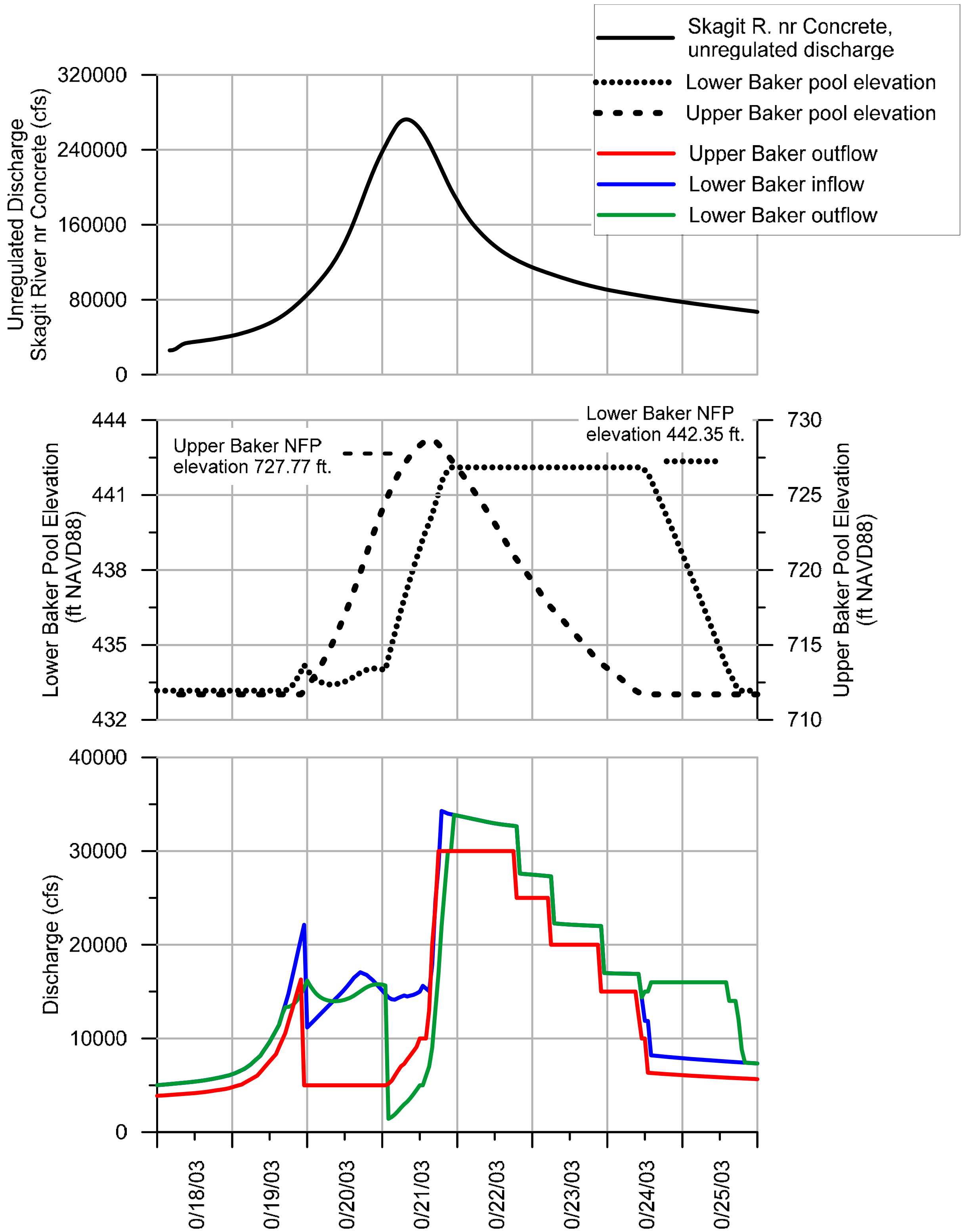


Figure 4-8: Spreadsheet model reservoir routing results for 100-year event occurring on 1 December with 20,000 acre-ft of Lower Baker reservoir flood control storage.

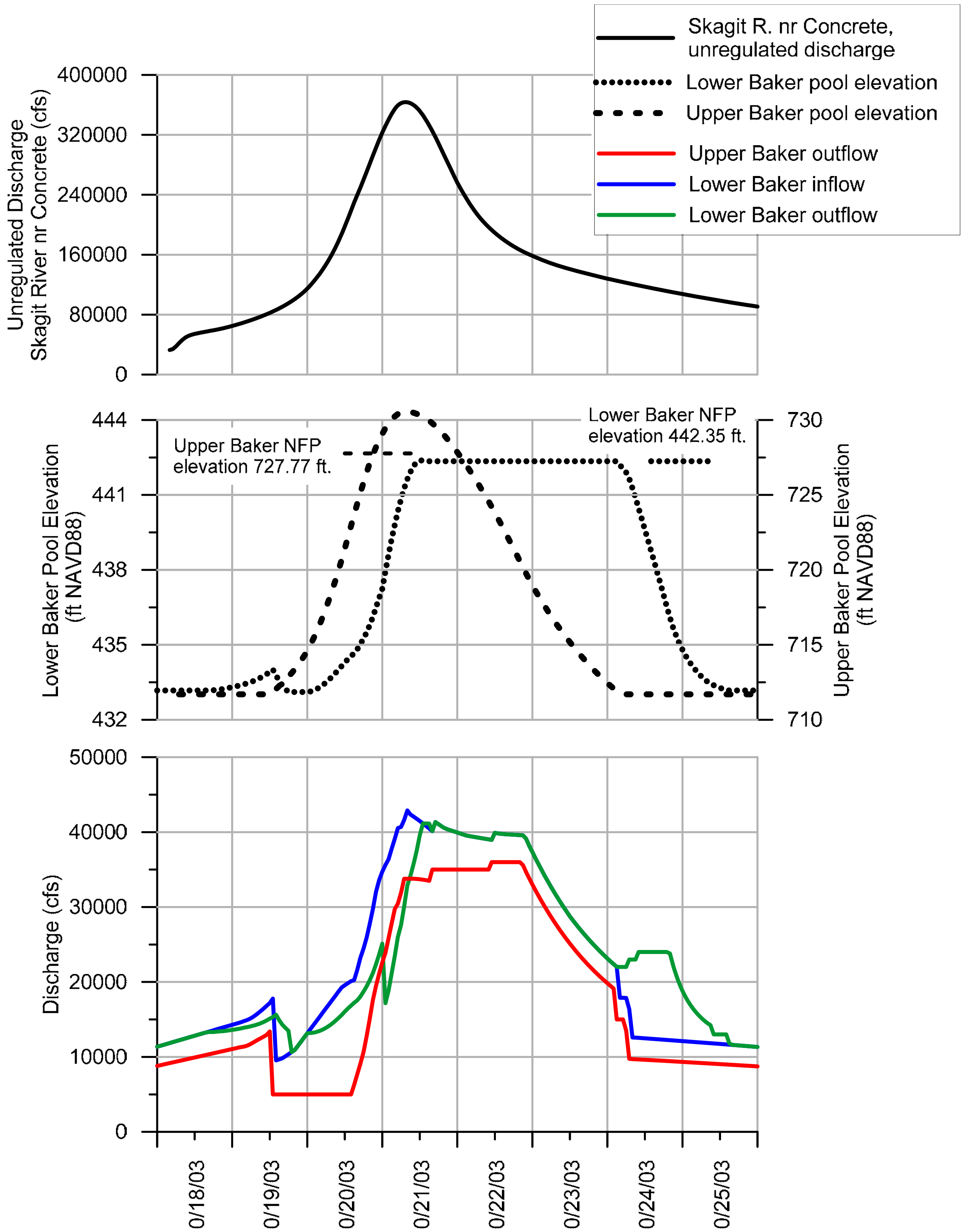


Figure 4-9: Spreadsheet model reservoir routing results for 500-year event occurring on 1 December with 20,000 acre-ft of Lower Baker reservoir flood control storage.

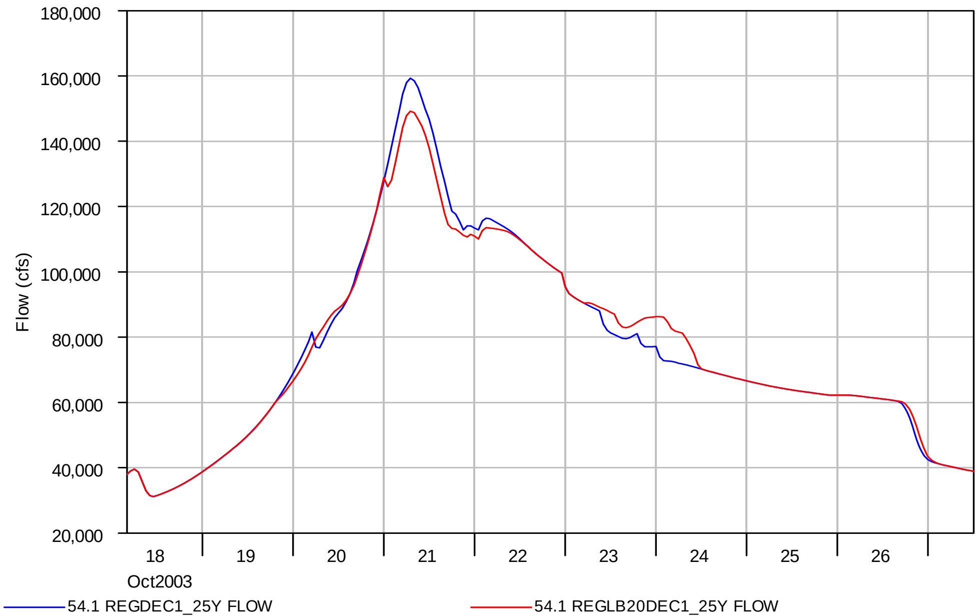


Figure 4-10: 25-year hydrographs, Skagit River near Concrete, for 1 December for existing regulation at Upper Baker and Ross Dams, with (red line) and without (blue line) flood control storage at Lower Baker.

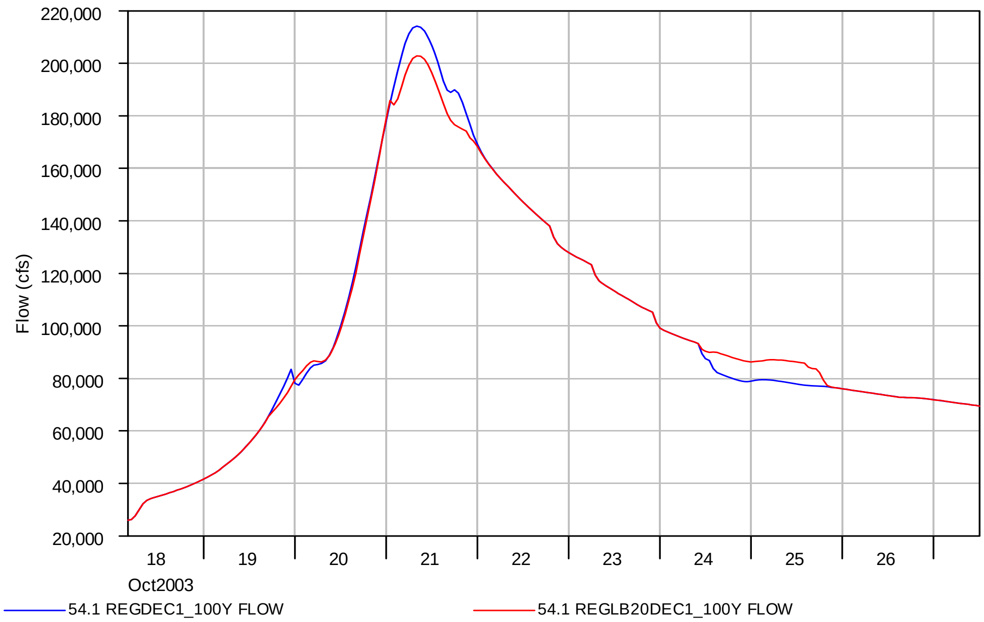


Figure 4-11: 100-year hydrographs, Skagit River near Concrete, for 1 December for existing regulation at Upper Baker and Ross Dams, with (red line) and without (blue line) flood control storage at Lower Baker.

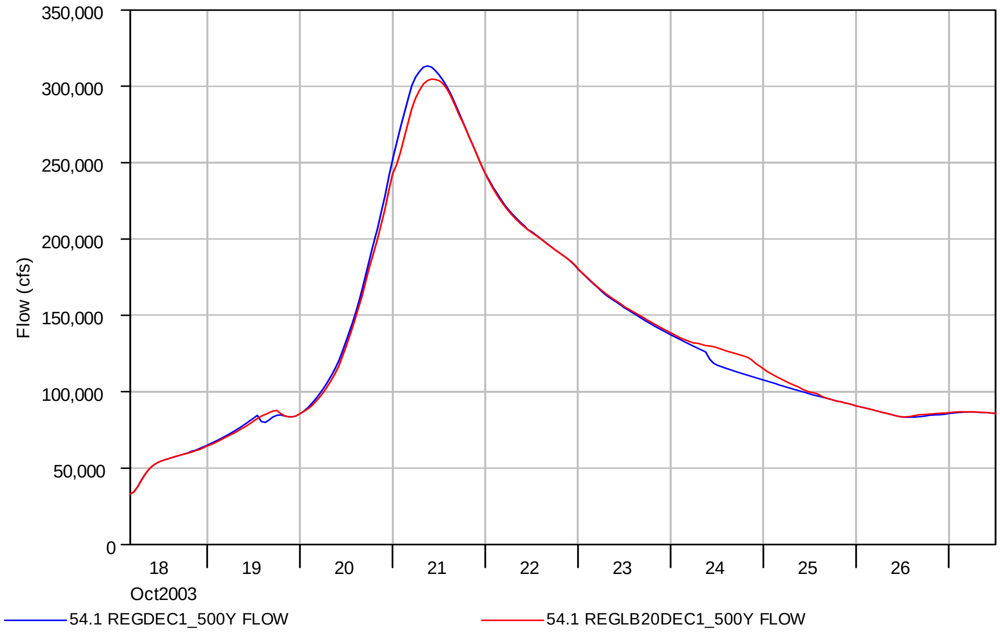


Figure 4-12: 500-year hydrographs, Skagit River near Concrete, for 1 December for existing regulation at Upper Baker and Ross Dams, with (red line) and without (blue line) flood control storage at Lower Baker.

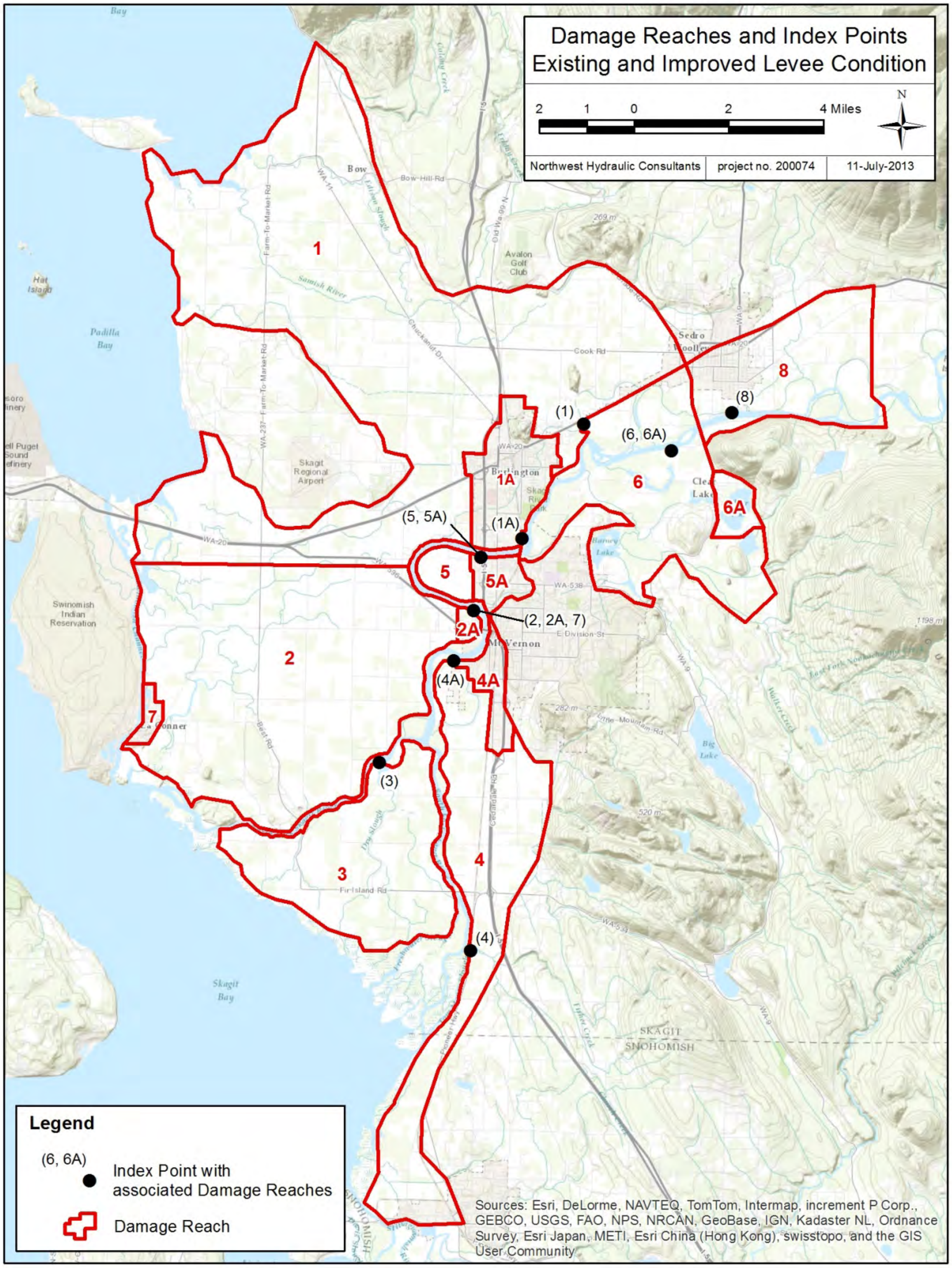


Figure 5-1: Damage Reaches and Index Points

Figure 5-2: Existing Condition 100-Year With-Breach Stage Hydrograph, North Fork XS 829.

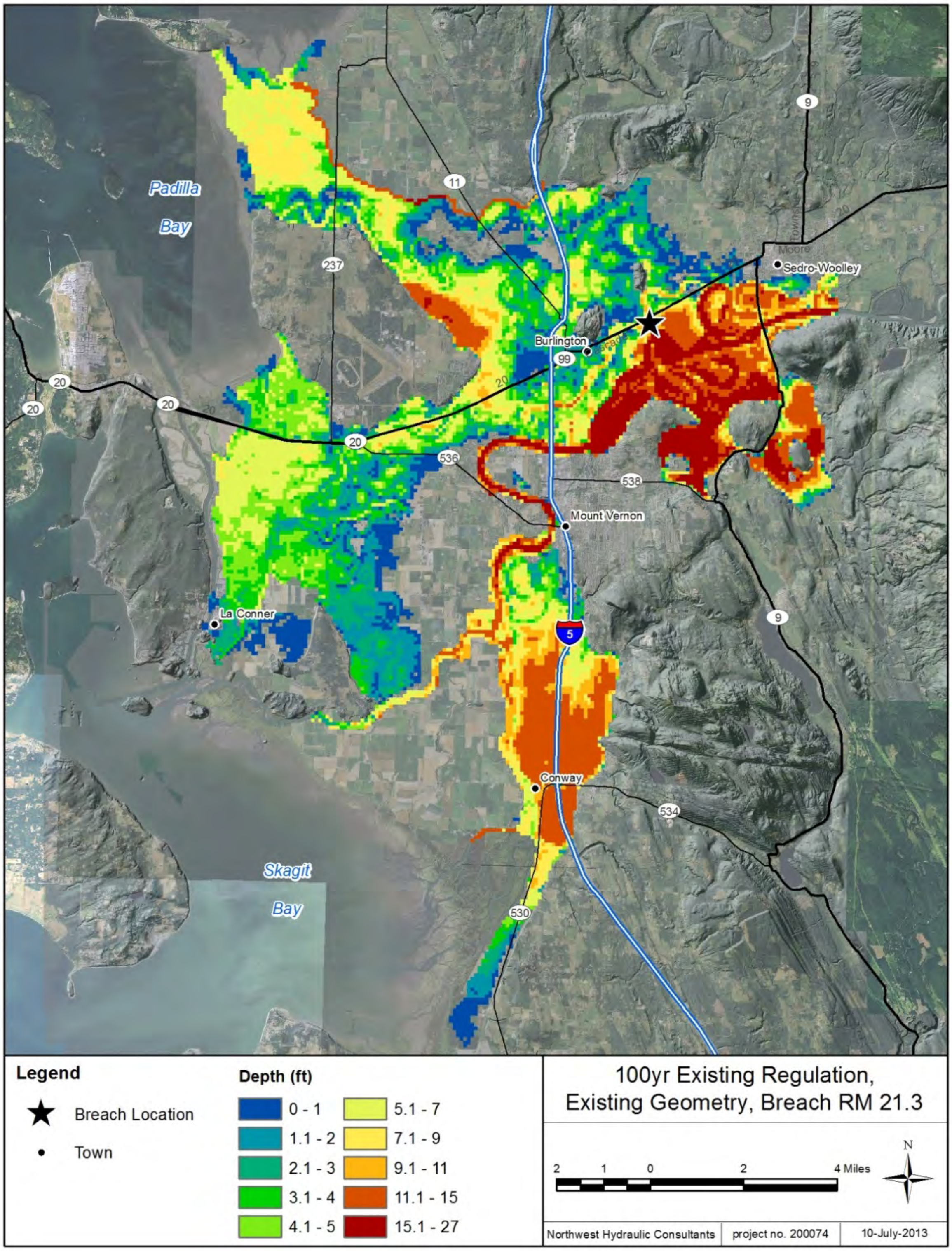


Figure 5-3: Maximum flood depths for 100-year event under existing conditions with levee breach at RM 21.3

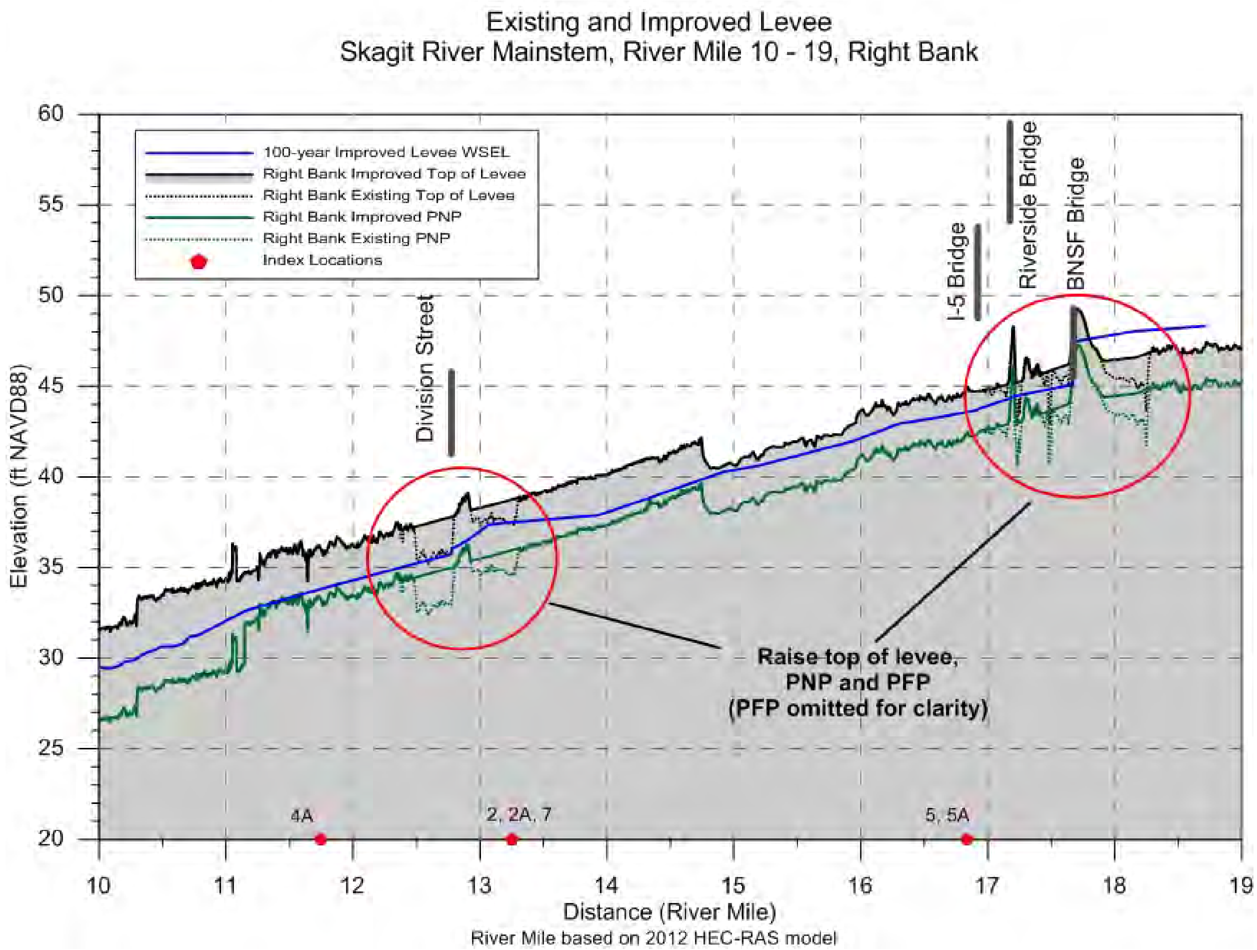


Figure 5-4: Levee improvements, right bank mainstem Skagit River

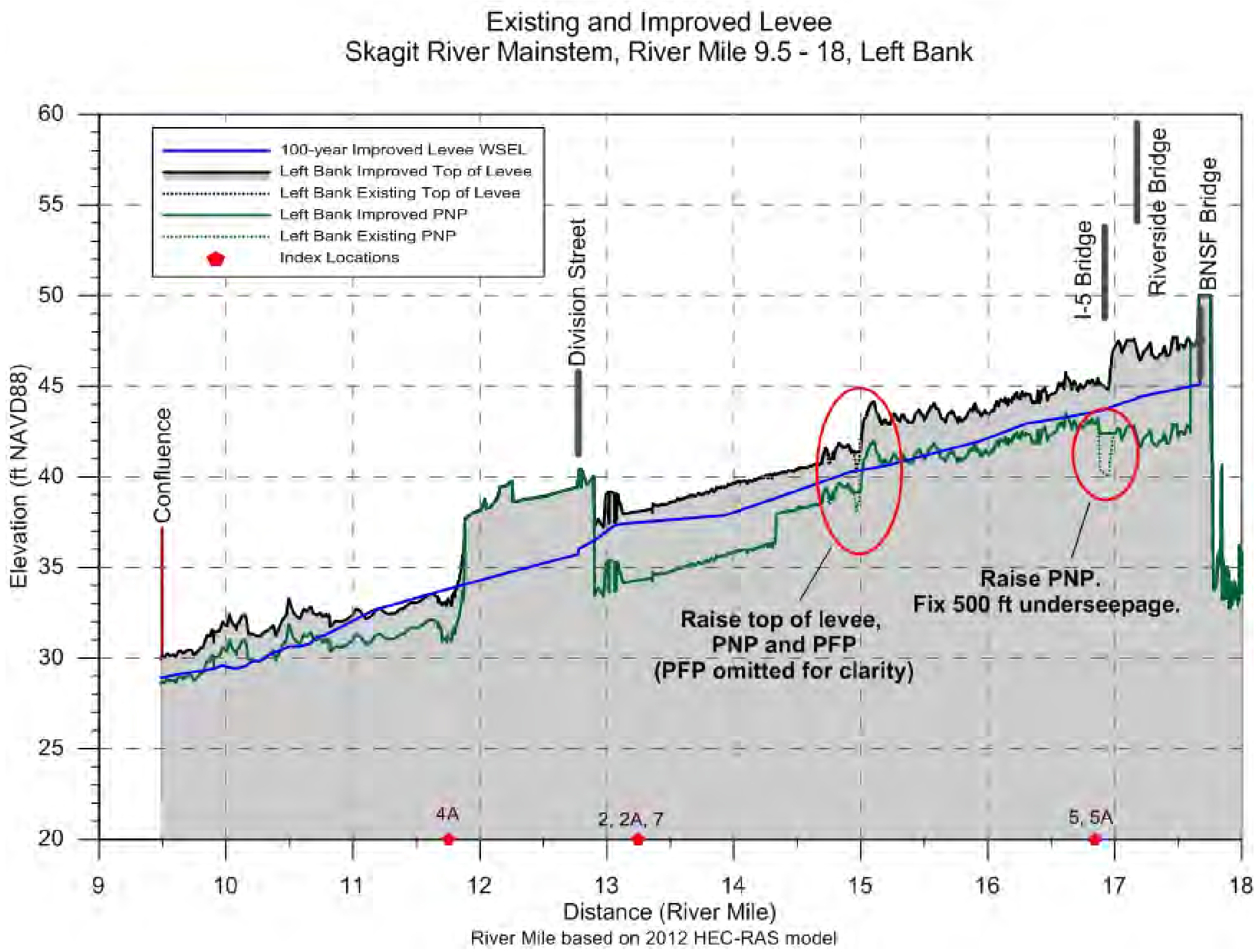


Figure 5-5: Levee improvements, left bank mainstem Skagit River

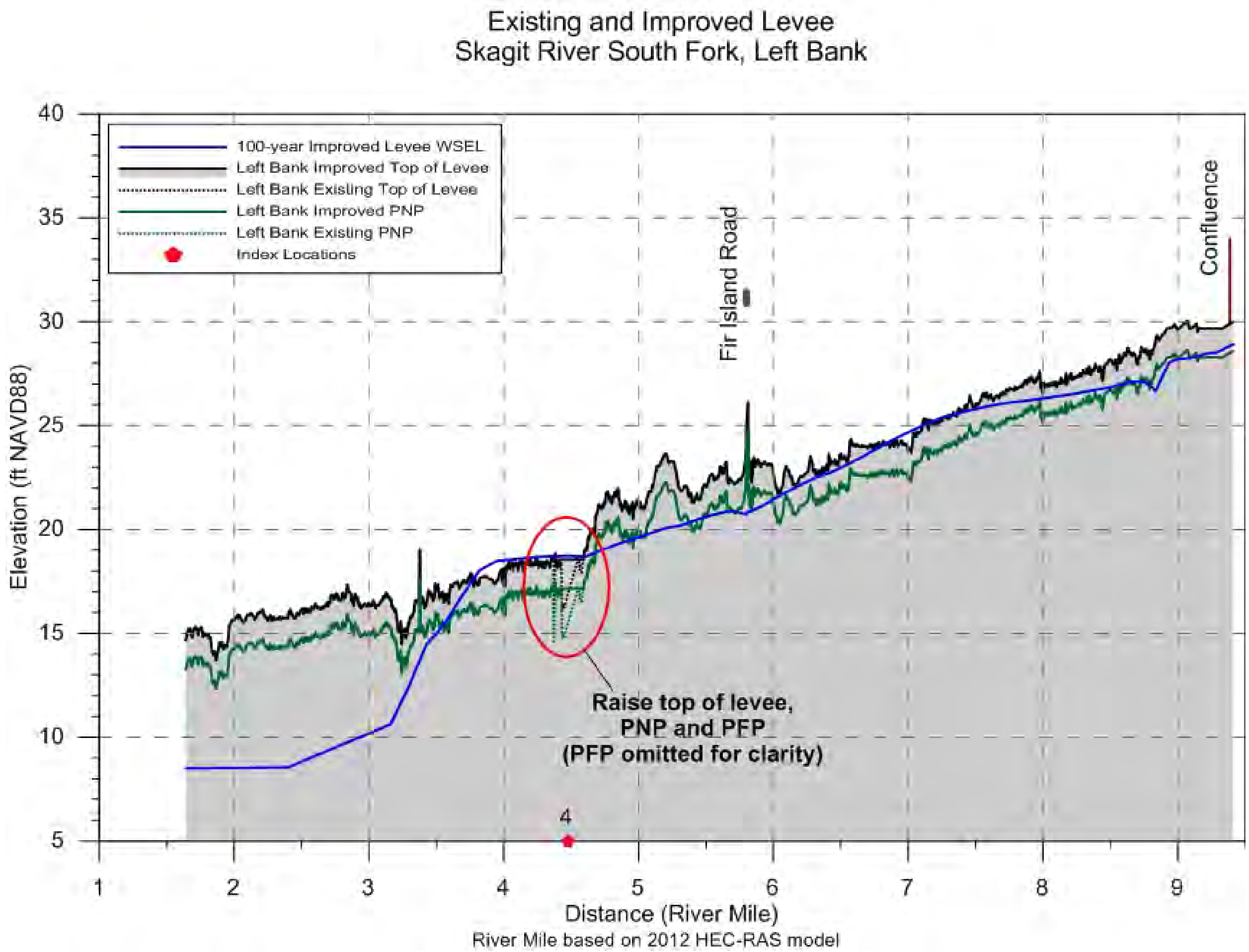


Figure 5-6: Levee improvements, left bank South Fork Skagit River

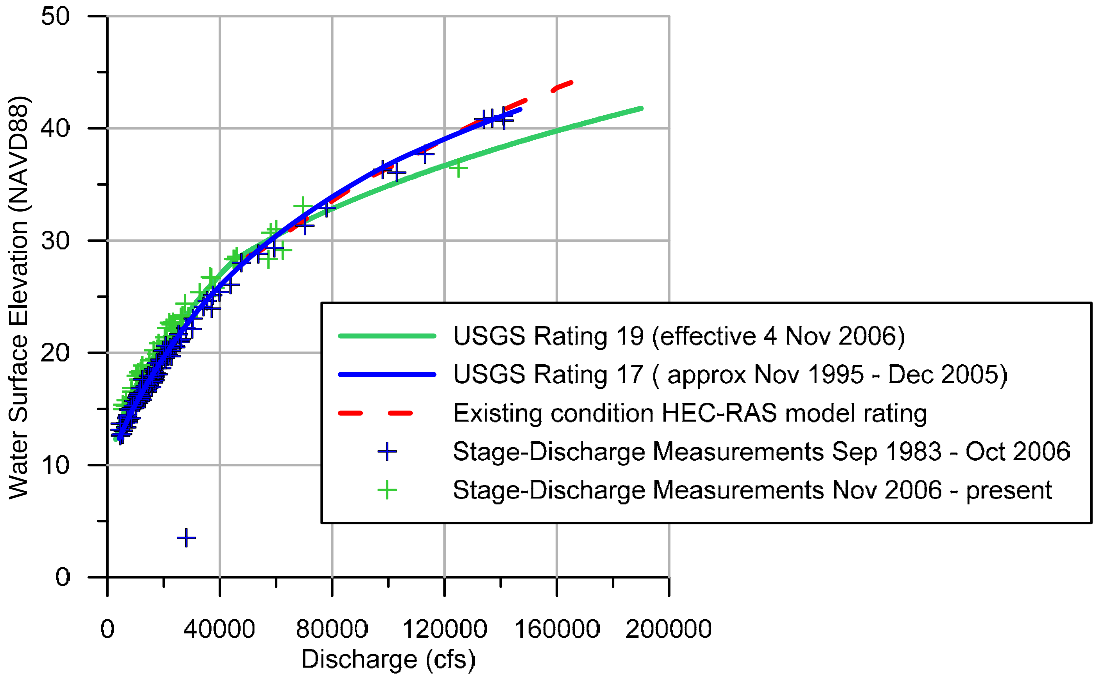


Figure 5-7: Stage-discharge measurements and ratings, USGS gage 12200500, Skagit River near Mount Vernon

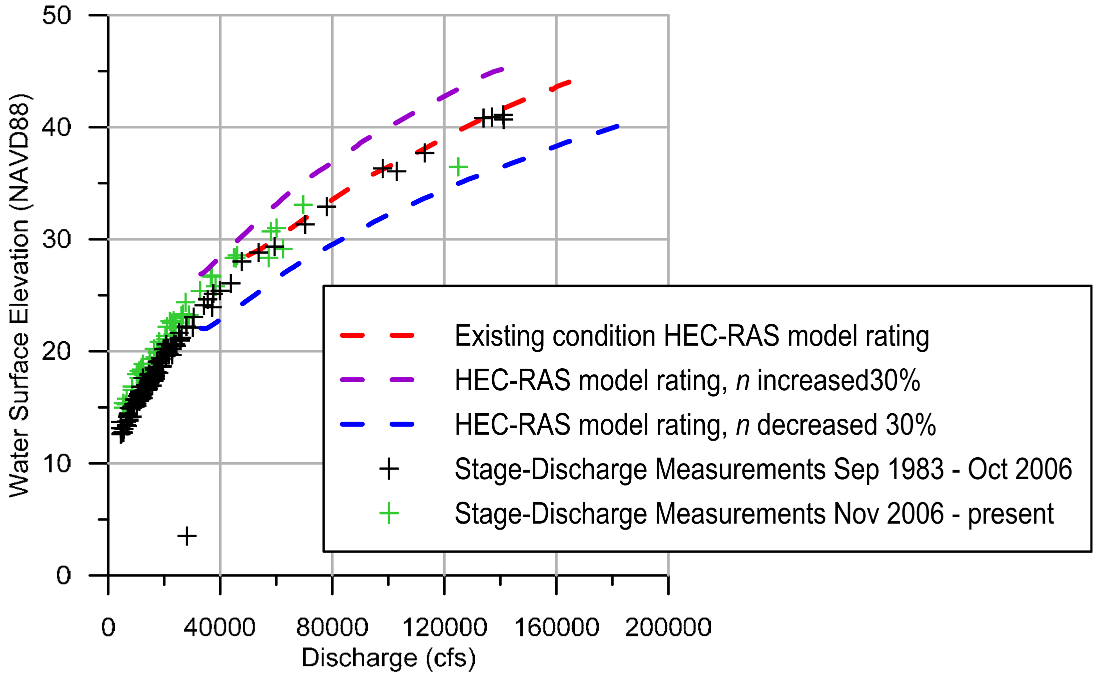


Figure 5-8: Effect of uncertainty in Manning's n on stage-discharge rating, USGS gage 12200500, Skagit River near Mount Vernon

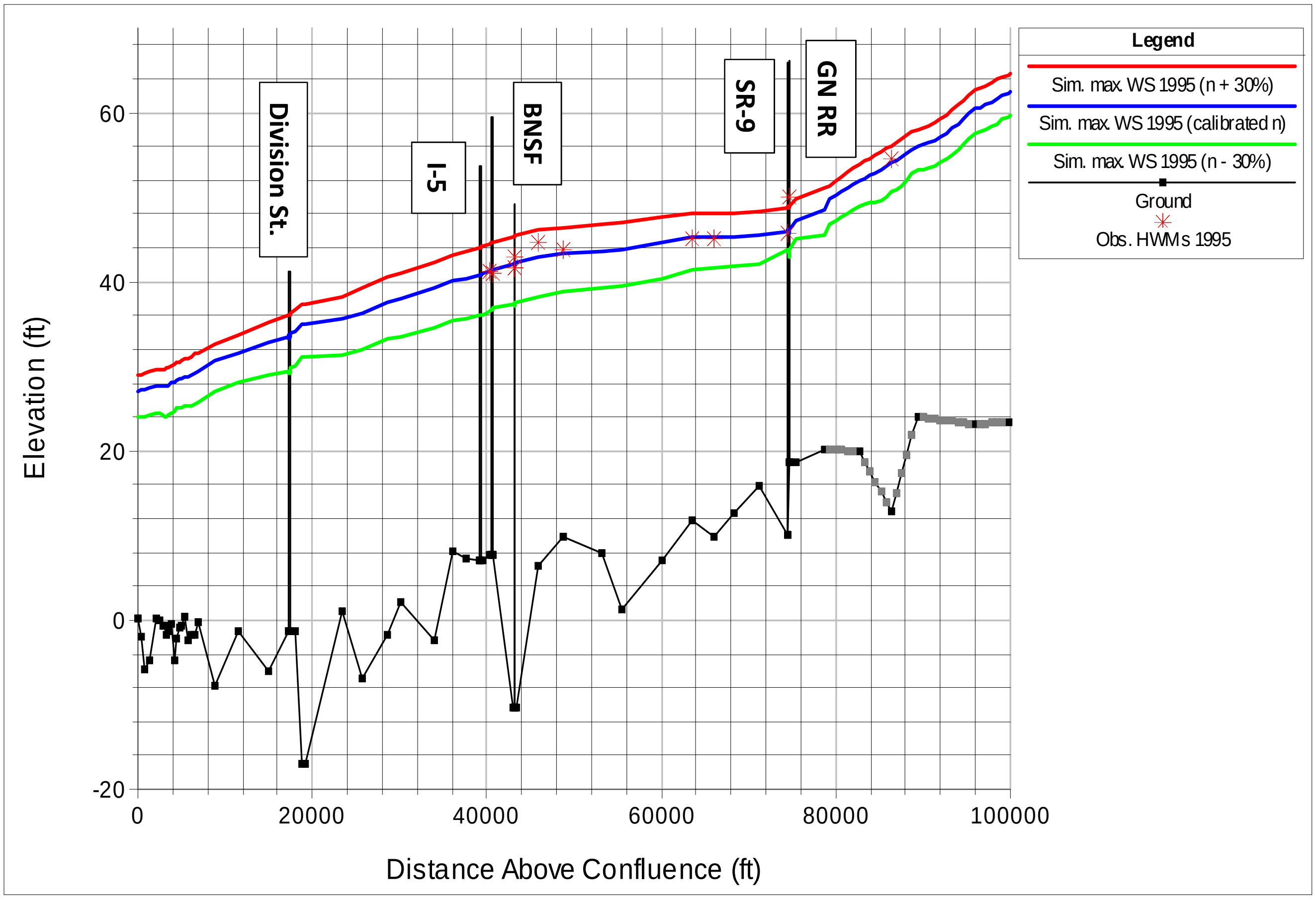


Figure 5-9: HEC-RAS water surface profiles for November 1995 event with n varied ± 30%

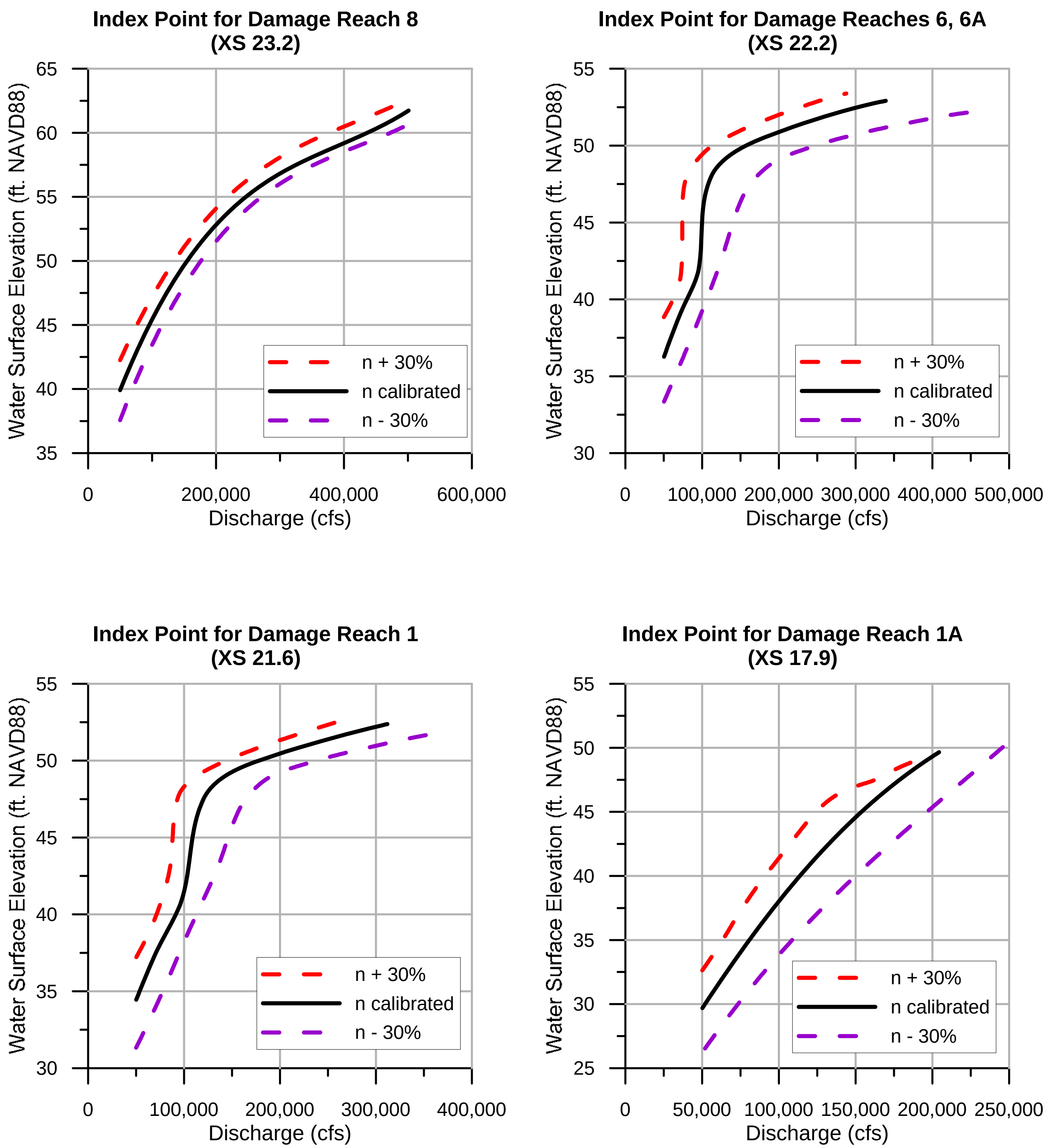


Figure 5-10: Stage-discharge ratings with uncertainty in roughness, existing conditions

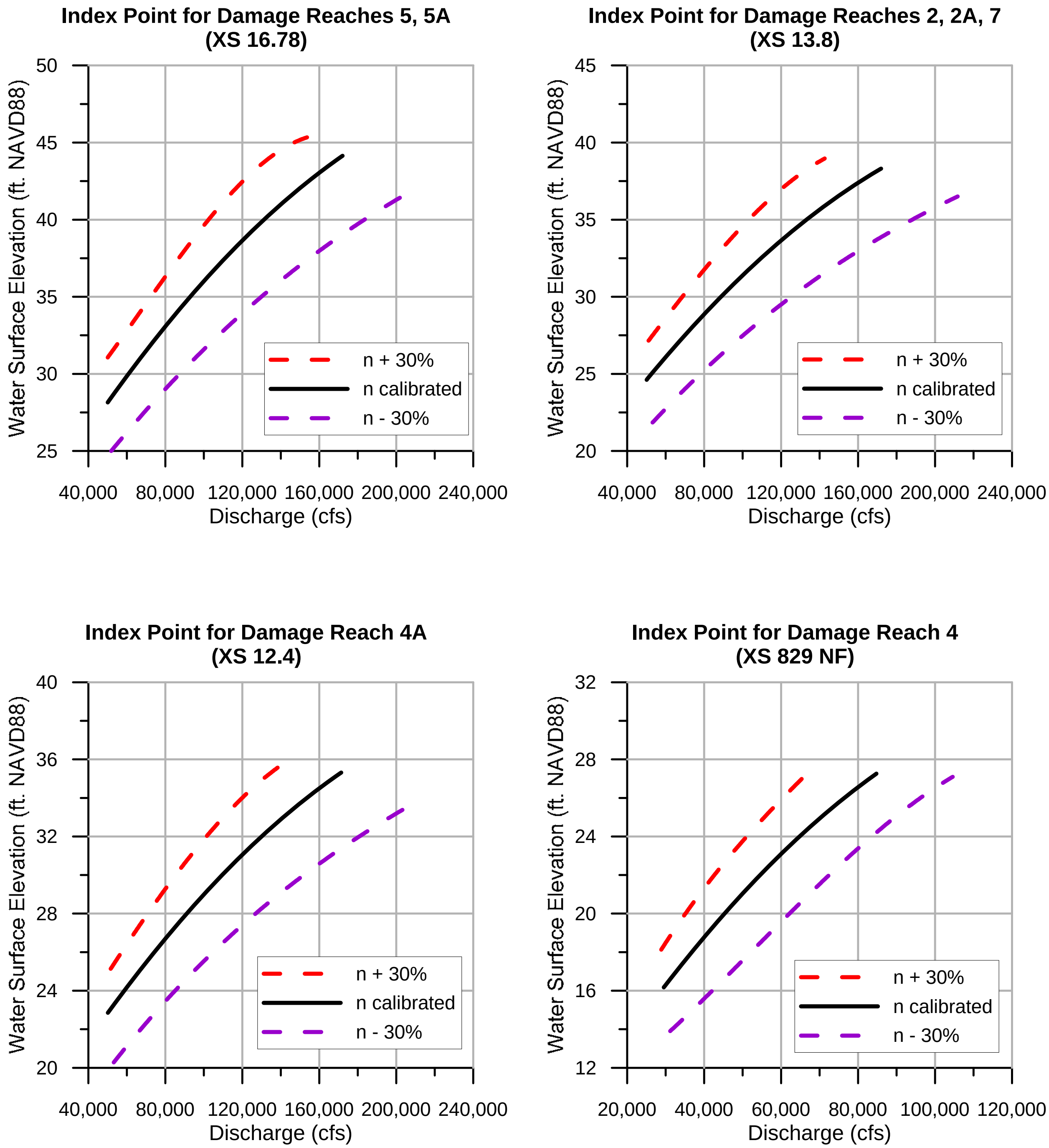


Figure 5-10 (cont.): Stage-discharge ratings with uncertainty in roughness, existing conditions

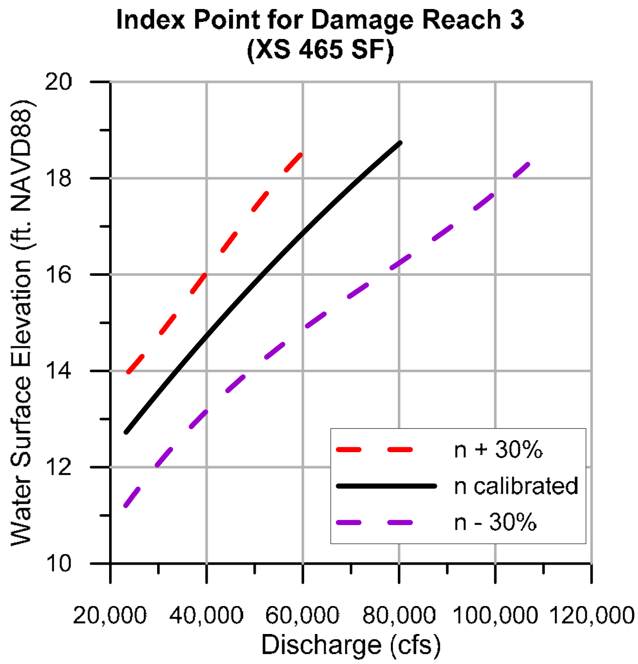


Figure 5-10 (cont.): Stage-discharge ratings with uncertainty in roughness, existing conditions

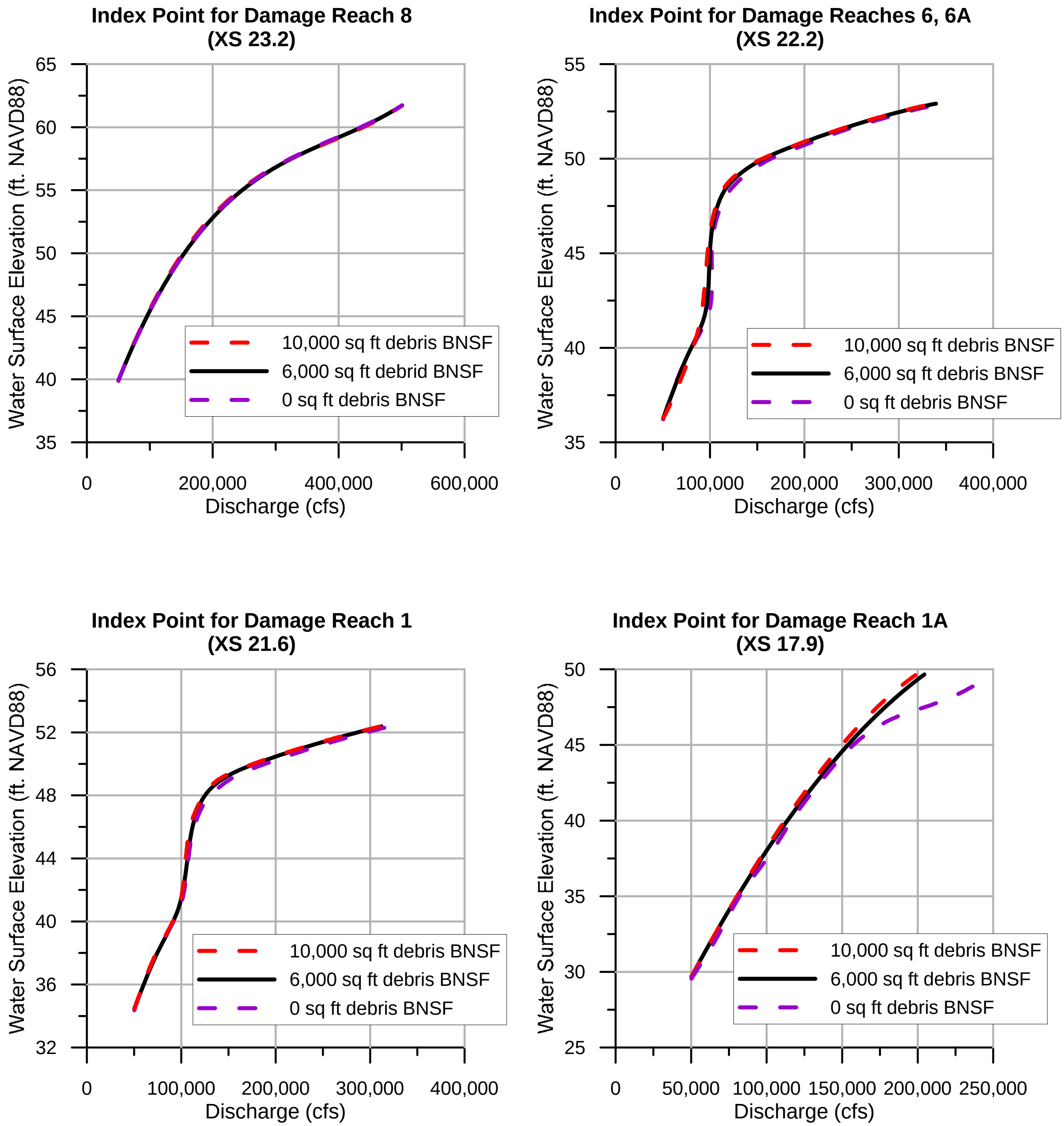


Figure 5-11 Stage-discharge ratings with uncertainty in bridge debris, existing conditions

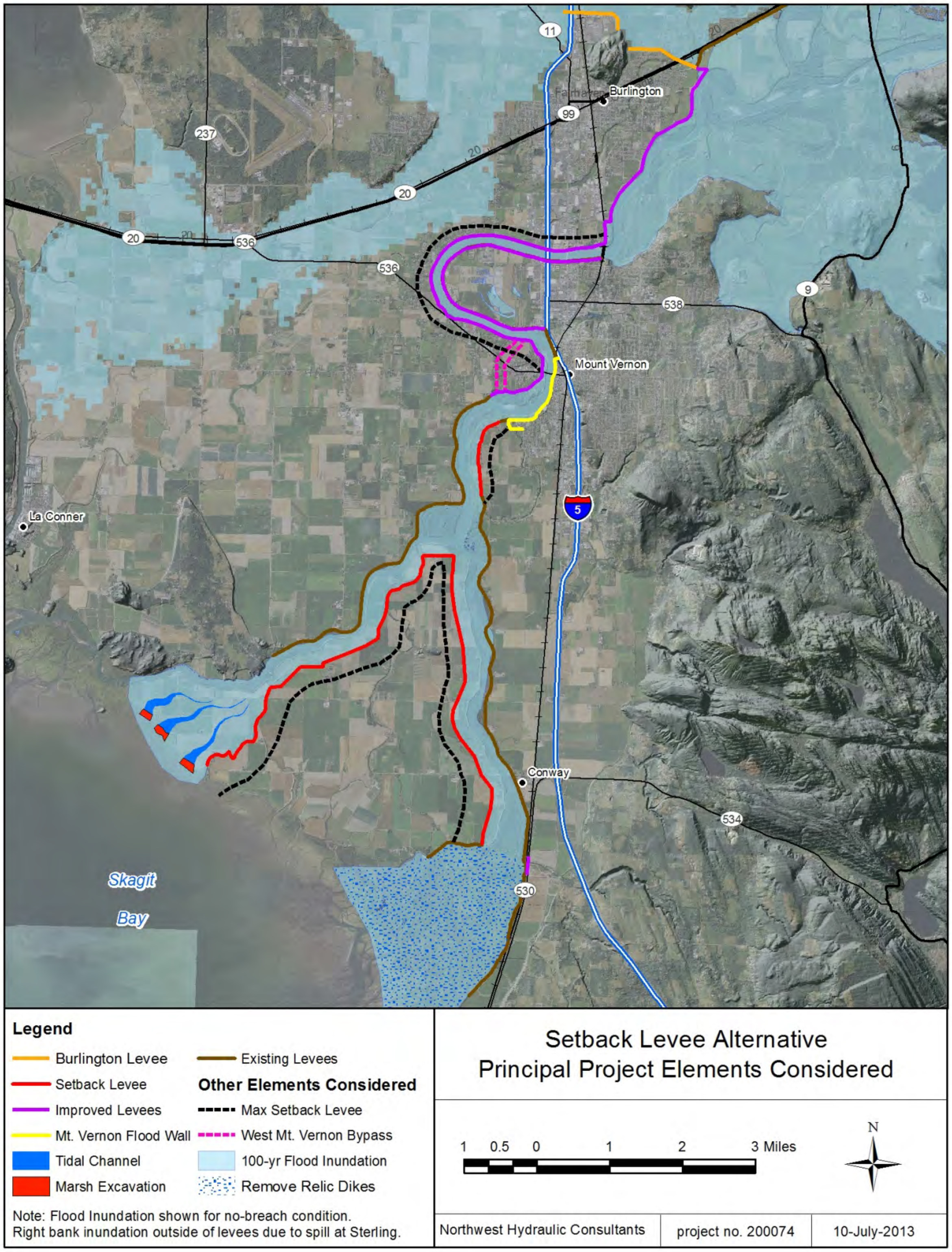


Figure 6-1: Setback Levee Alternative Project Elements

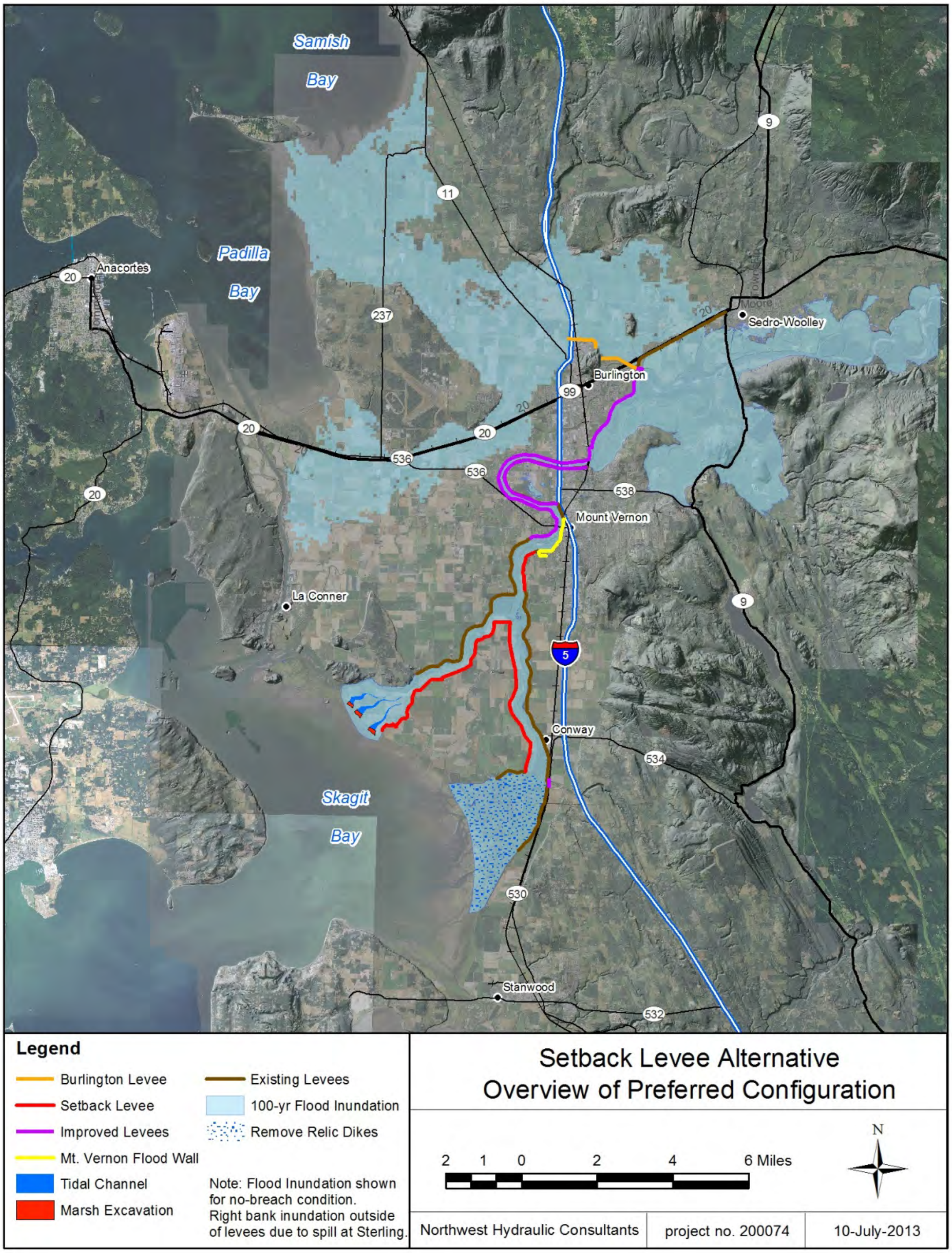
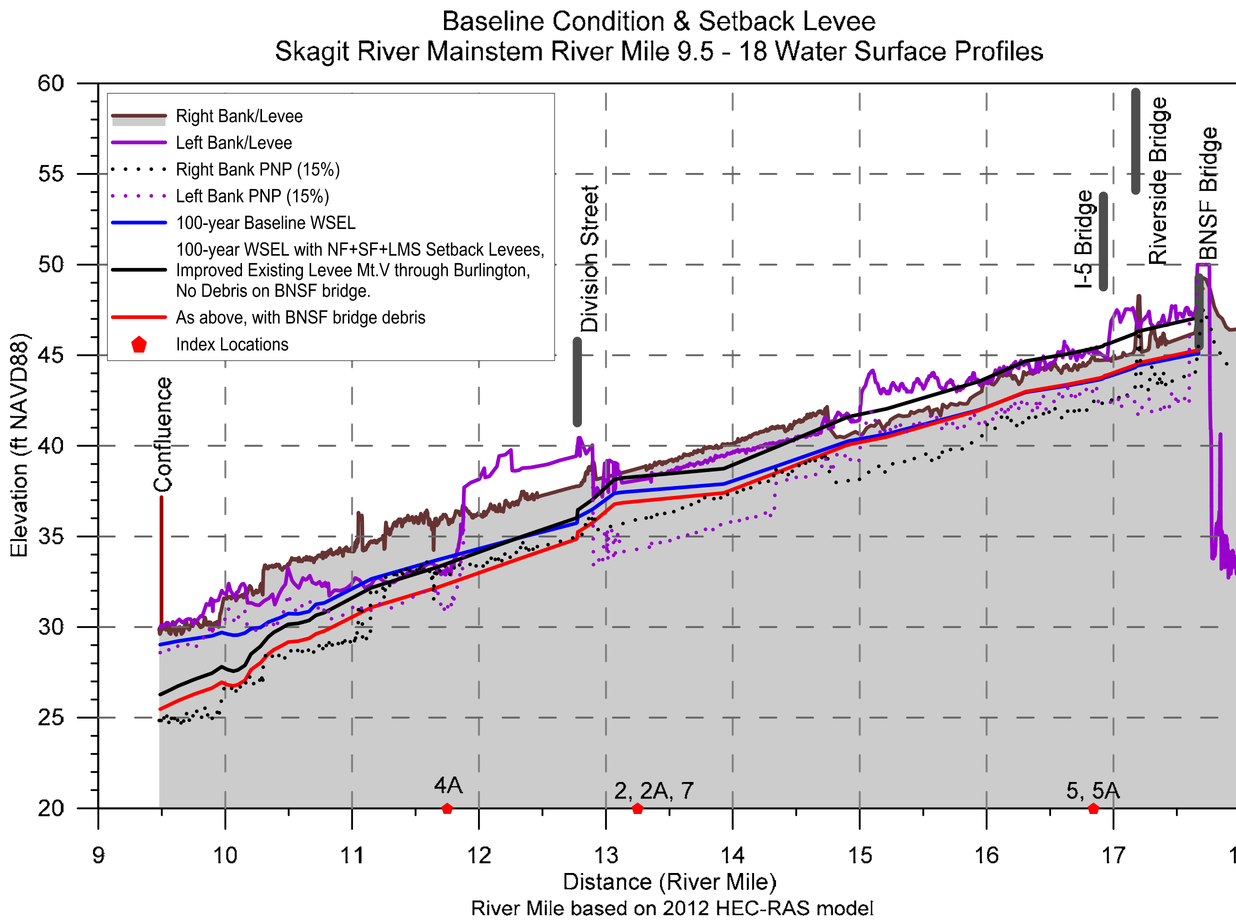
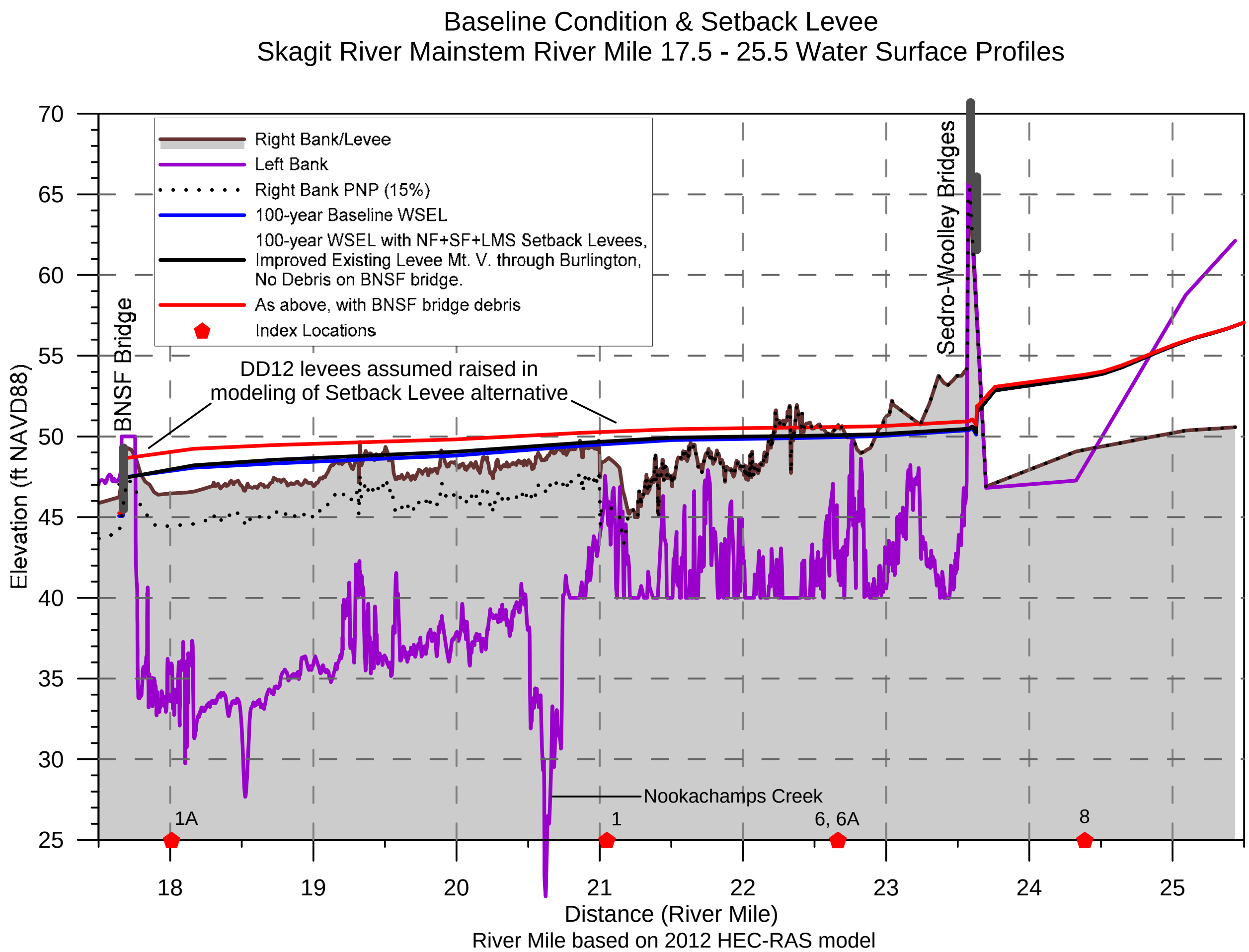


Figure 6-2: Setback Levee Alternative Preferred Configuration



Note: Levee elevations and PNP profiles represent the improved levee "baseline" condition.

Figure 6-3: Setback Levee Alternative: Water Surface and PNP Profiles, RM 9.5 to RM 18



Note: Levee elevations and PNP profiles represent the improved levee "baseline" condition.

Figure 6-4: Setback Levee Alternative: Water Surface and PNP Profiles, RM 17.5 to RM 25.5

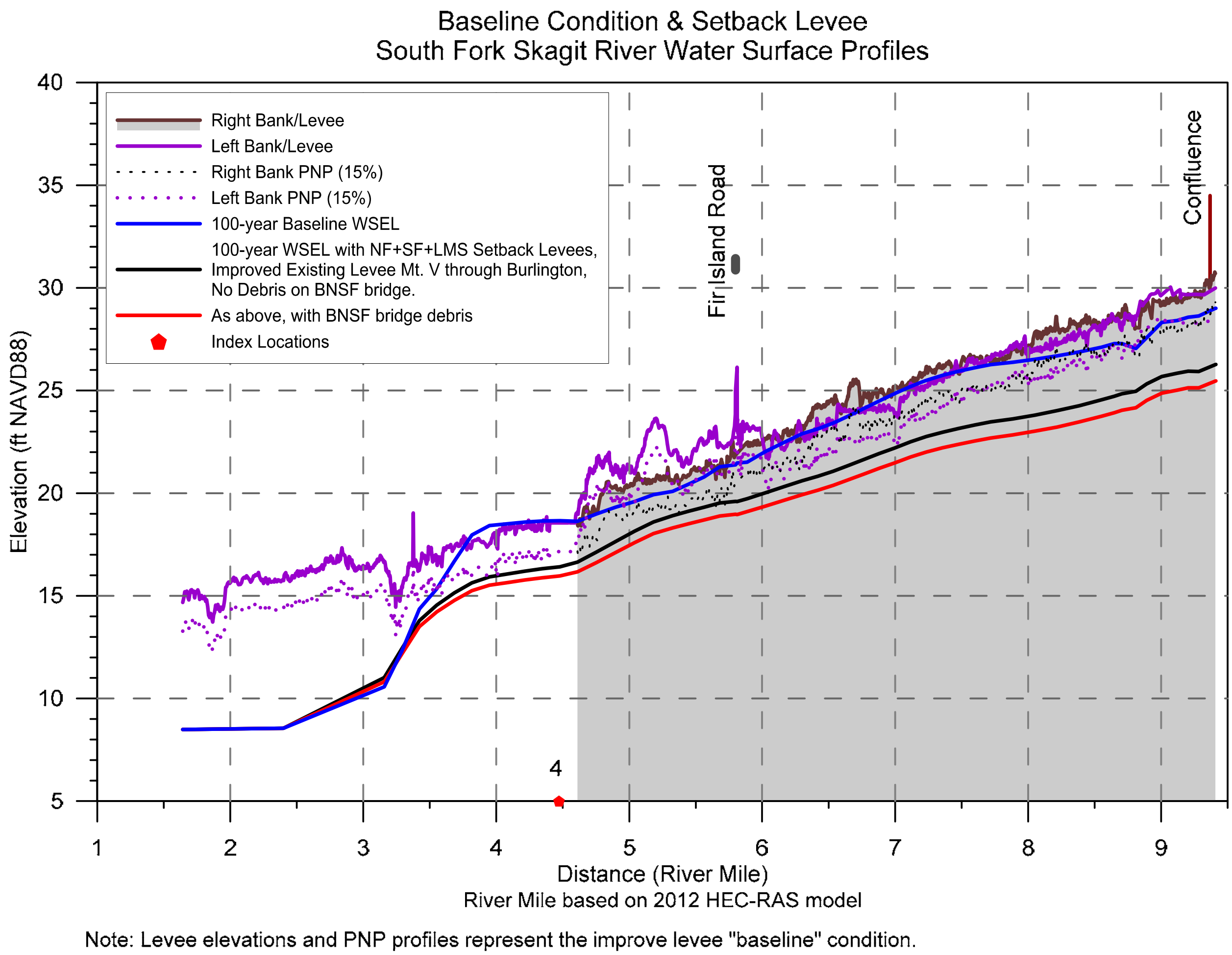


Figure 6-5: Setback Levee Alternative: Water Surface and PNP Profiles, South Fork

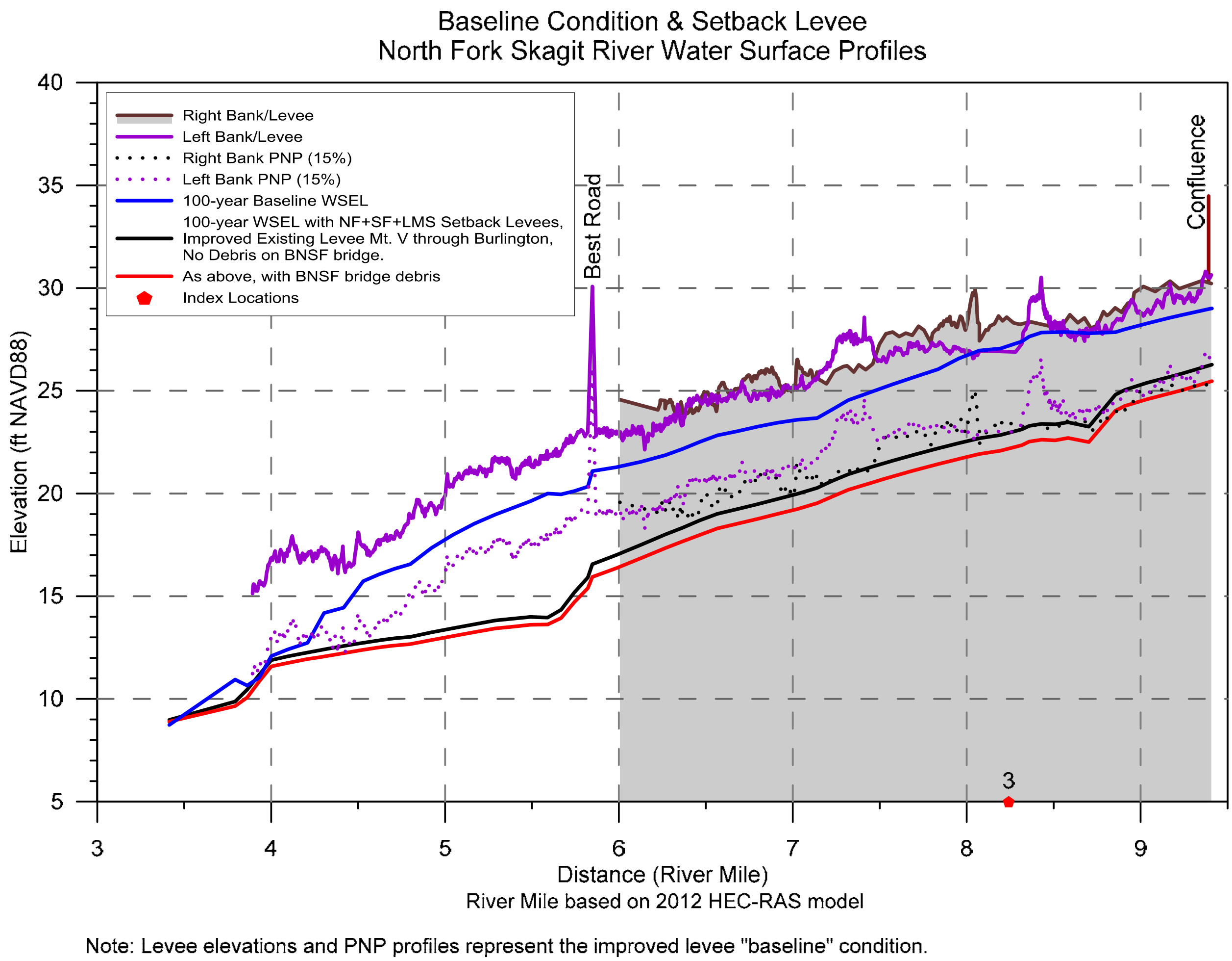


Figure 6-6: Setback Levee Alternative: Water Surface and PNP Profiles, North Fork

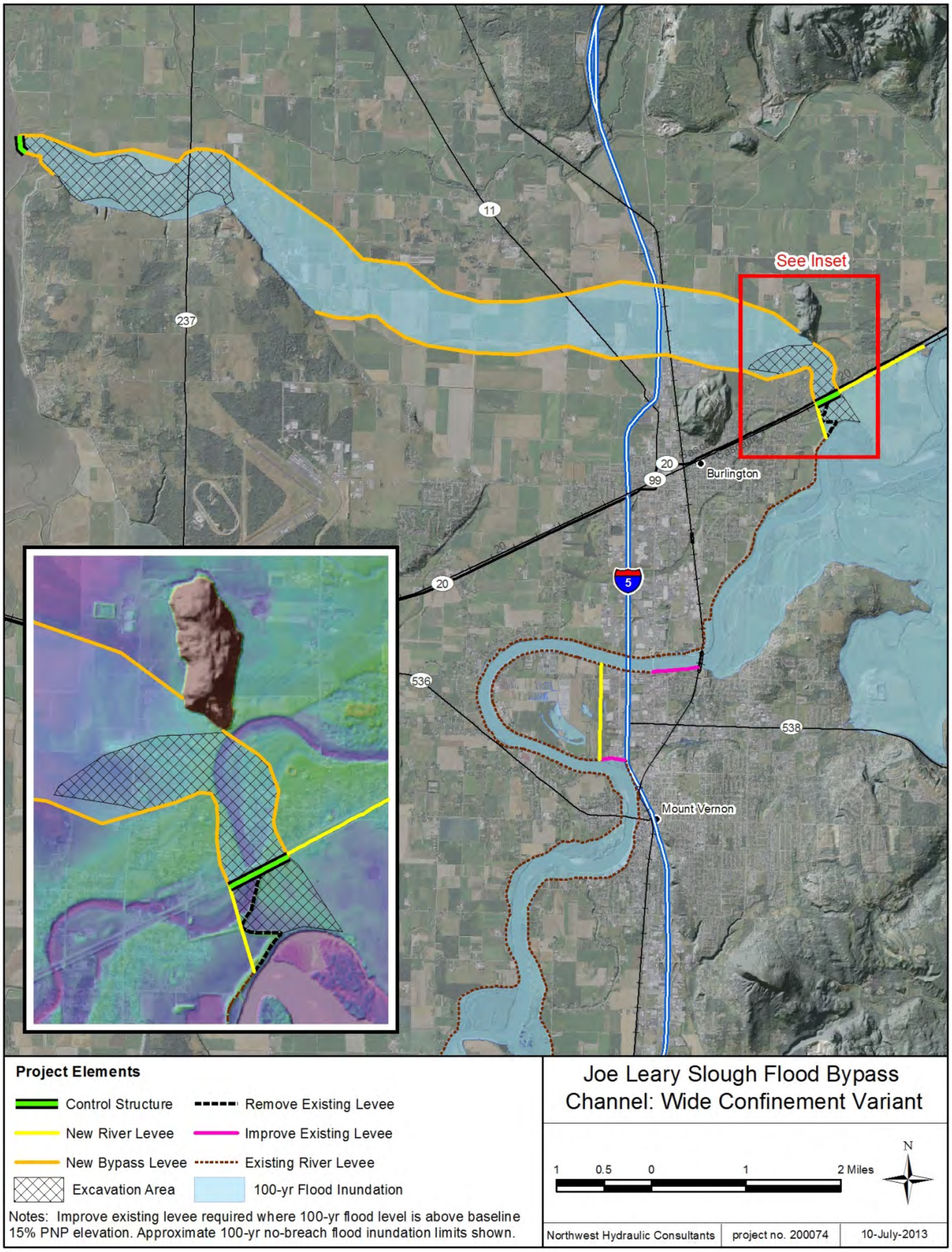


Figure 6-7: Joe Leary Slough Flood Bypass: Wide Confinement Variant

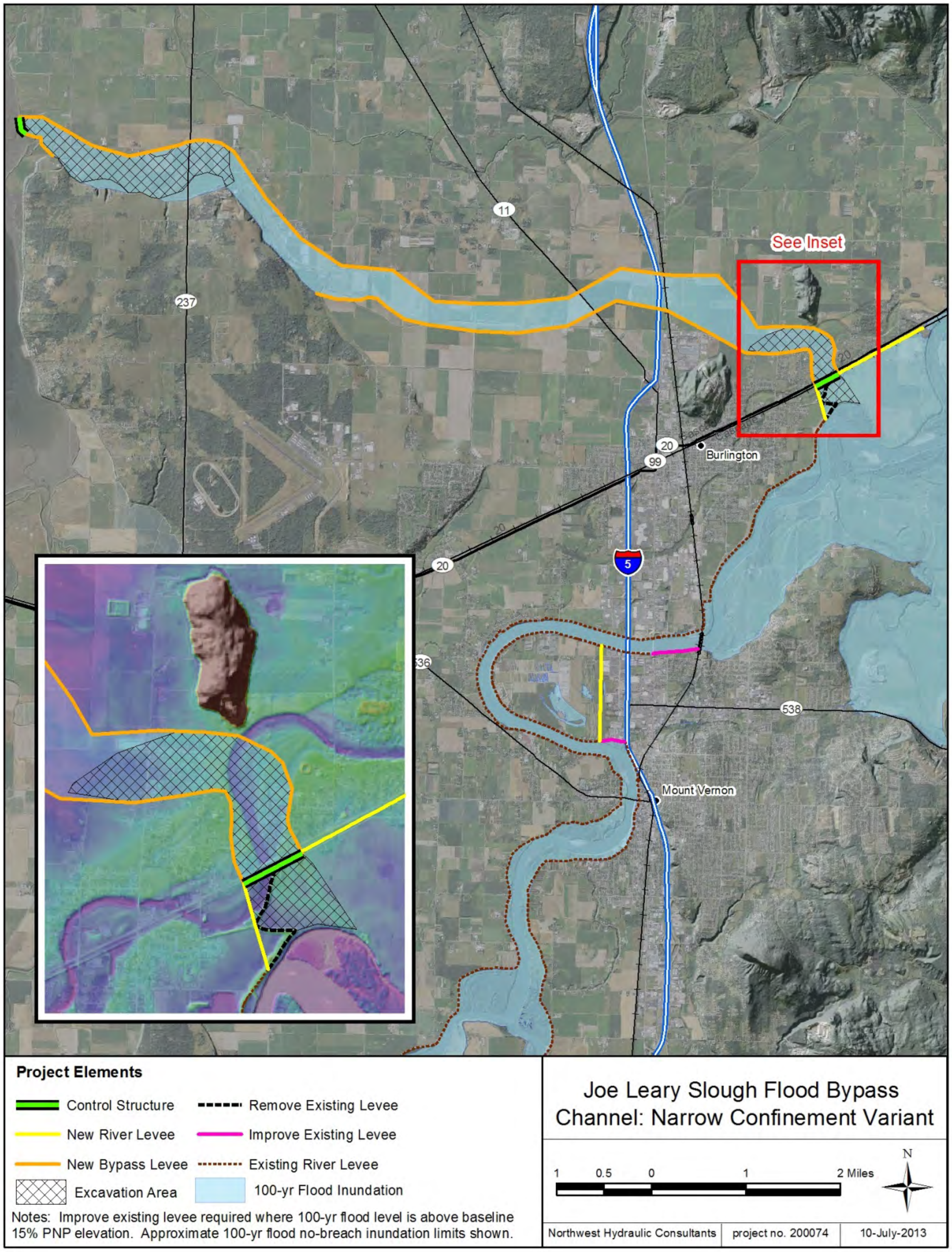


Figure 6-8: Joe Leary Slough Flood Bypass: Narrow Confinement Variant

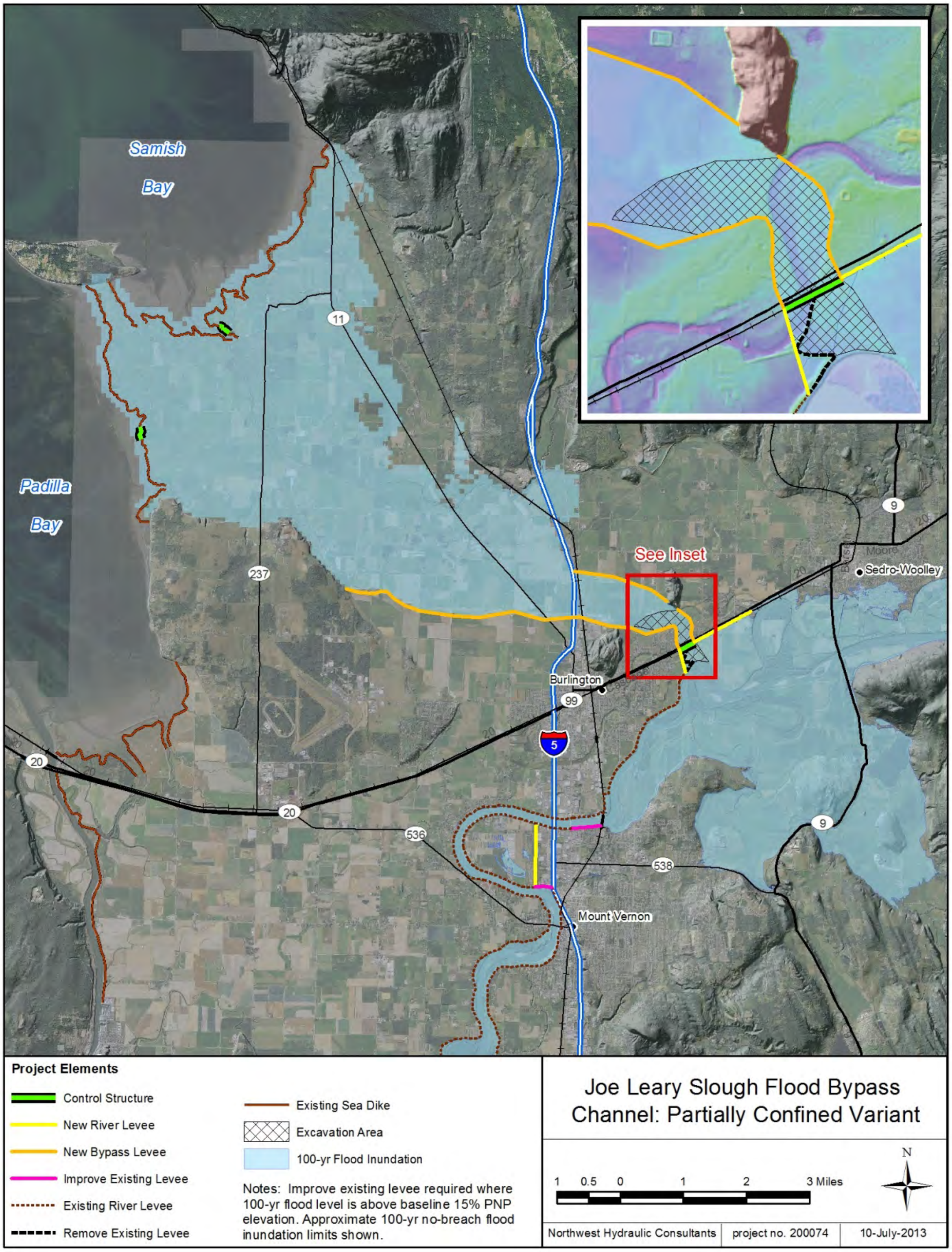


Figure 6-9: Joe Leary Slough Flood Bypass: Partially Confined Variant

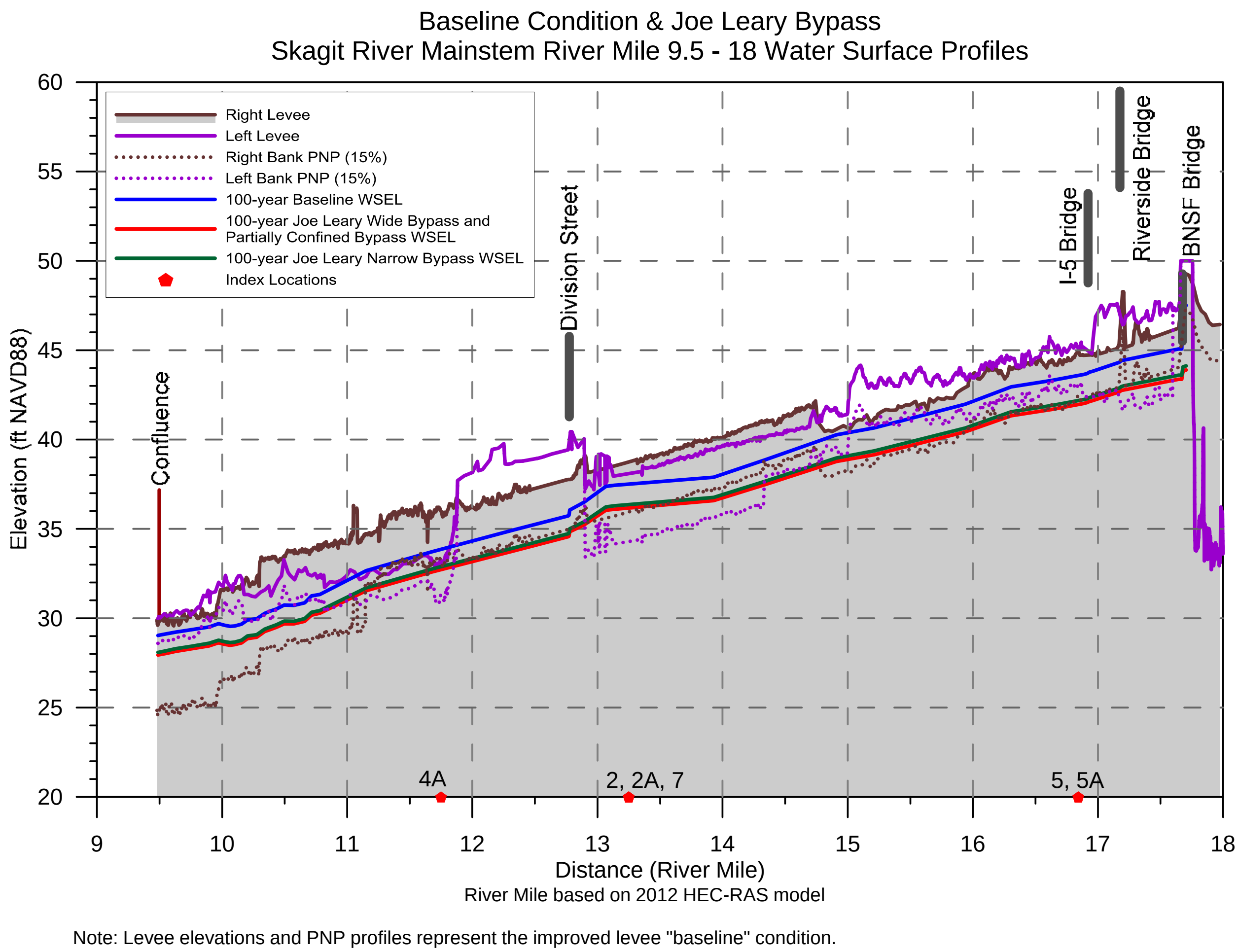
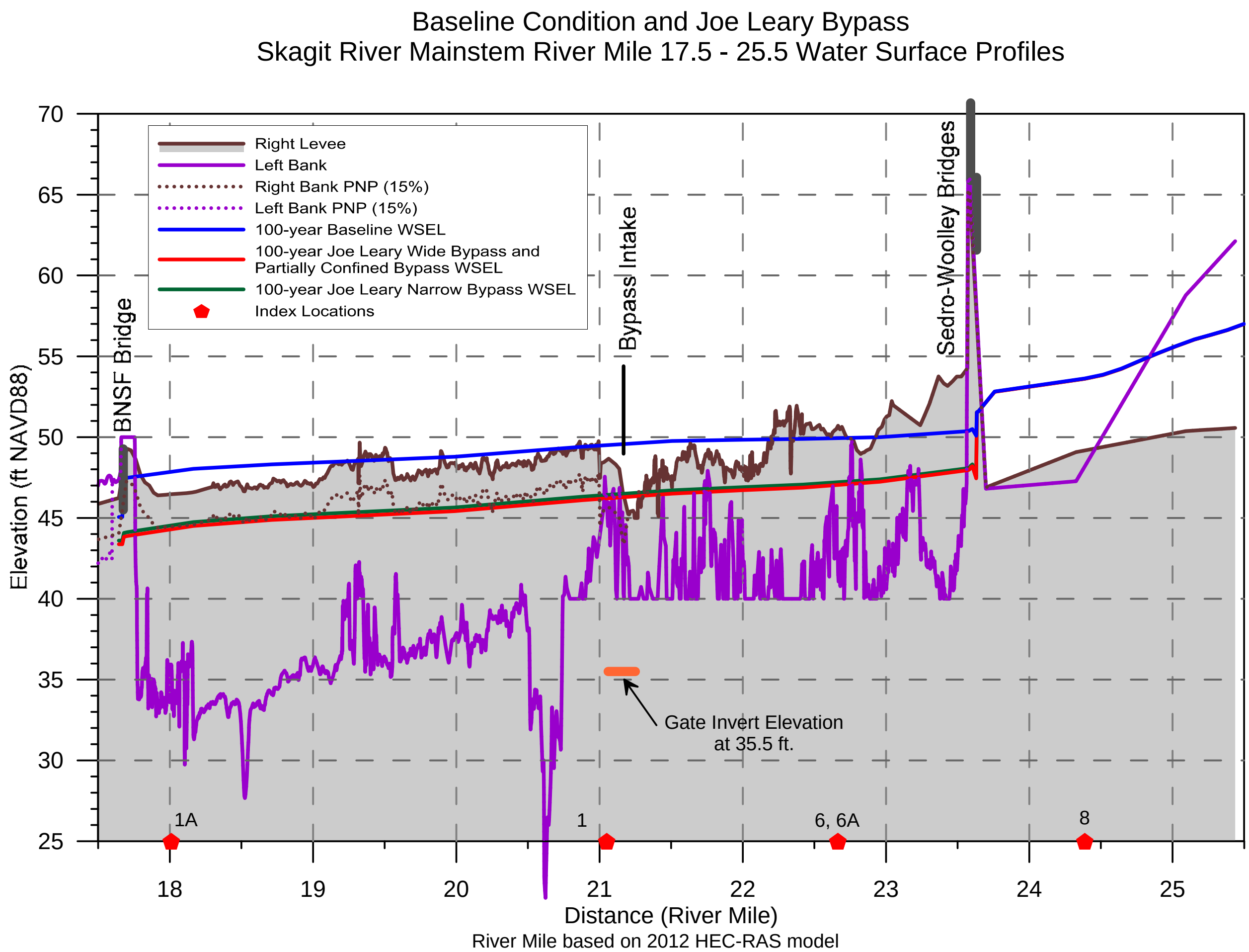


Figure 6-10: Joe Leary Slough Flood Bypass: Water Surface and PNP Profiles, RM 9.5 to RM 18



Note: Levee elevations and PNP profiles represent the improved levee "baseline" condition.

Figure 6-11: Joe Leary Slough Flood Bypass: Water Surface and PNP Profiles, RM 17.5 to RM 22.5

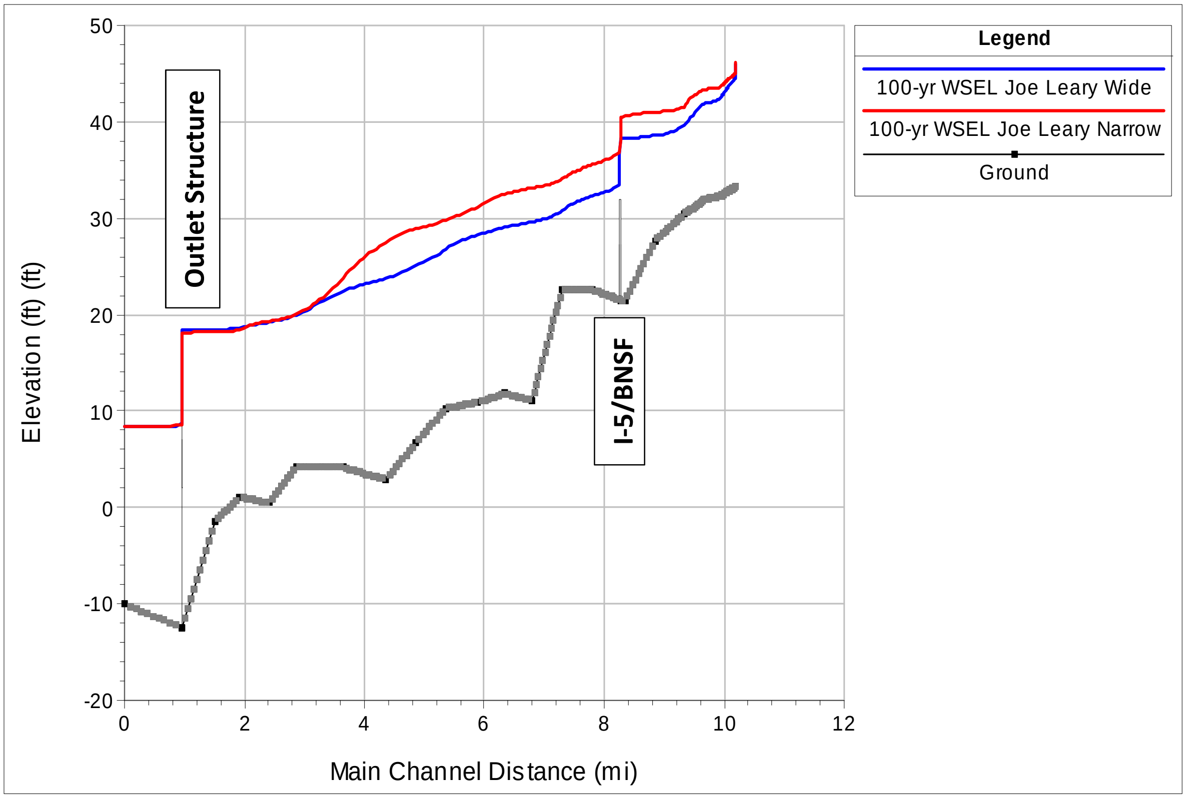


Figure 6-12: Joe Leary Slough Flood Bypass: Bypass Channel 100-yr Water Surface Elevations

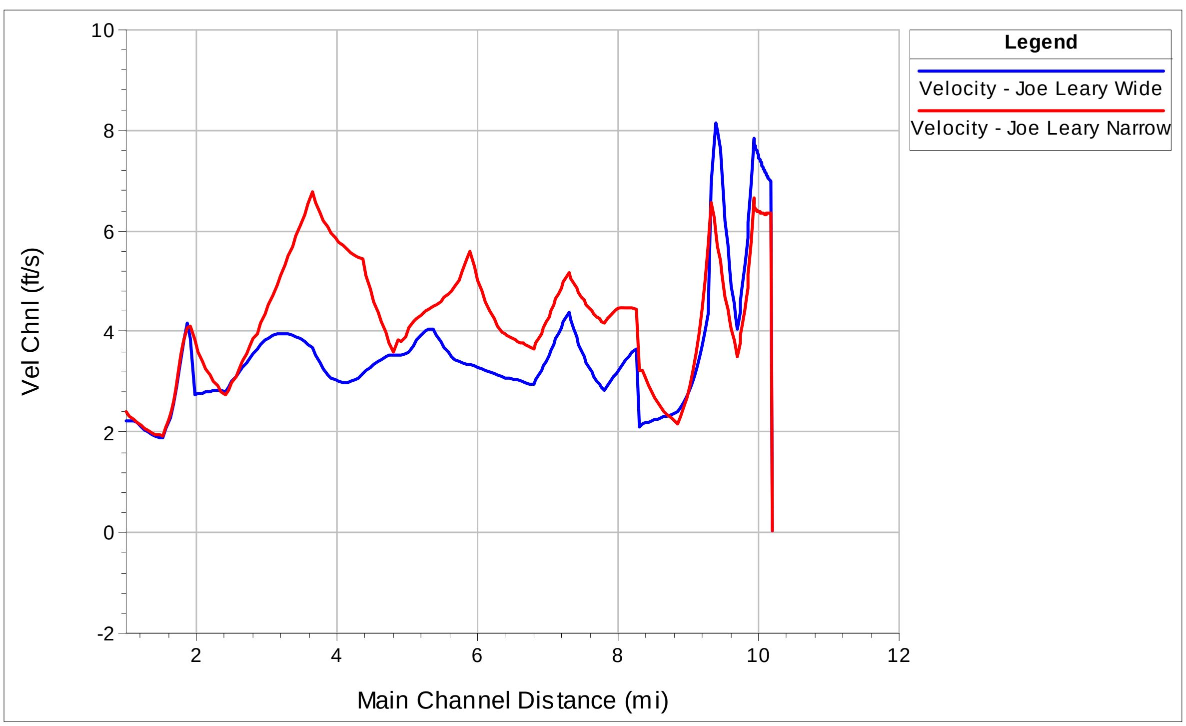


Figure 6-13: Joe Leary Slough Flood Bypass: Bypass Channel Velocity for 100-yr Event

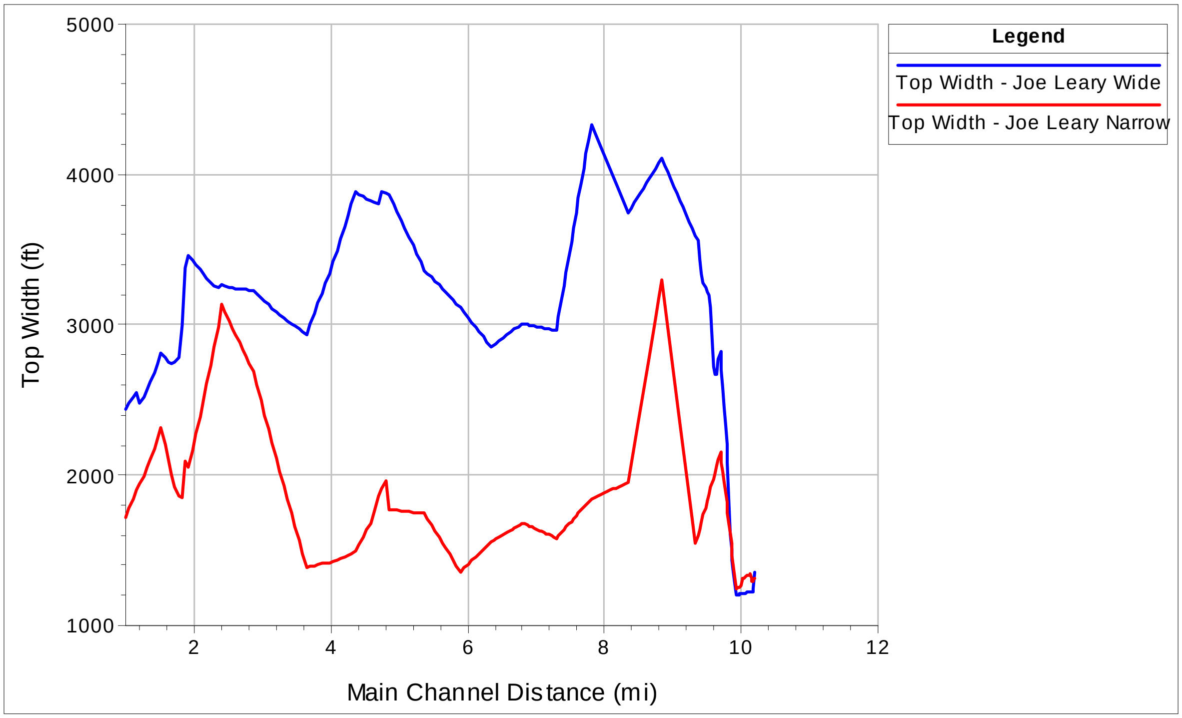


Figure 6-14: Joe Leary Slough Flood Bypass: Bypass Channel 100-yr Water Surface Top Width

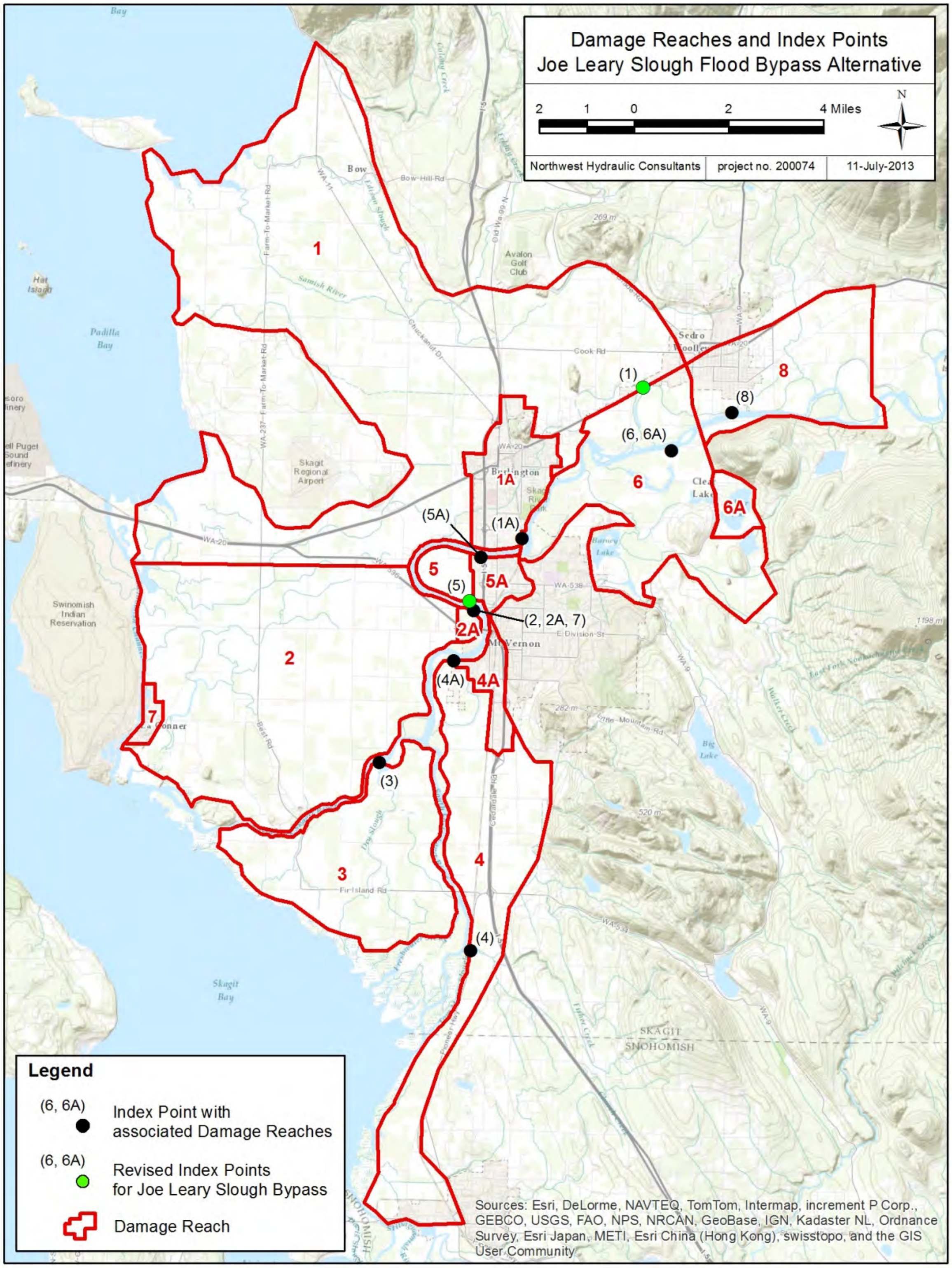


Figure 6-15: Damage Reaches and Index Points for Joe Leary Slough Flood Bypass Alternative

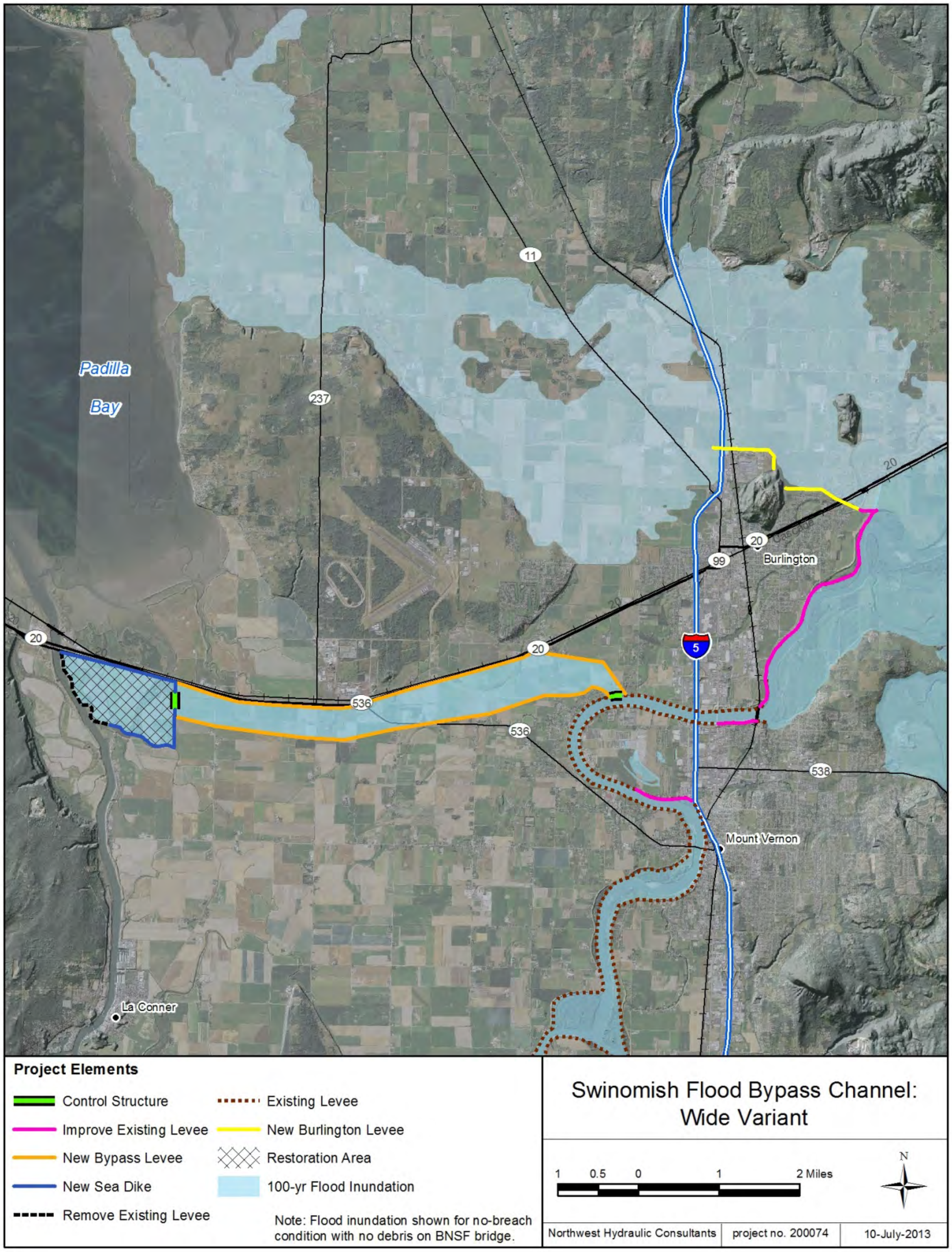


Figure 6-16: Swinomish Flood Bypass: Wide Confinement Variant

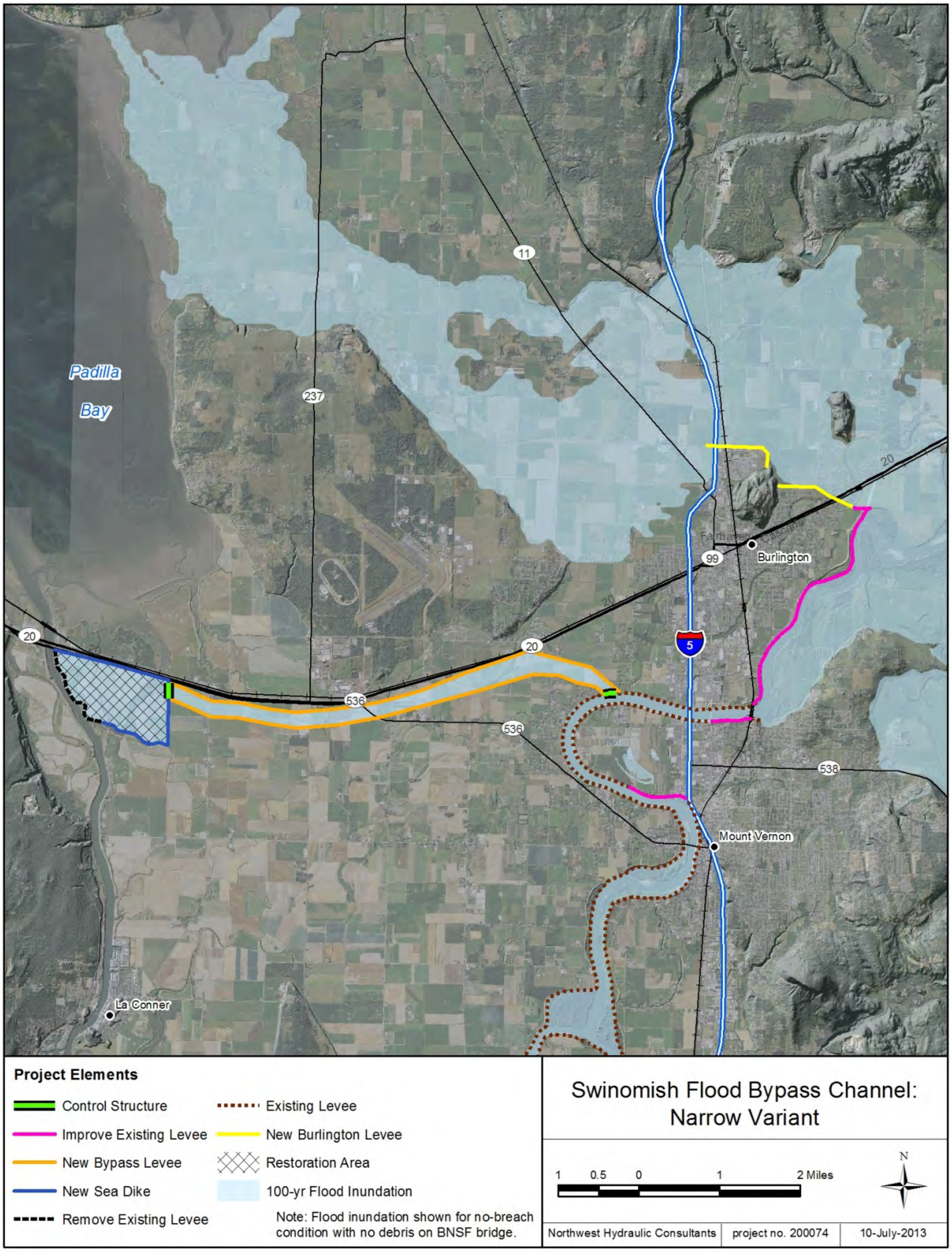


Figure 6-17: Swinomish Flood Bypass: Narrow Confinement Variant

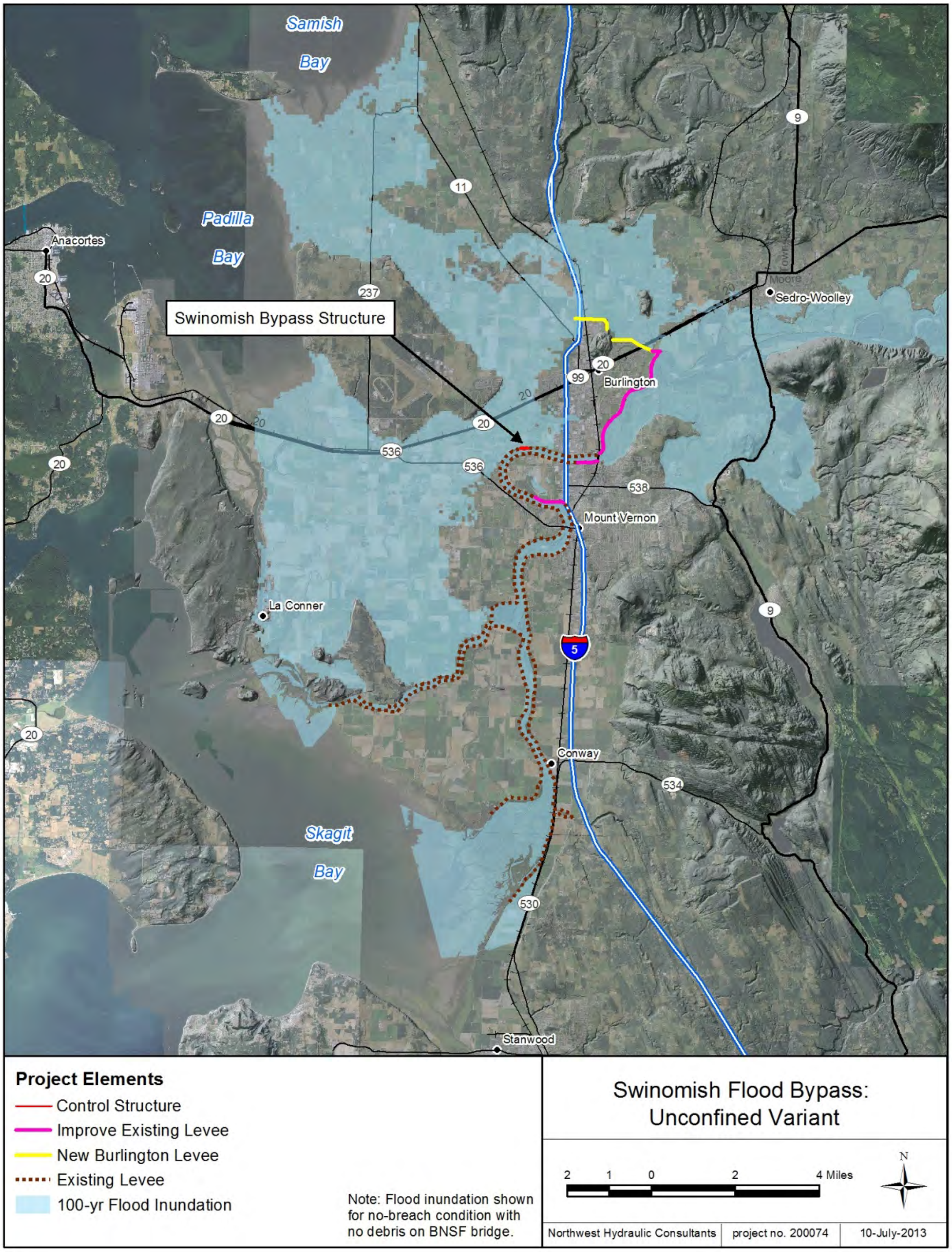


Figure 6-18: Swinomish Flood Bypass: Unconfined Variant

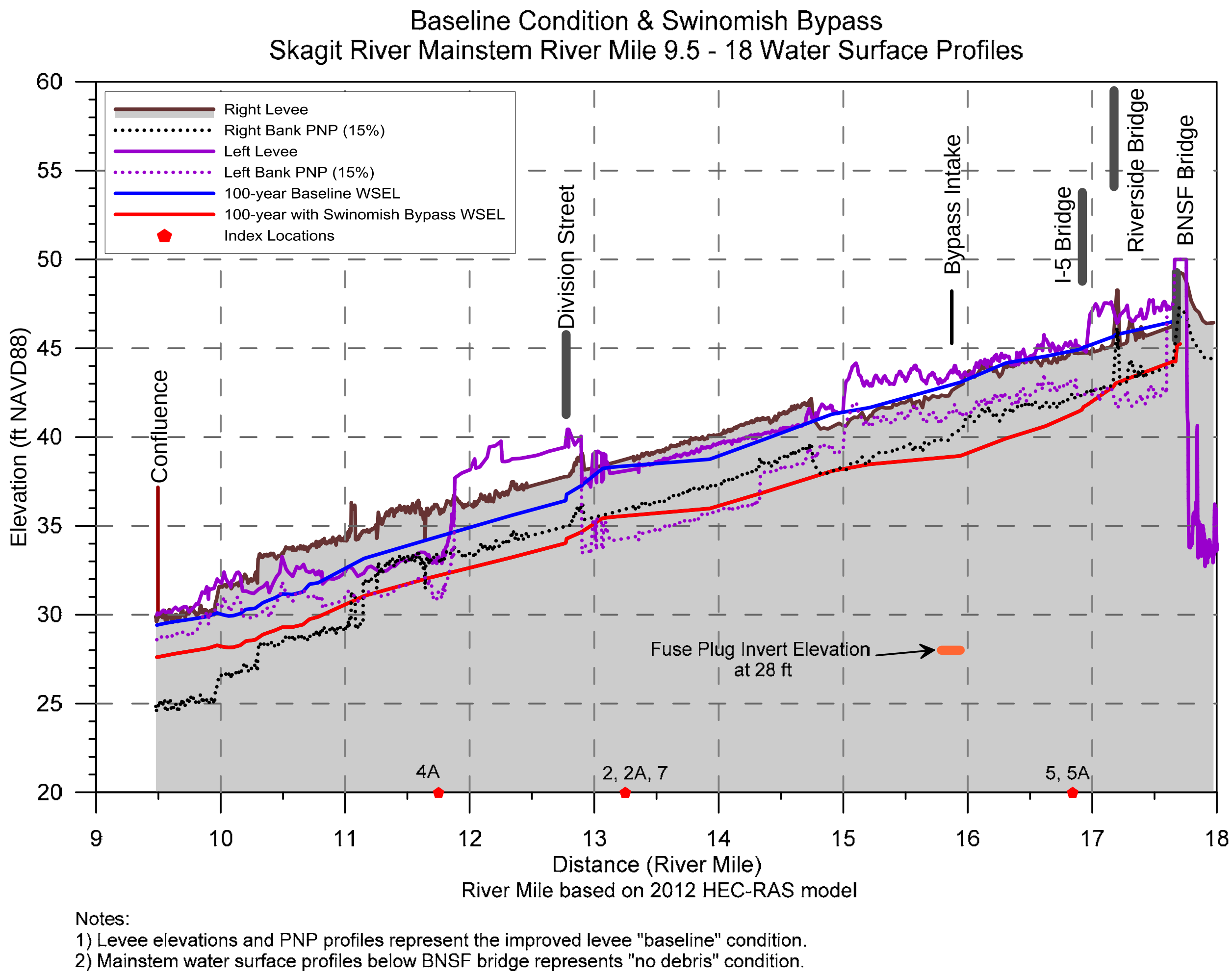
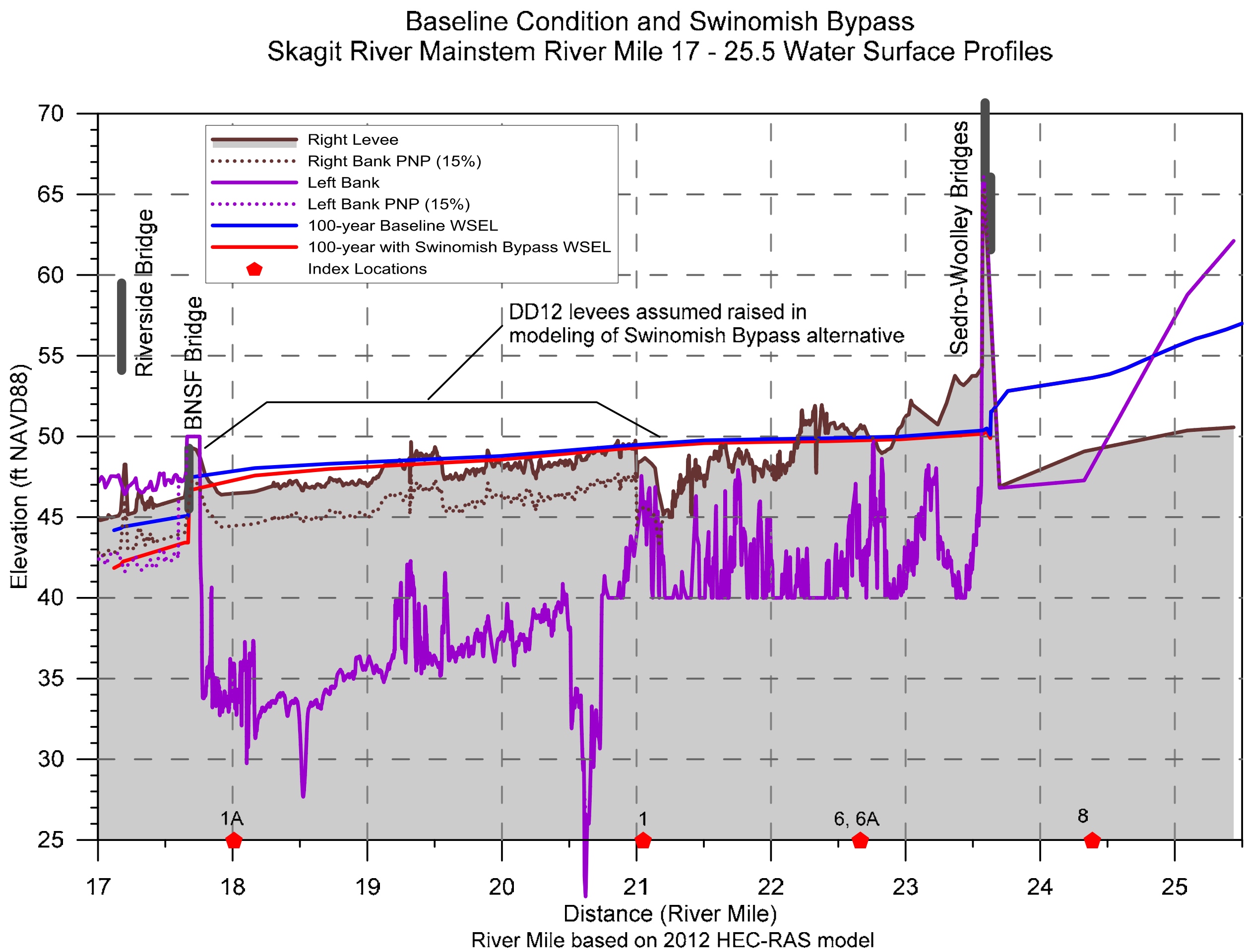


Figure 6-19: Swinomish Flood Bypass: Water Surface and PNP Profiles, RM 9.5 to RM 18



Notes:
1) Levee elevations and PNP profiles represent the improved levee "baseline" condition.
2) Mainstem water surface profiles upstream from BNSF bridge represent "with debris" condition.

Figure 6-20: Swinomish Flood Bypass: Water Surface and PNP Profiles, RM 17 to RM 25.5

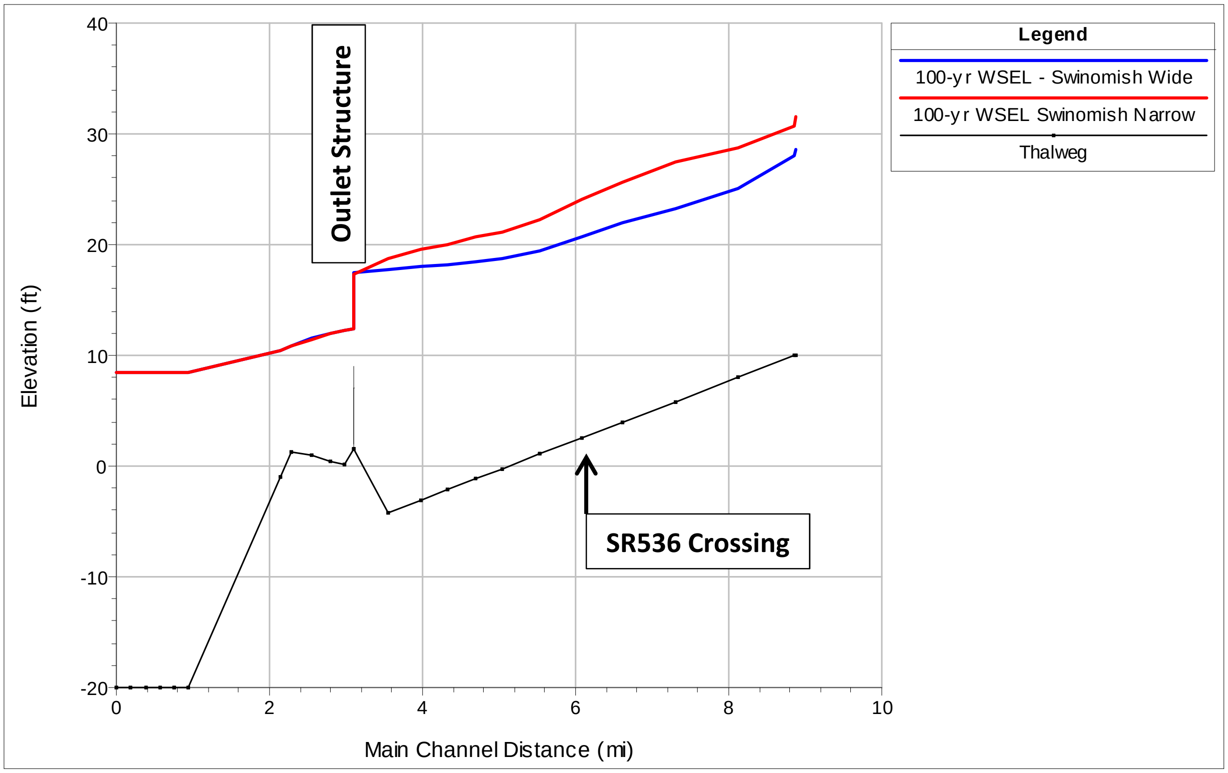


Figure 6-21: Swinomish Flood Bypass: Bypass Channel 100-yr Water Surface Elevations

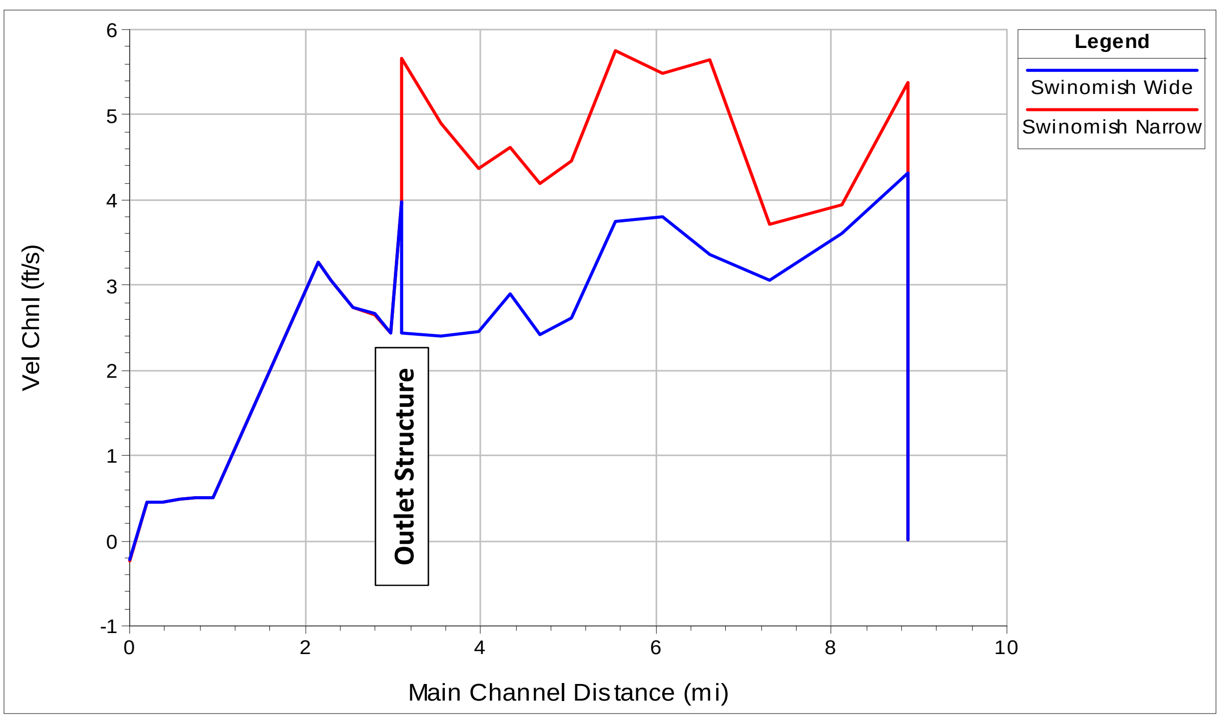


Figure 6-22: Swinomish Flood Bypass: Bypass Channel Velocity for 100-year Event

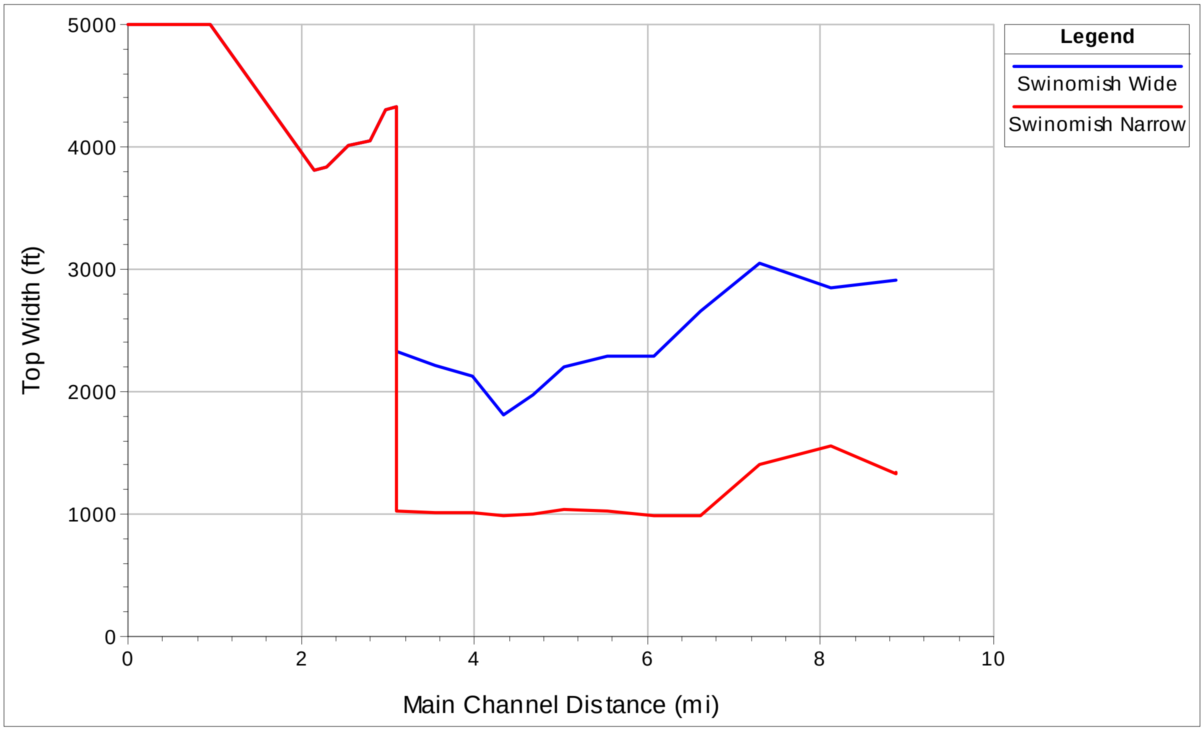


Figure 6-23: Swinomish Flood Bypass: Bypass Channel 100-yr Water Surface Top Width

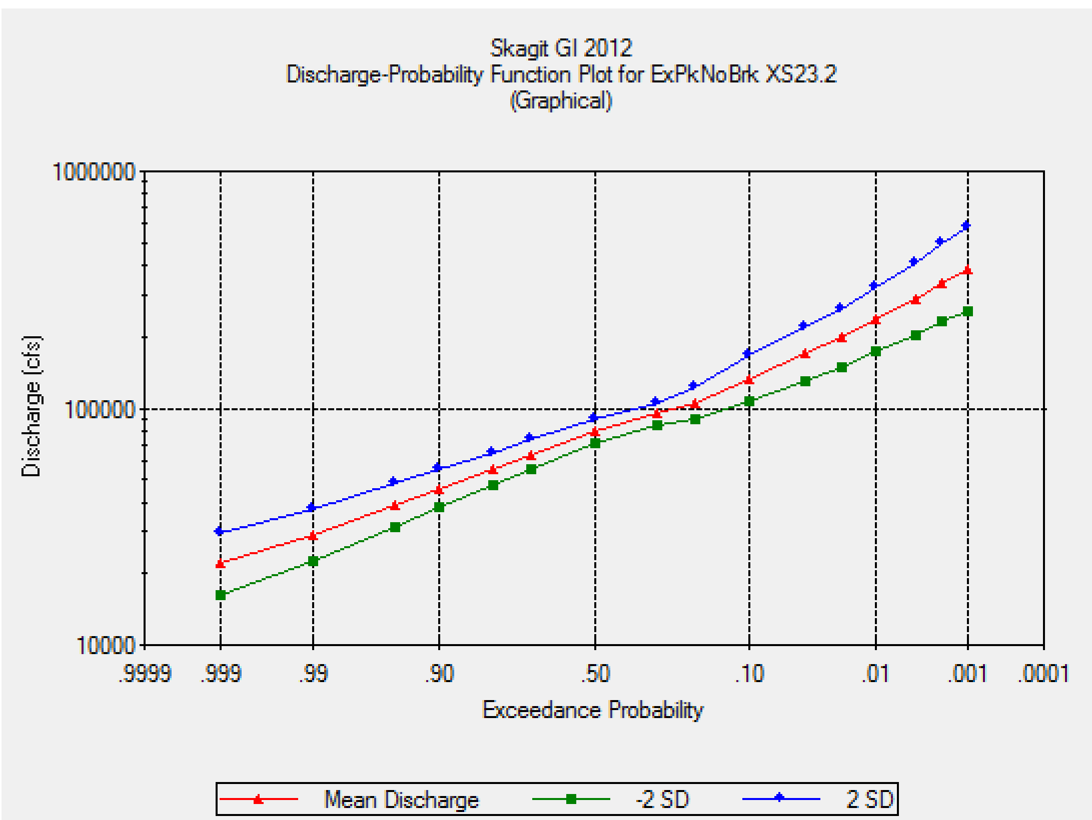
APPENDIX 5-1

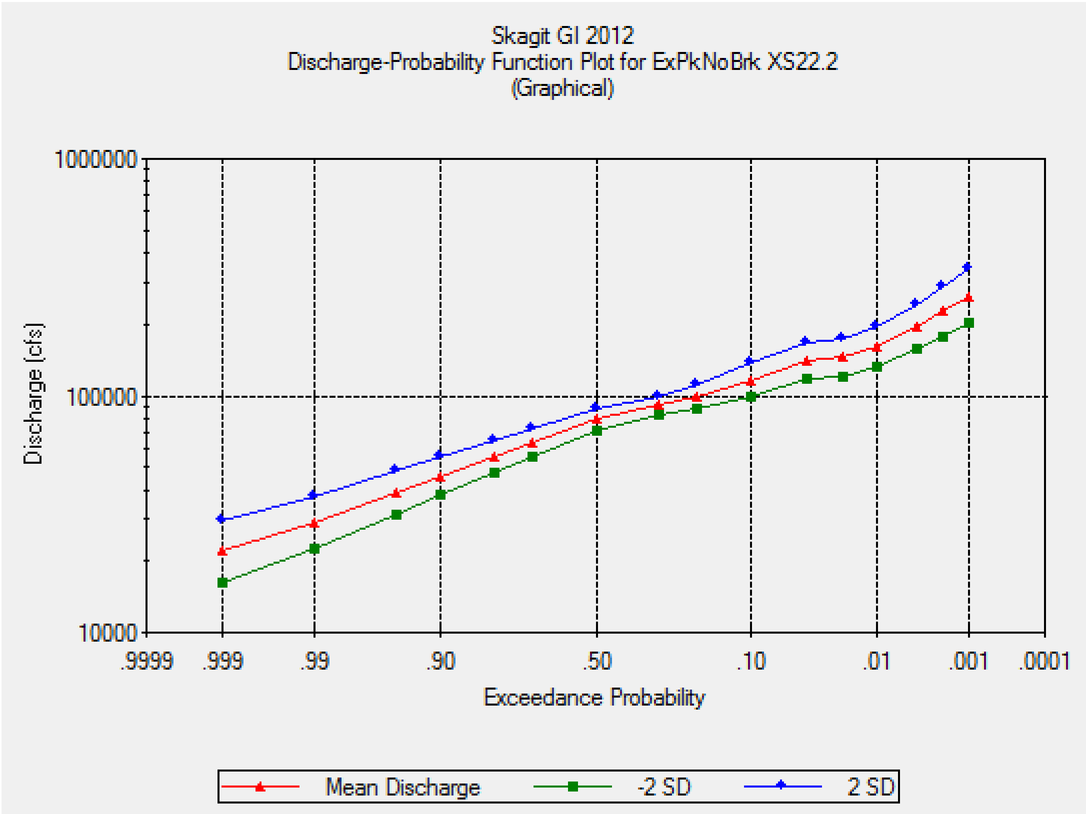
Existing Condition No-Breach Simulation Results

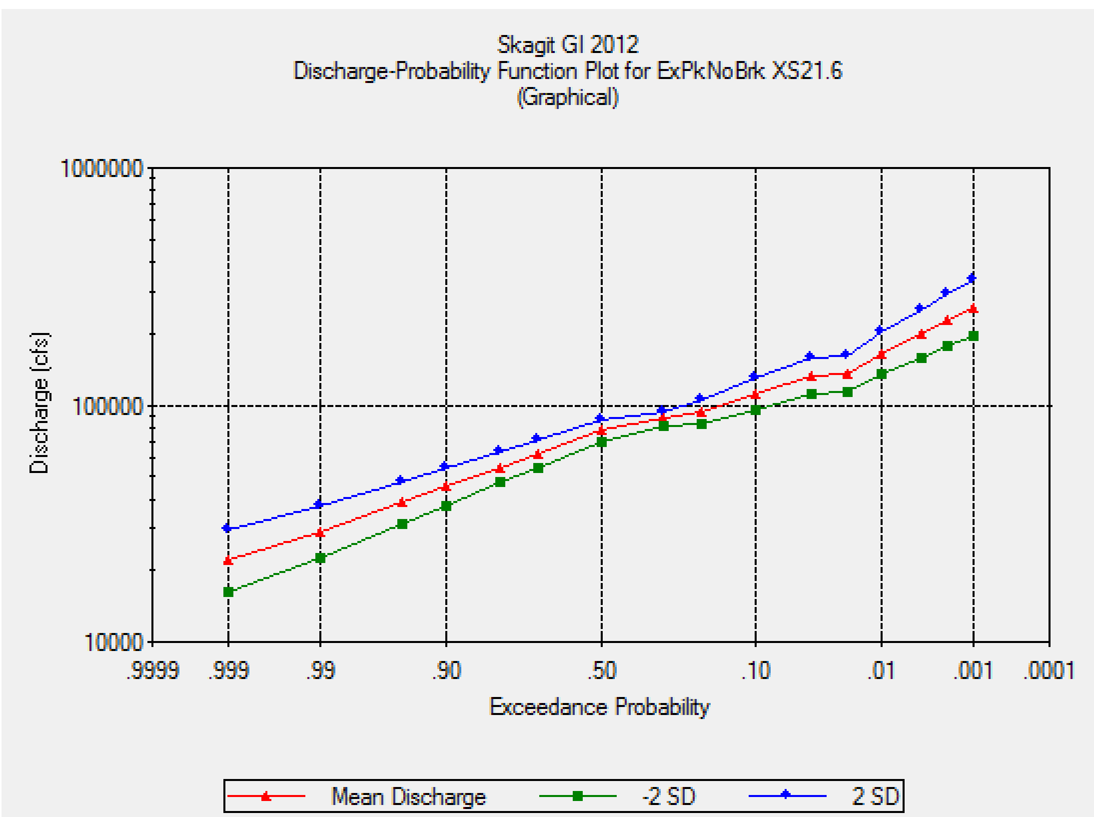
Existing Condition No-Breach Simulation Results

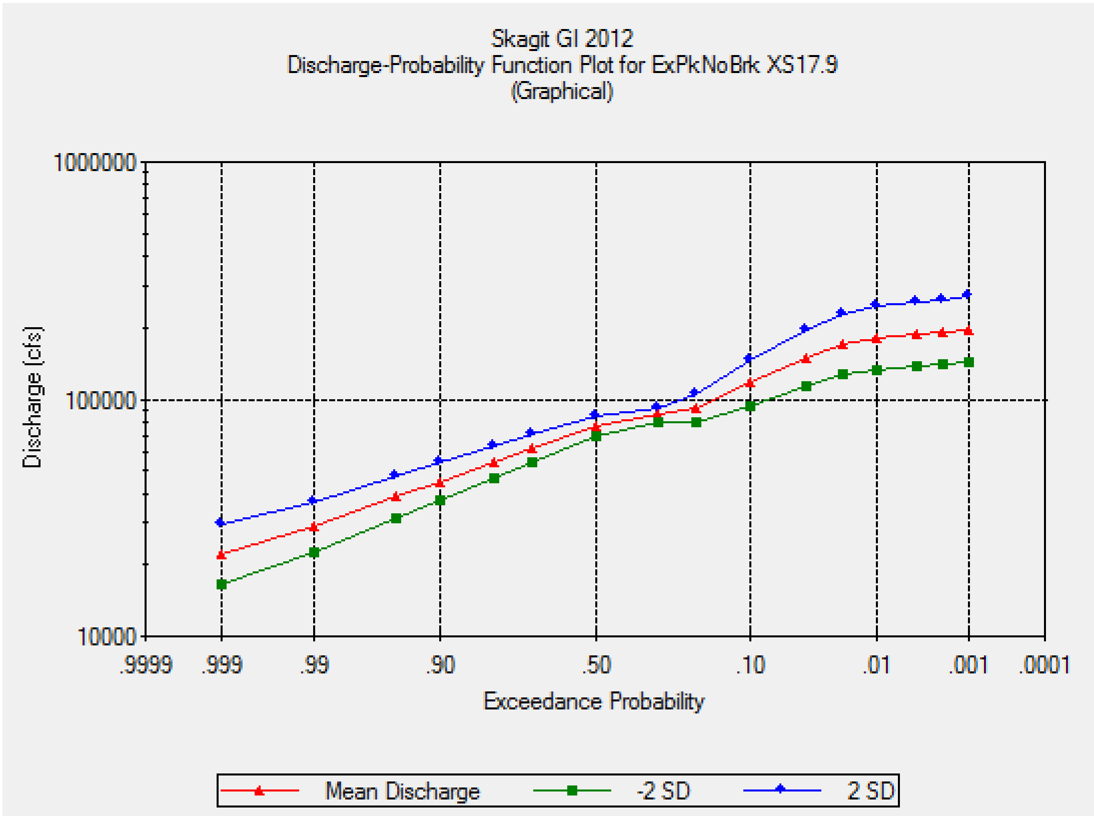
Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)								
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	80,400	79,500	77,900	77,100	77,000	76,900	76,900	34,900	41,700
5	105,200	99,000	93,600	91,300	91,300	91,300	91,300	42,400	48,900
10	133,300	116,800	111,300	117,100	117,100	117,100	117,000	55,800	61,200
25	169,600	140,100	132,500	149,200	149,100	149,000	149,000	72,200	76,600
50	197,500	144,900	135,200	170,600	163,300	163,300	163,300	78,300	82,500
75	220,100	156,500	152,500	177,600	165,500	165,500	165,500	78,900	83,200
100	235,800	161,200	164,800	180,700	166,400	166,400	166,300	79,100	83,400
250	289,900	195,300	199,000	187,400	168,000	168,000	167,800	79,500	83,800
500	337,400	227,400	226,800	191,800	169,000	168,900	168,700	79,700	84,100

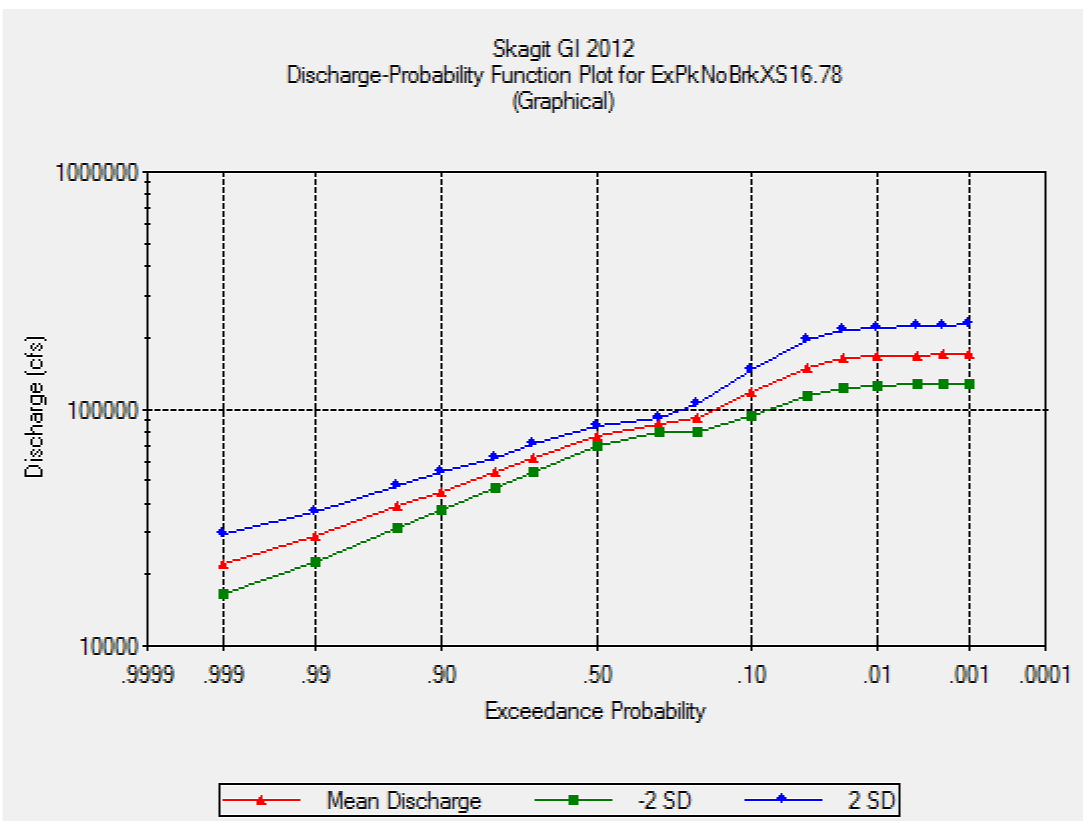
Indicates possible overestimation of flow due to questionable HEC-RAS model behavior - data should not be used without further evaluation

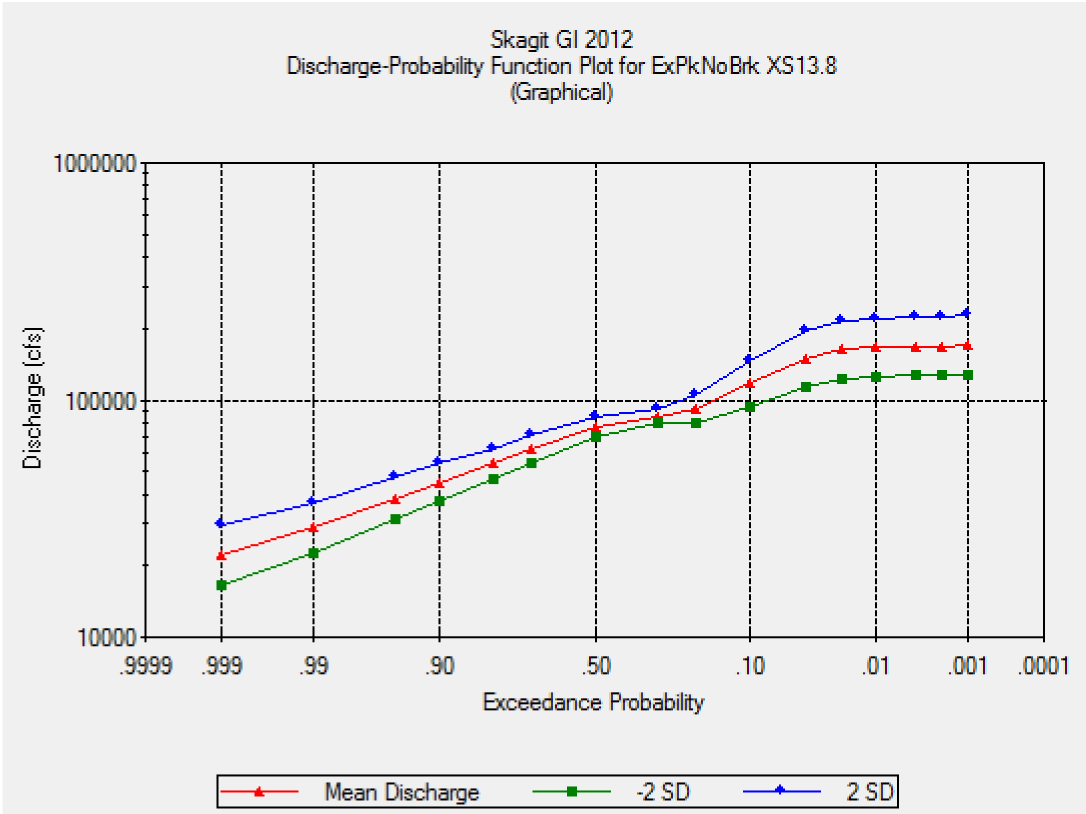


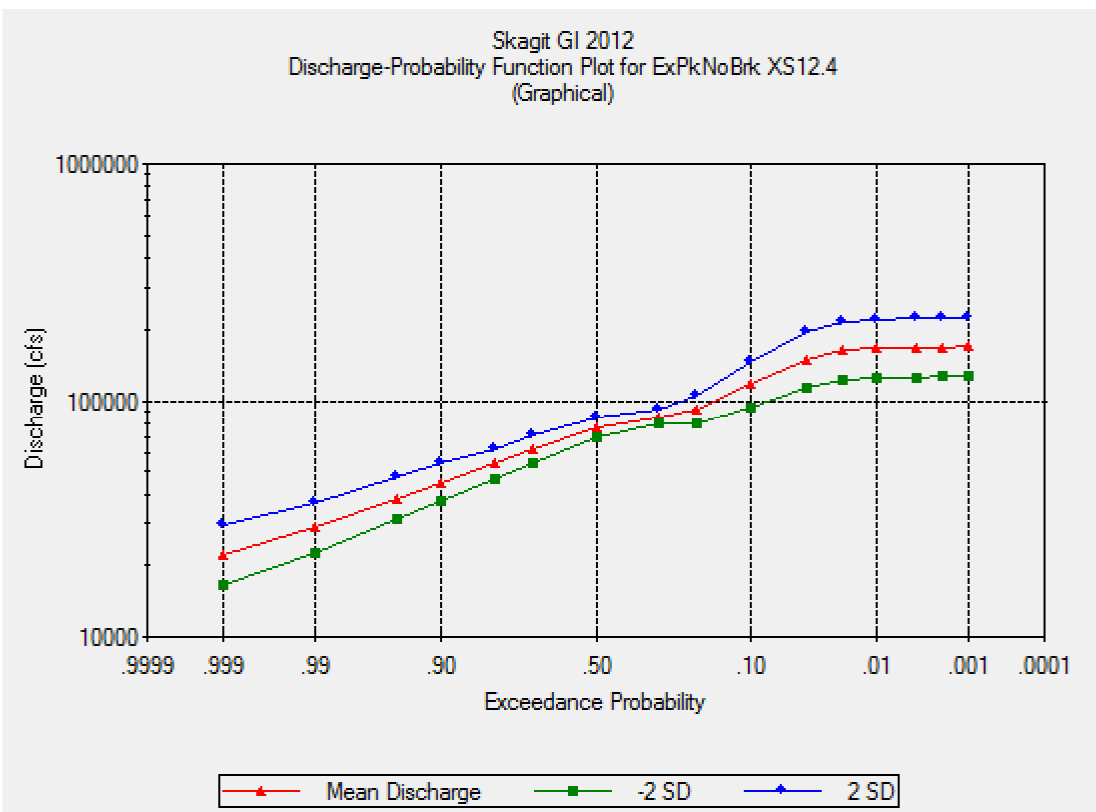

Skagit GI 2012
Discharge-Probability Function Plot for ExPkNoBrk XS22.2
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

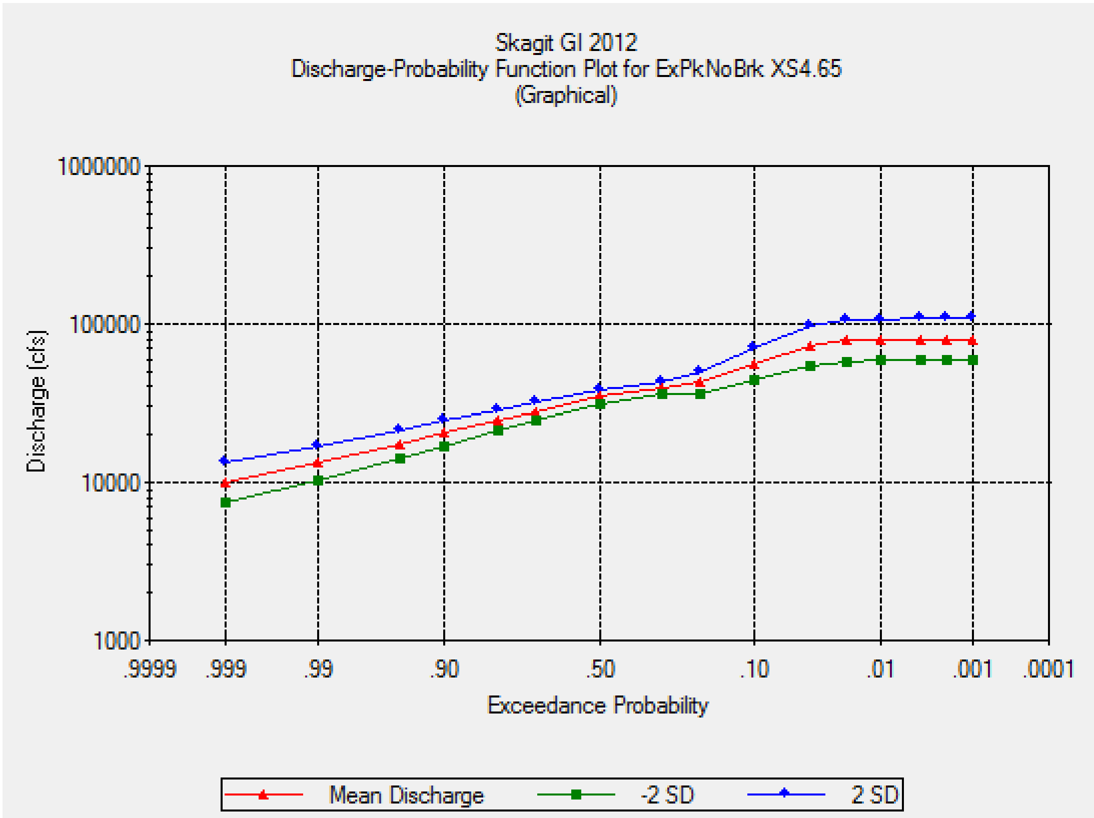

Skagit GI 2012
Discharge-Probability Function Plot for ExPkNoBrk XS21.6
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

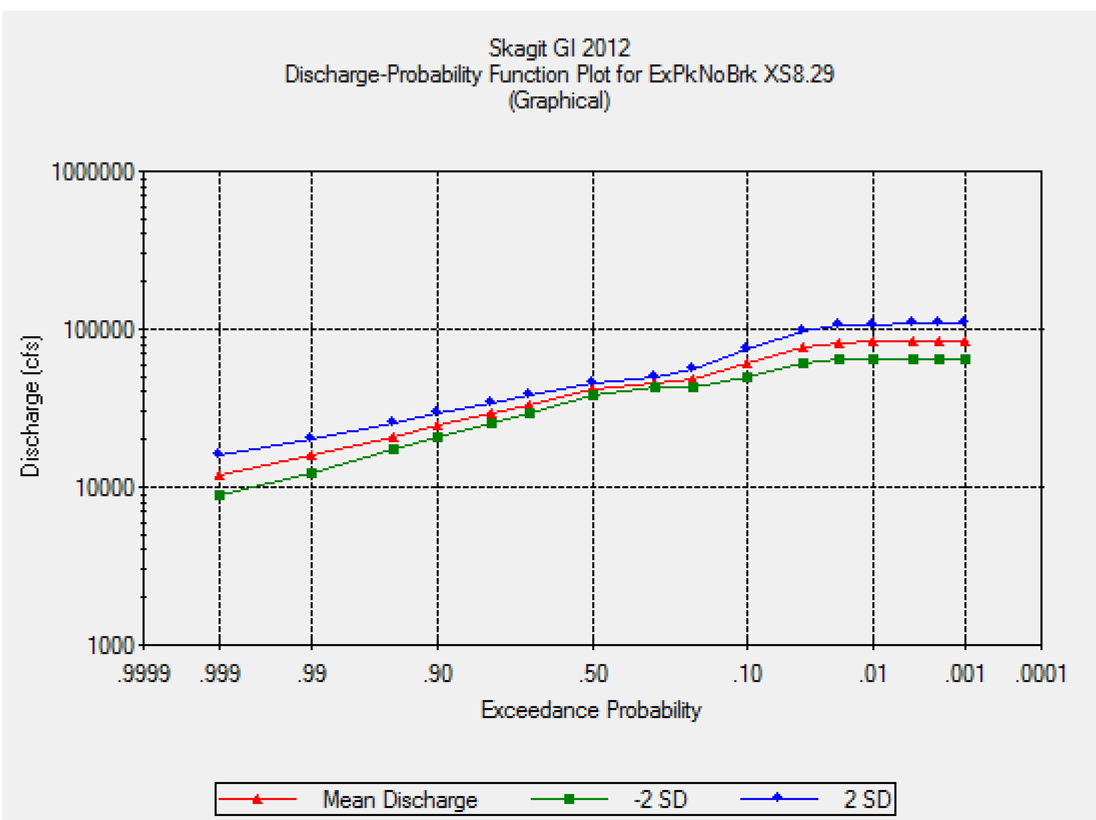

Skagit GI 2012
Discharge-Probability Function Plot for ExPkNoBrk XS17.9
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

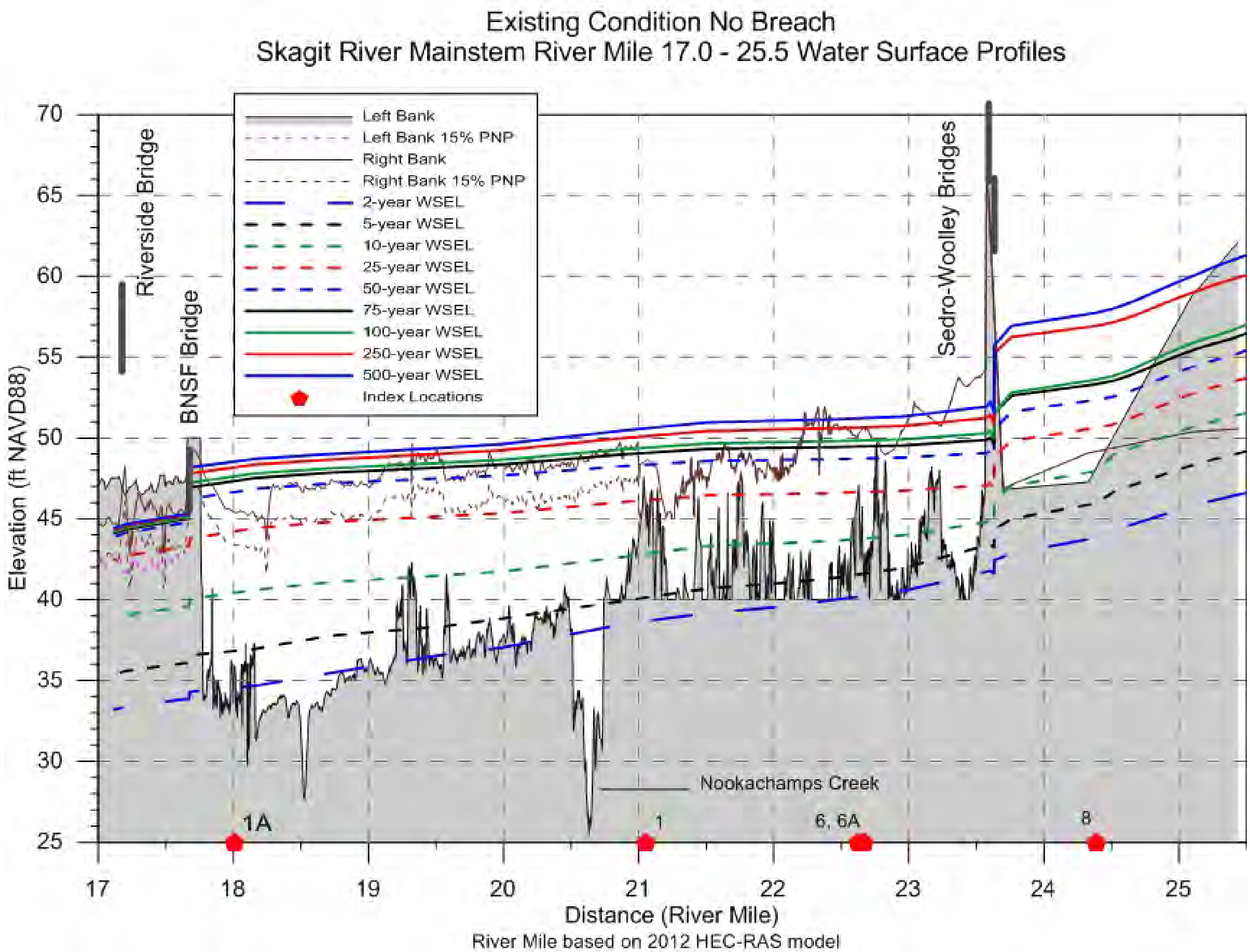

Skagit GI 2012
Discharge-Probability Function Plot for ExPkNoBrkXS16.78
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

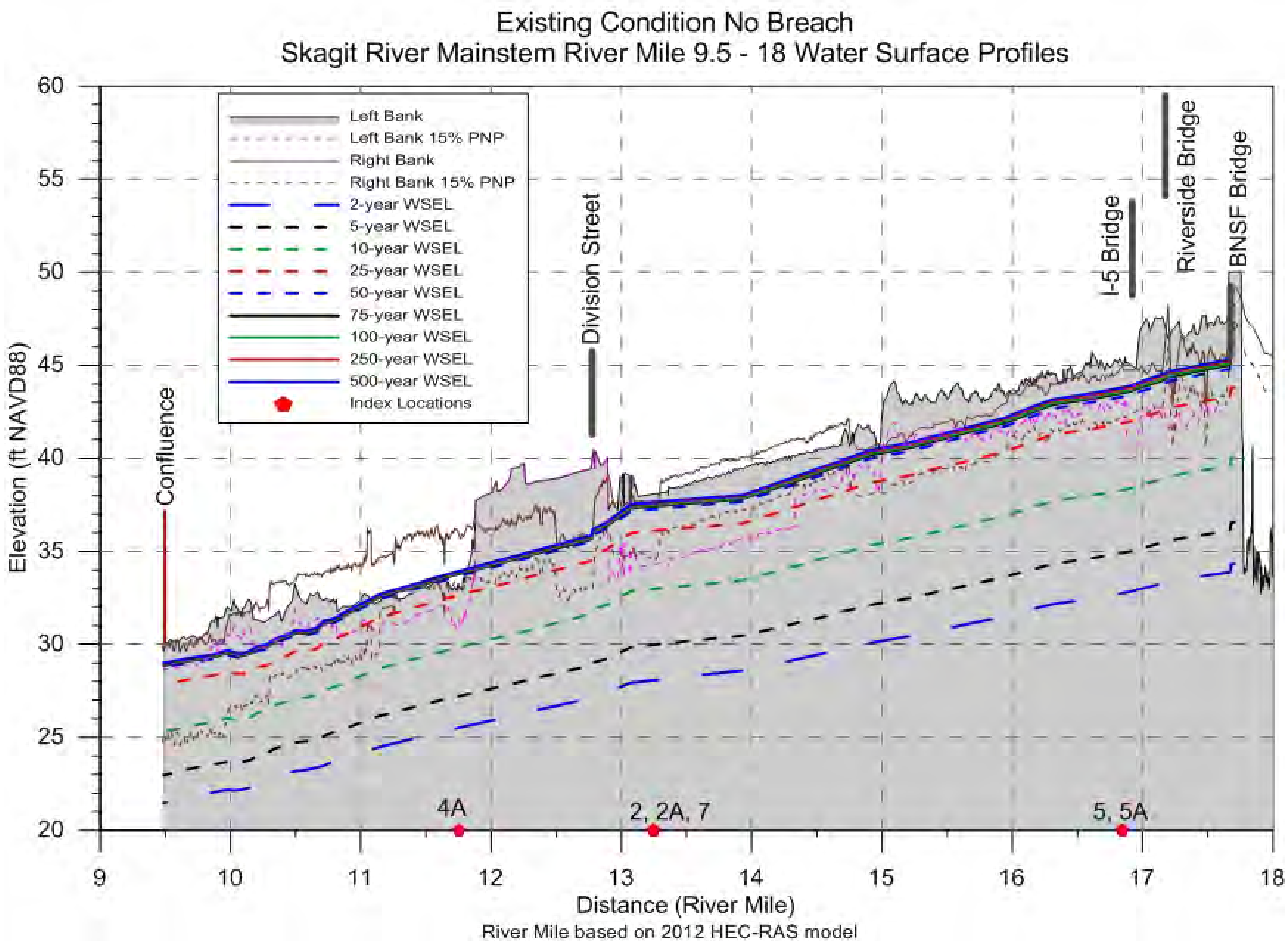

Skagit GI 2012
Discharge-Probability Function Plot for ExPkNoBrk XS13.8
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Skagit GI 2012
Discharge-Probability Function Plot for ExPkNoBrk XS12.4
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

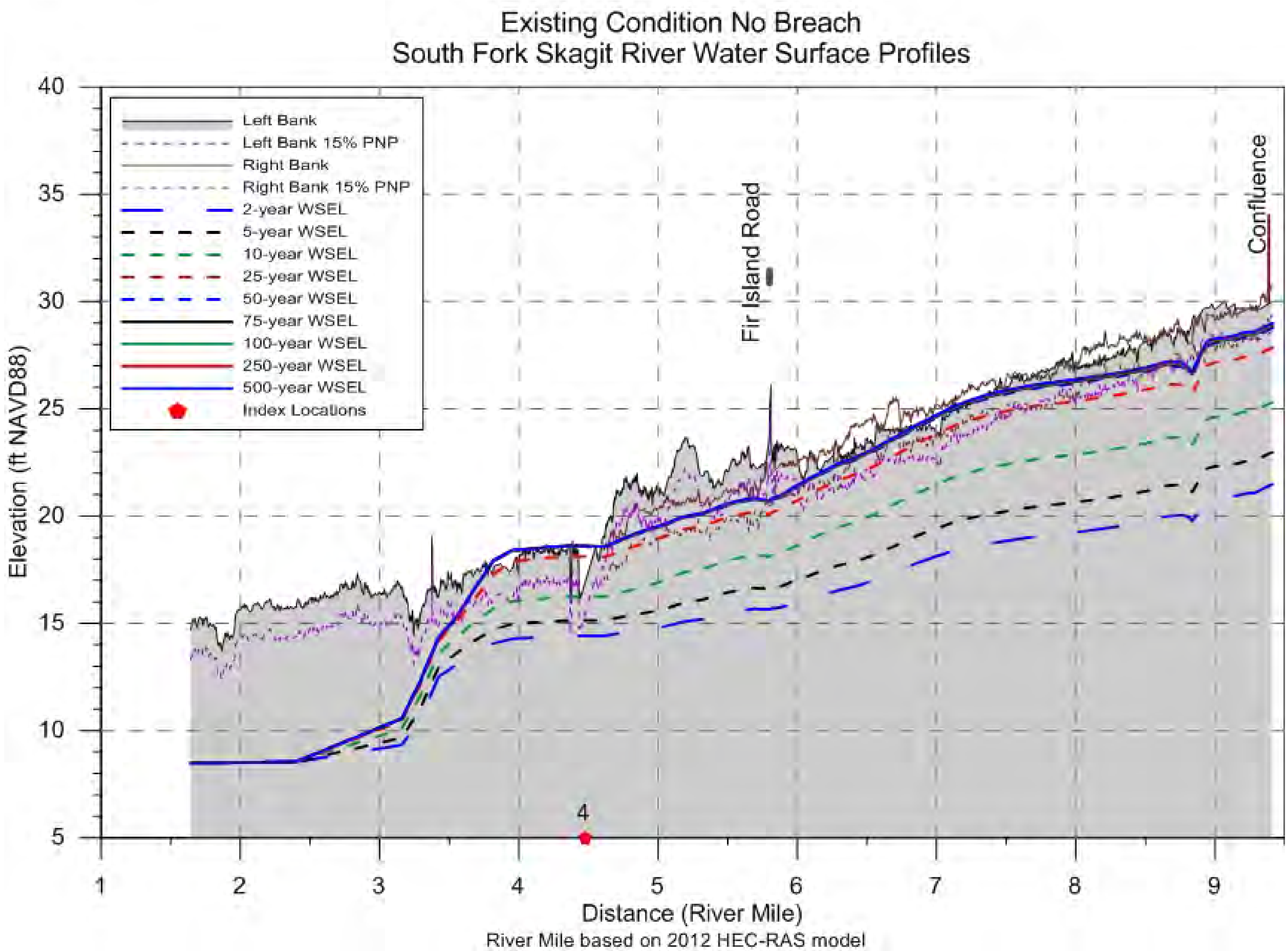

Skagit GI 2012
Discharge-Probability Function Plot for ExPkNoBrk XS4.65
(Graphical)
Discharge (cfs)
1000000
100000
10000
1000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

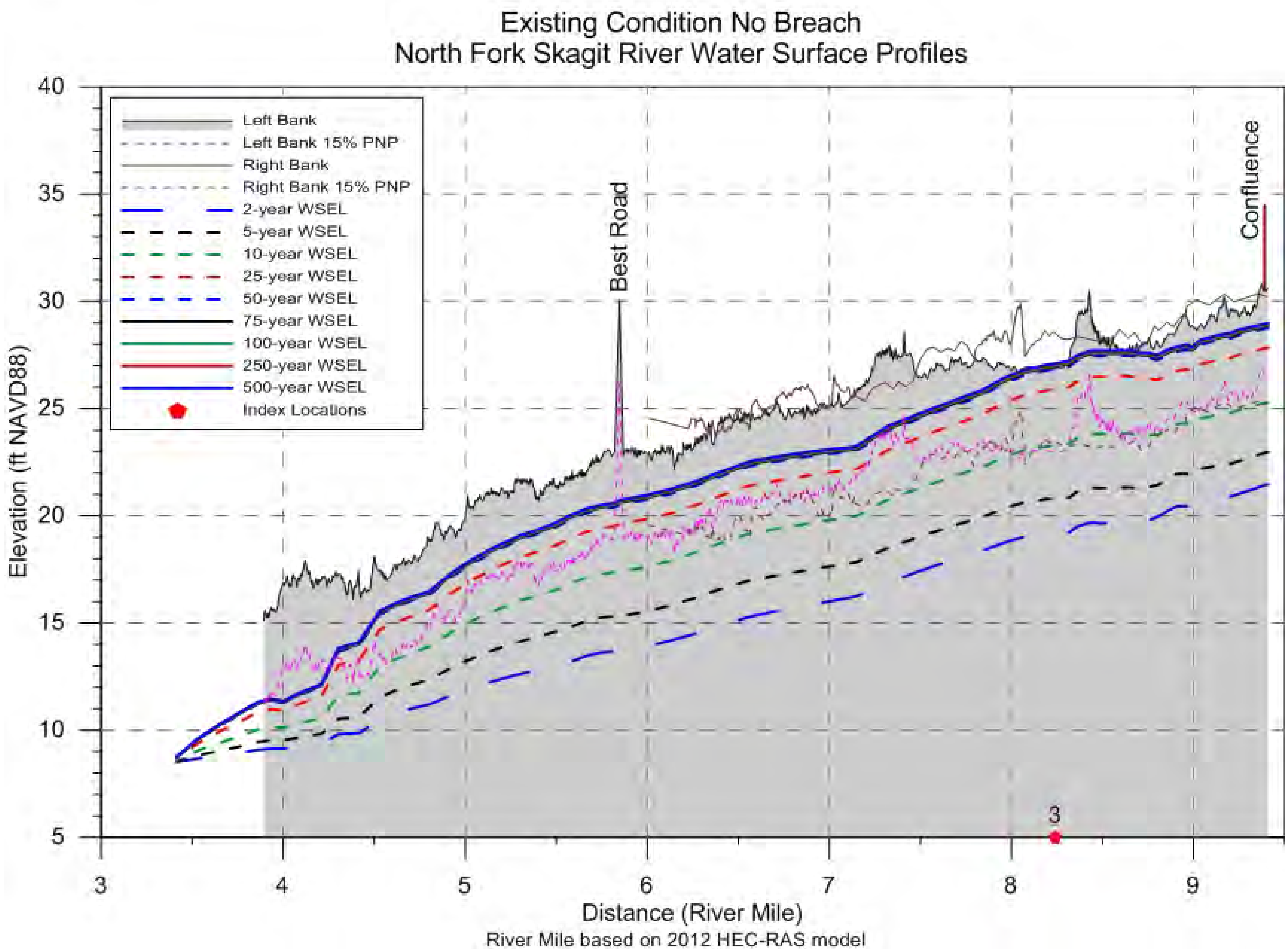

Skagit GI 2012
Discharge-Probability Function Plot for ExPkNoBrk XS8.29
(Graphical)
Discharge (cfs)
1000000
100000
10000
1000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

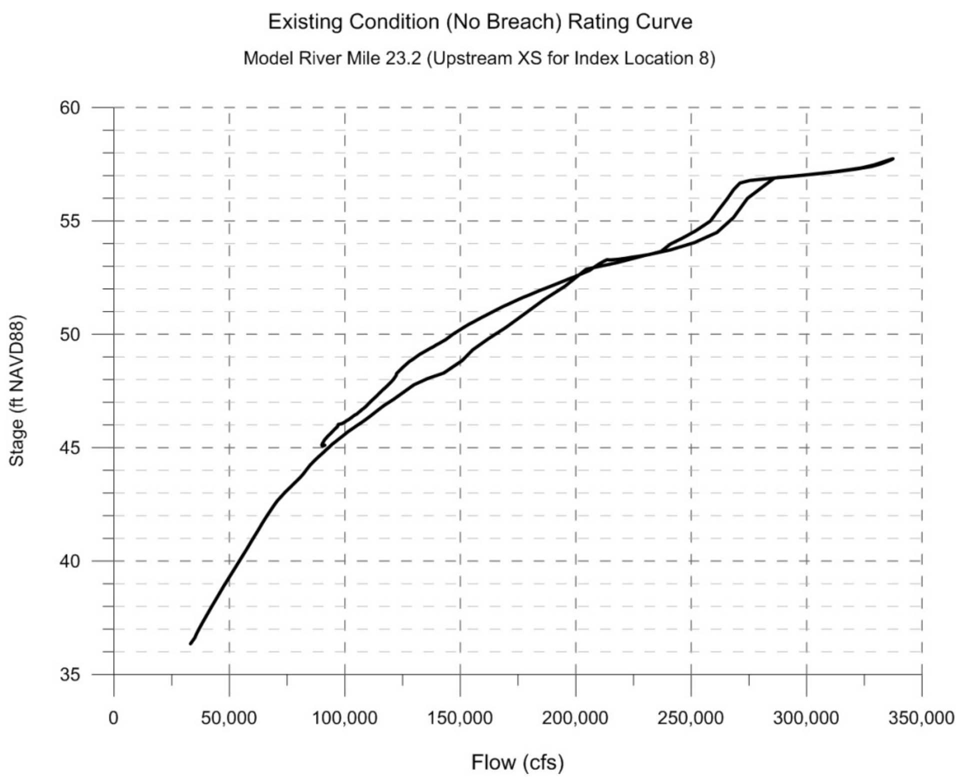

Existing Condition No Breach
Skagit River Mainstem River Mile 17.0 - 25.5 Water Surface Profiles
Left Bank
Left Bank 15% PNP
Right Bank
Right Bank 15% PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Index Locations
Riverside Bridge
BNSF Bridge
Sedro-Woolley Bridges
Nookachamps Creek
1A
1
6, 6A
8
Elevation (ft NAVD88)
70
65
60
55
50
45
40
35
30
25
17
18
19
20
21
22
23
24
25
Distance (River Mile)
River Mile based on 2012 HEC-RAS model



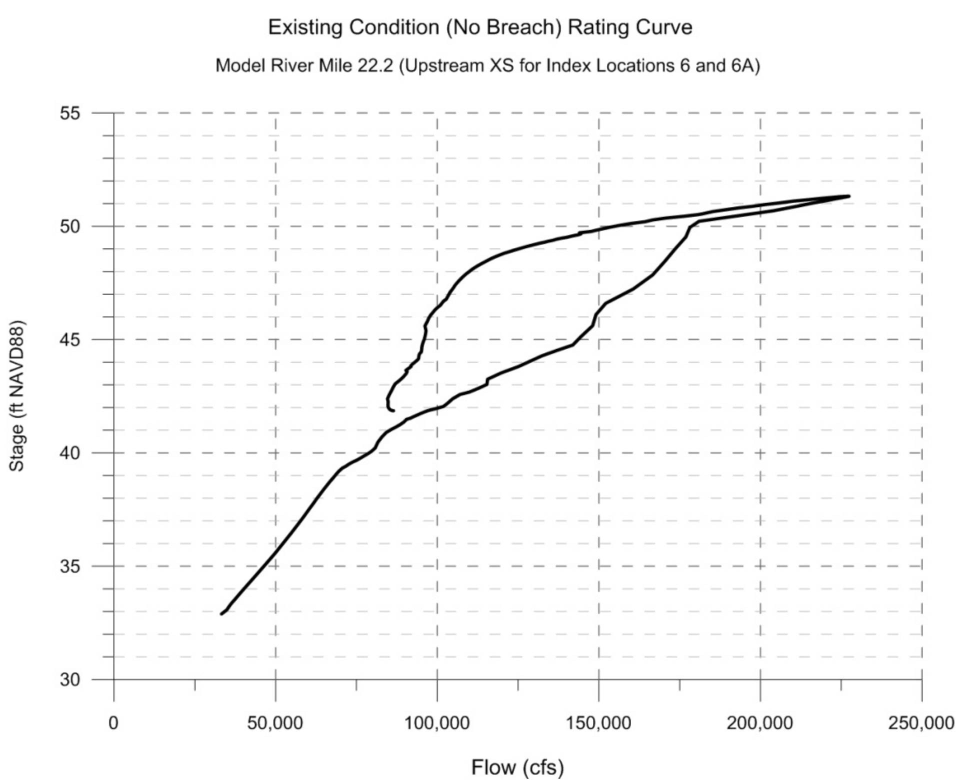
Existing Condition No Breach
Skagit River Mainstem River Mile 9.5 - 18 Water Surface Profiles
Left Bank
Left Bank 15% PNP
Right Bank
Right Bank 15% PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Index Locations
Confluence
Division Street
I-5 Bridge
Riverside Bridge
BNSF Bridge
4A
2, 2A, 7
5, 5A
Elevation (ft NAVD88)
Distance (River Mile)
River Mile based on 2012 HEC-RAS model


Existing Condition No Breach
South Fork Skagit River Water Surface Profiles
Left Bank
Left Bank 15% PNP
Right Bank
Right Bank 15% PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Index Locations
Fir Island Road
Confluence
4
Elevation (ft NAVD88)
40
35
30
25
20
15
10
5
1
2
3
4
5
6
7
8
9
Distance (River Mile)
River Mile based on 2012 HEC-RAS model


Existing Condition No Breach
North Fork Skagit River Water Surface Profiles
Left Bank
Left Bank 15% PNP
Right Bank
Right Bank 15% PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Index Locations
Best Road
Confluence
3
Elevation (ft NAVD88)
40
35
30
25
20
15
10
5
3
4
5
6
7
8
9
Distance (River Mile)
River Mile based on 2012 HEC-RAS model



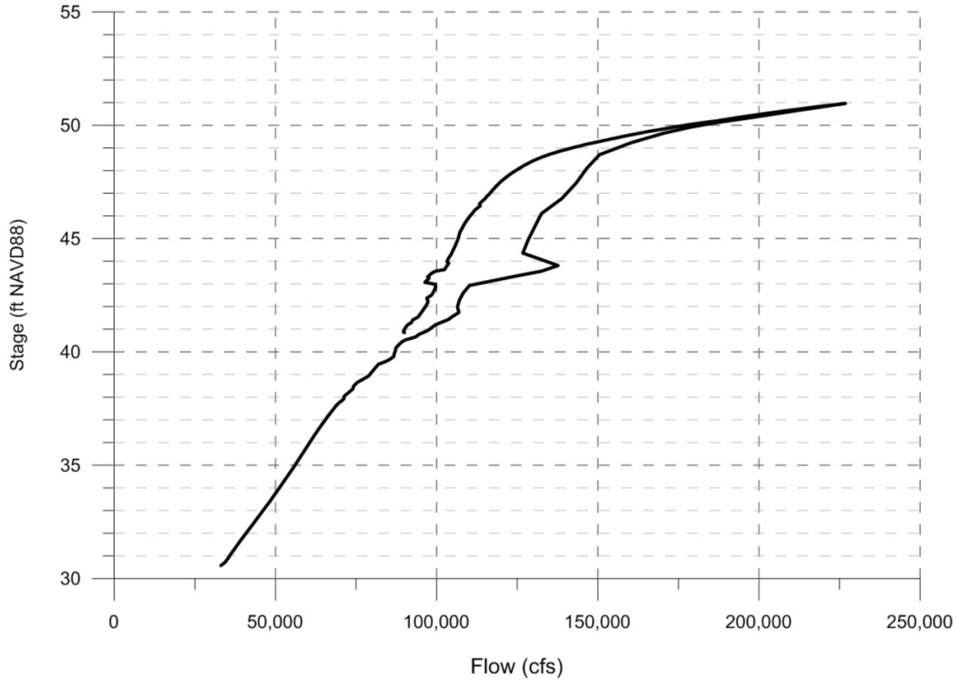
Existing Condition (No Breach) Rating Curve
Model River Mile 23.2 (Upstream XS for Index Location 8)
Stage (ft NAVD88)
60
55
50
45
40
35
0
50,000
100,000
150,000
200,000
250,000
300,000
350,000
Flow (cfs)



Existing Condition (No Breach) Rating Curve
Model River Mile 22.2 (Upstream XS for Index Locations 6 and 6A)
Stage (ft NAVD88)
55
50
45
40
35
30
0
50,000
100,000
150,000
200,000
250,000
Flow (cfs)

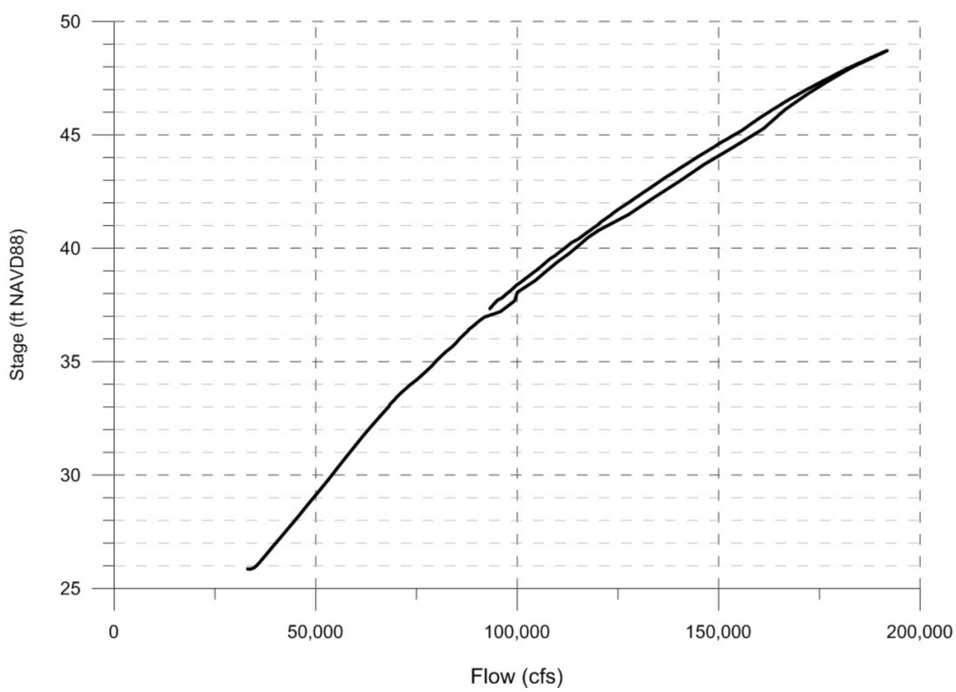
Existing Condition (No Breach) Rating Curve

Model River Mile 21.6 (Upstream XS for Index Location 1)



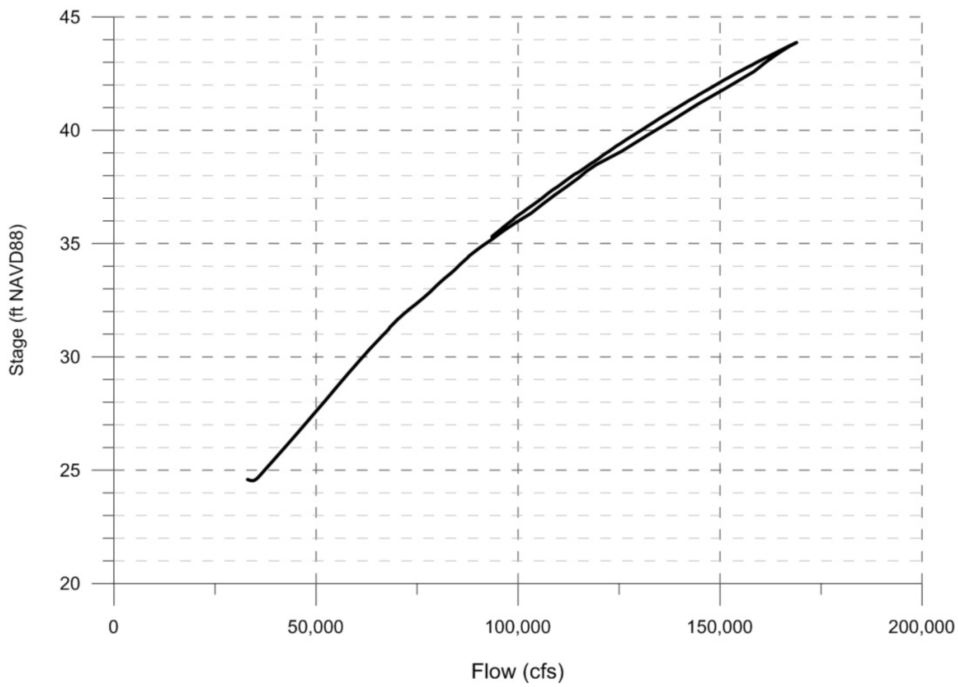
Existing Condition (No Breach) Rating Curve

Model River Mile 17.9 (Upstream XS for Index Location 1A)



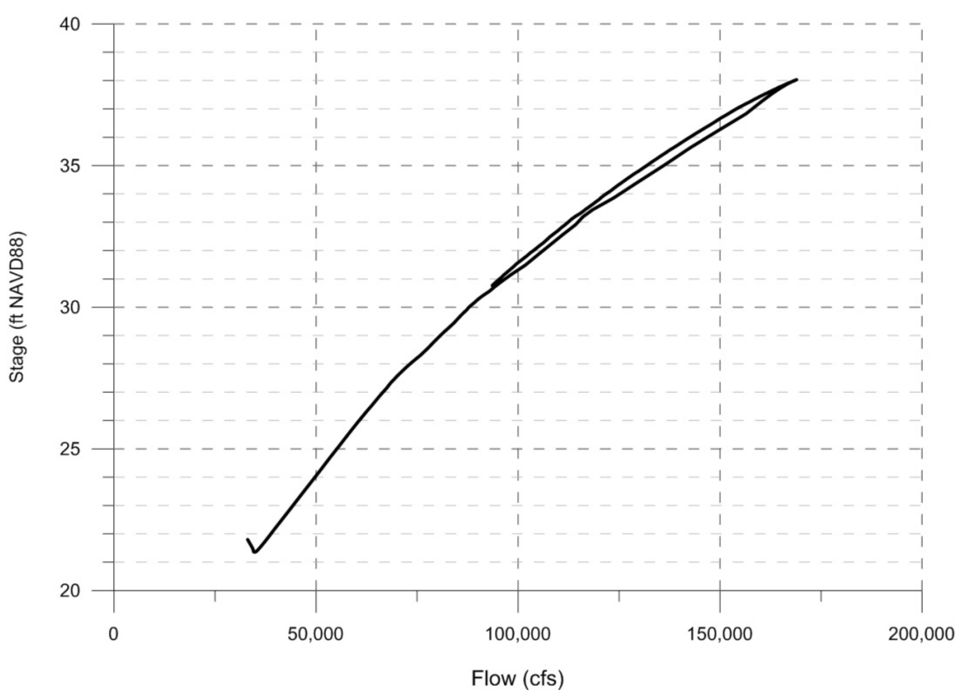
Existing Condition (No Breach) Rating Curve

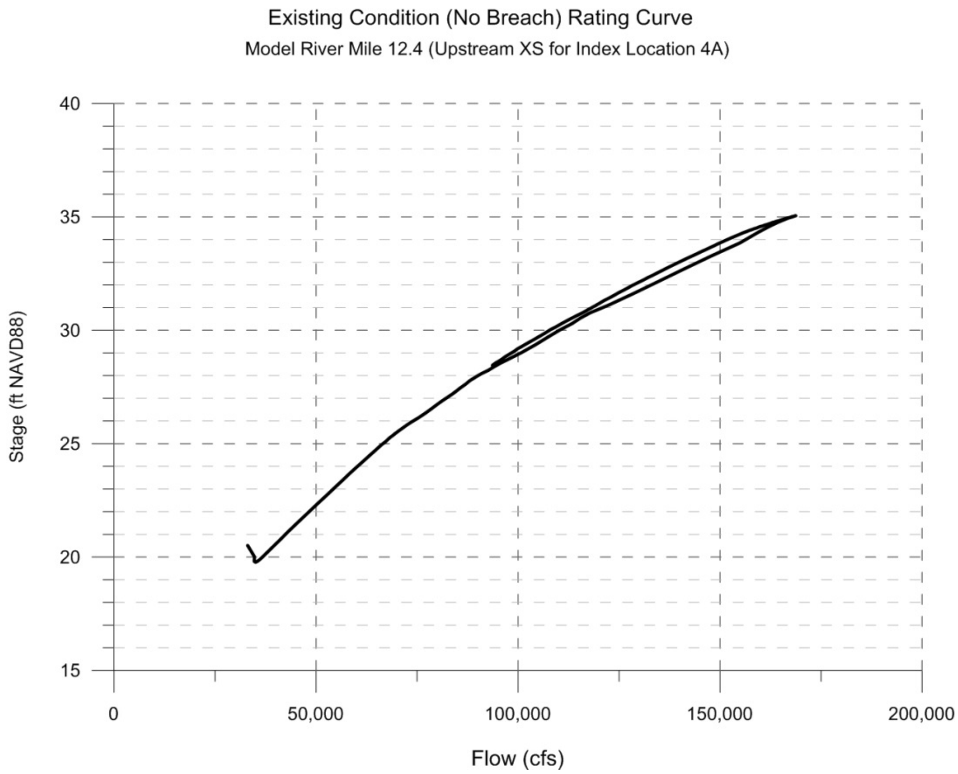
Model River Mile 16.78 (Upstream XS for Index Locations 5 and 5A)

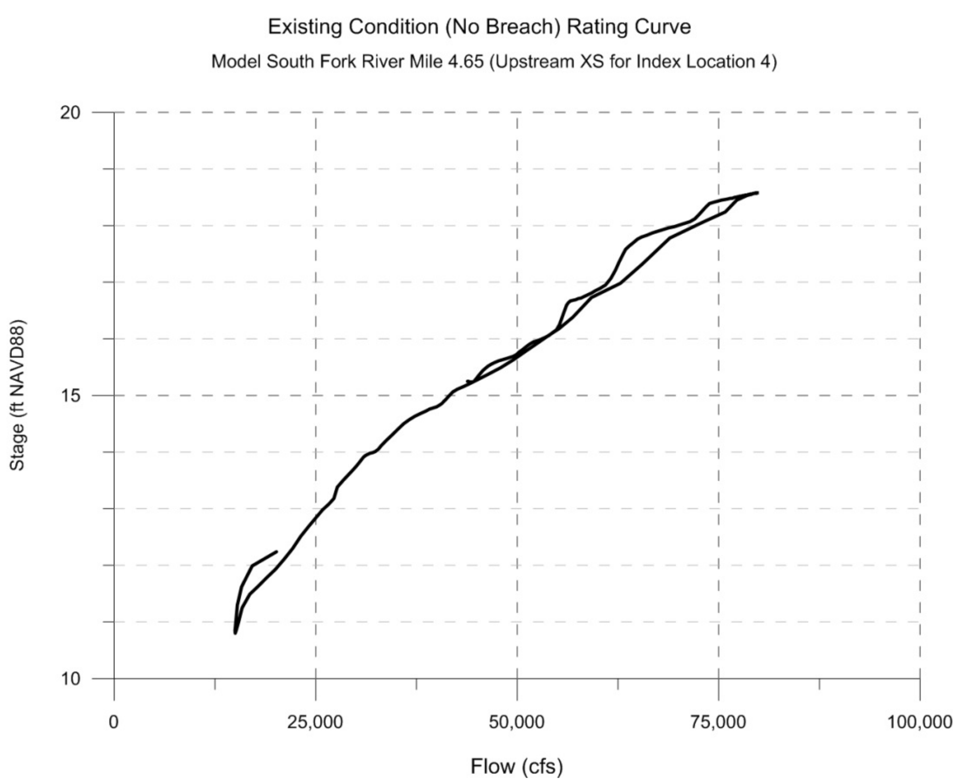


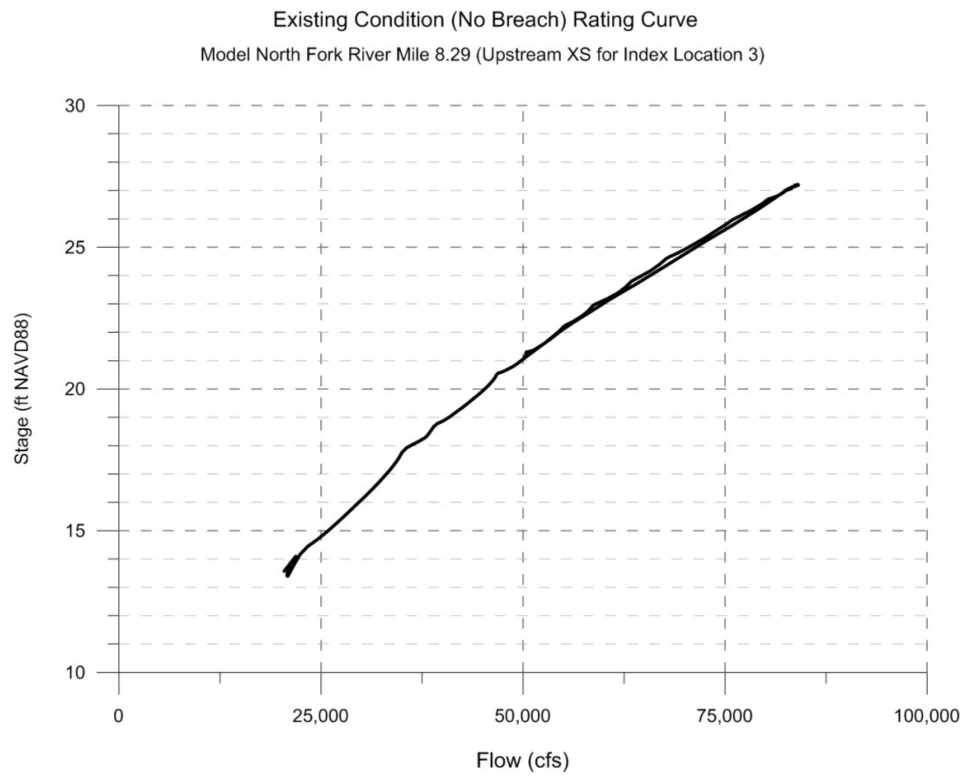
Existing Condition (No Breach) Rating Curve

Model River Mile 13.8 (Upstream XS for Index Locations 2, 2A, and 7)




Existing Condition (No Breach) Rating Curve
Model River Mile 12.4 (Upstream XS for Index Location 4A)
Stage (ft NAVD88)
40
35
30
25
20
15
0
50,000
100,000
150,000
200,000
Flow (cfs)


Existing Condition (No Breach) Rating Curve
Model South Fork River Mile 4.65 (Upstream XS for Index Location 4)
Stage (ft NAVD88)
20
15
10
0
25,000
50,000
75,000
100,000
Flow (cfs)


Existing Condition (No Breach) Rating Curve
Model North Fork River Mile 8.29 (Upstream XS for Index Location 3)
30
25
20
15
10
Stage (ft NAVD88)
0
25,000
50,000
75,000
100,000
Flow (cfs)

APPENDIX 5-2

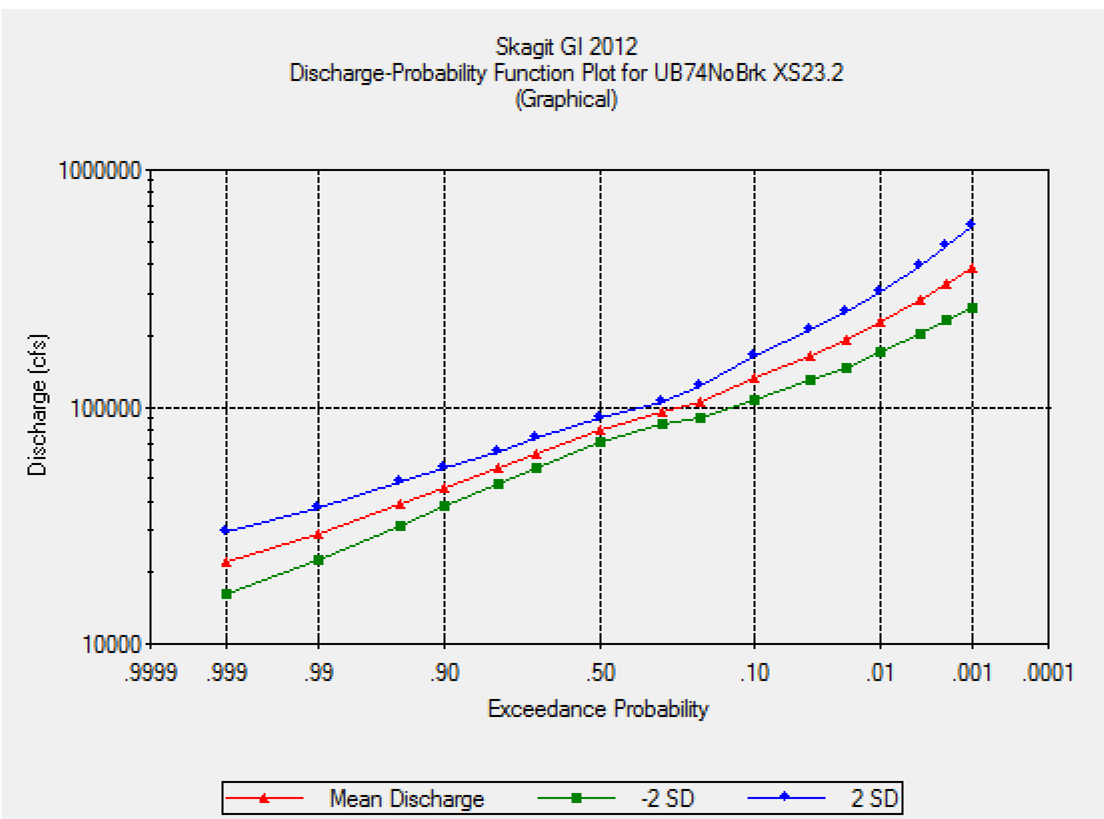
Additional Early Season Flood Regulation Storage
No-Breach Simulation Results

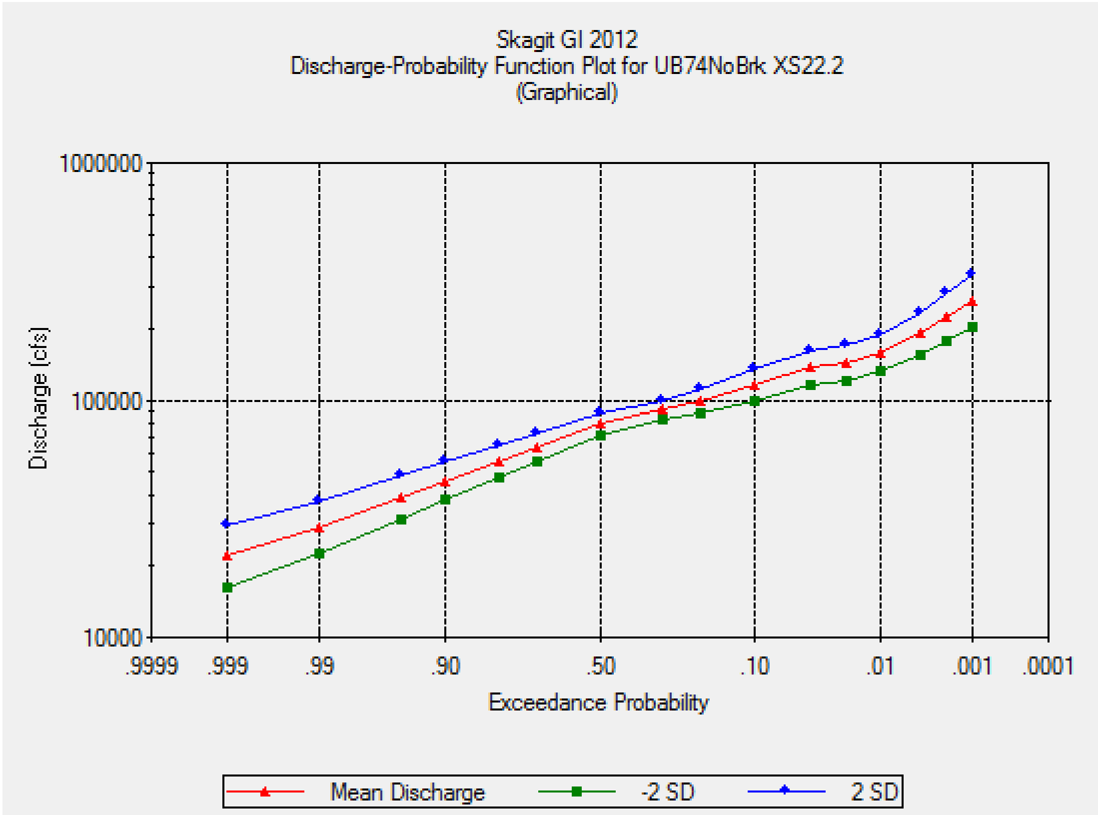
Additional Early Season Flood Regulation Storage No-Breach Simulation Results

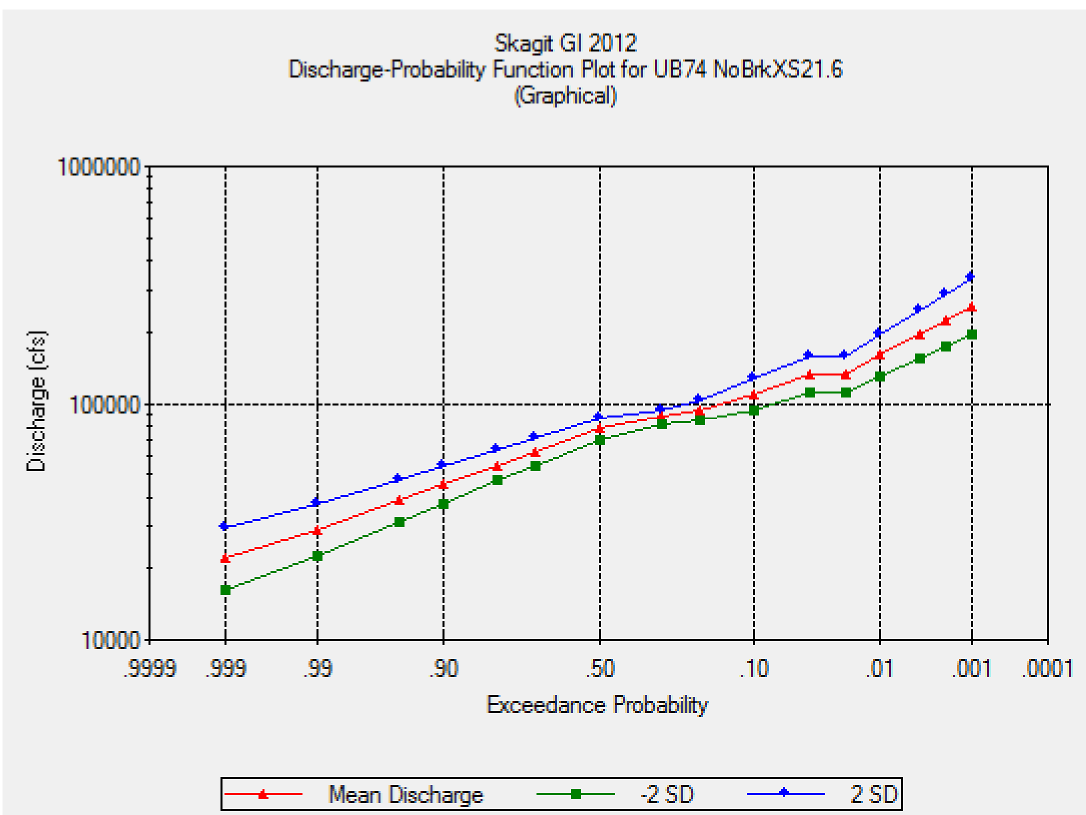
Flood quantiles (cfs), no breach, existing geometry, increased early season storage at Upper Baker

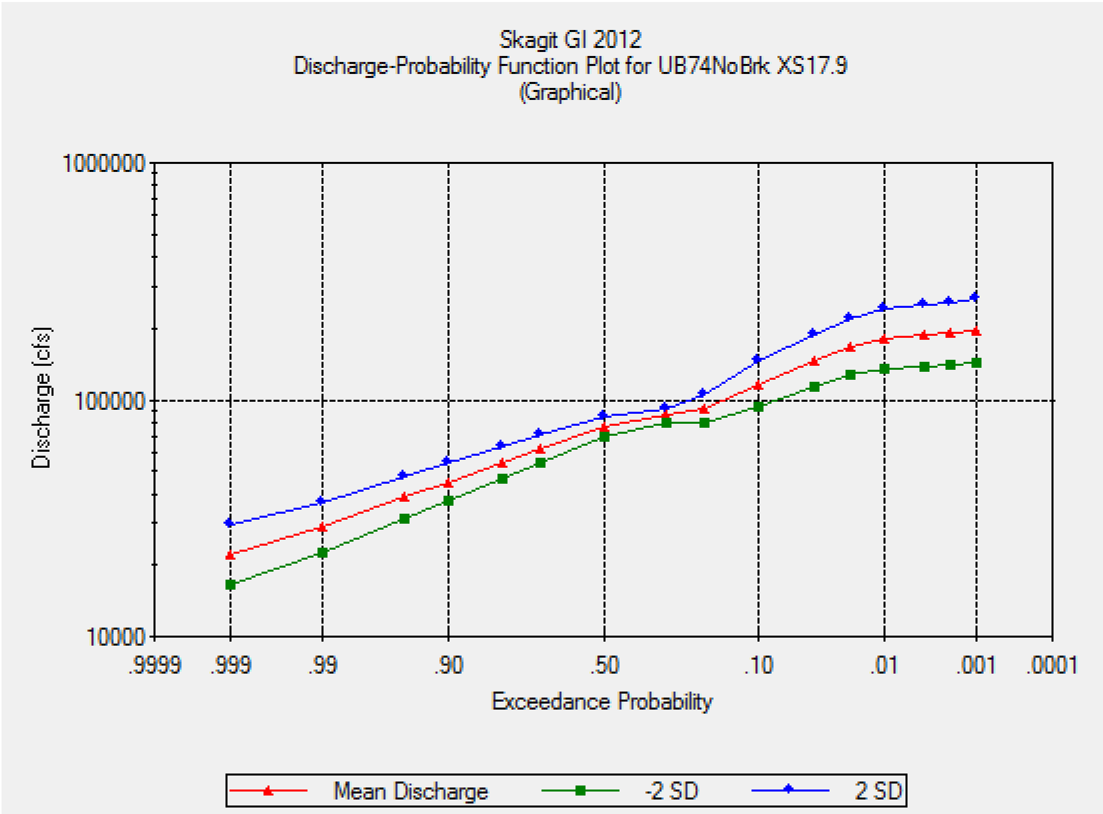
Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)								
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	80,400	79,500	77,900	77,100	77,000	76,900	76,900	34,900	41,700
5	105,100	99,000	93,600	91,300	91,200	91,200	91,200	42,400	48,900
10	131,800	115,900	109,600	116,400	116,300	116,300	116,300	55,400	60,800
25	165,400	137,200	132,100	145,900	145,800	145,700	145,700	70,400	75,100
50	191,900	143,100	132,700	167,700	162,200	162,100	162,100	78,000	82,200
75	213,500	152,200	147,900	176,200	165,100	165,100	165,100	78,800	83,000
100	229,100	157,900	159,800	179,500	166,000	166,000	166,000	79,100	83,300
250	282,500	190,200	194,500	186,600	167,800	167,800	167,700	79,500	83,800
500	333,200	224,500	224,400	191,400	168,900	168,900	168,700	79,700	84,000

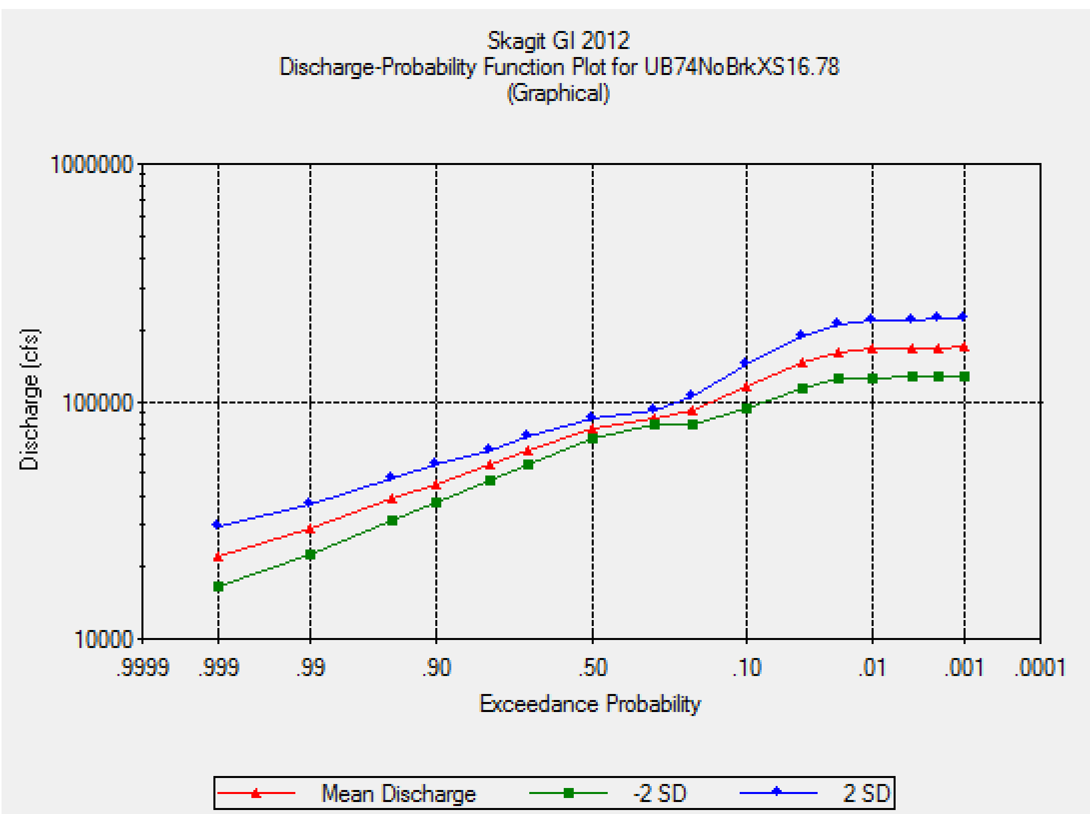
Indicates possible overestimation of flow due to questionable HEC-RAS model behavior - data should not be used without further evaluation

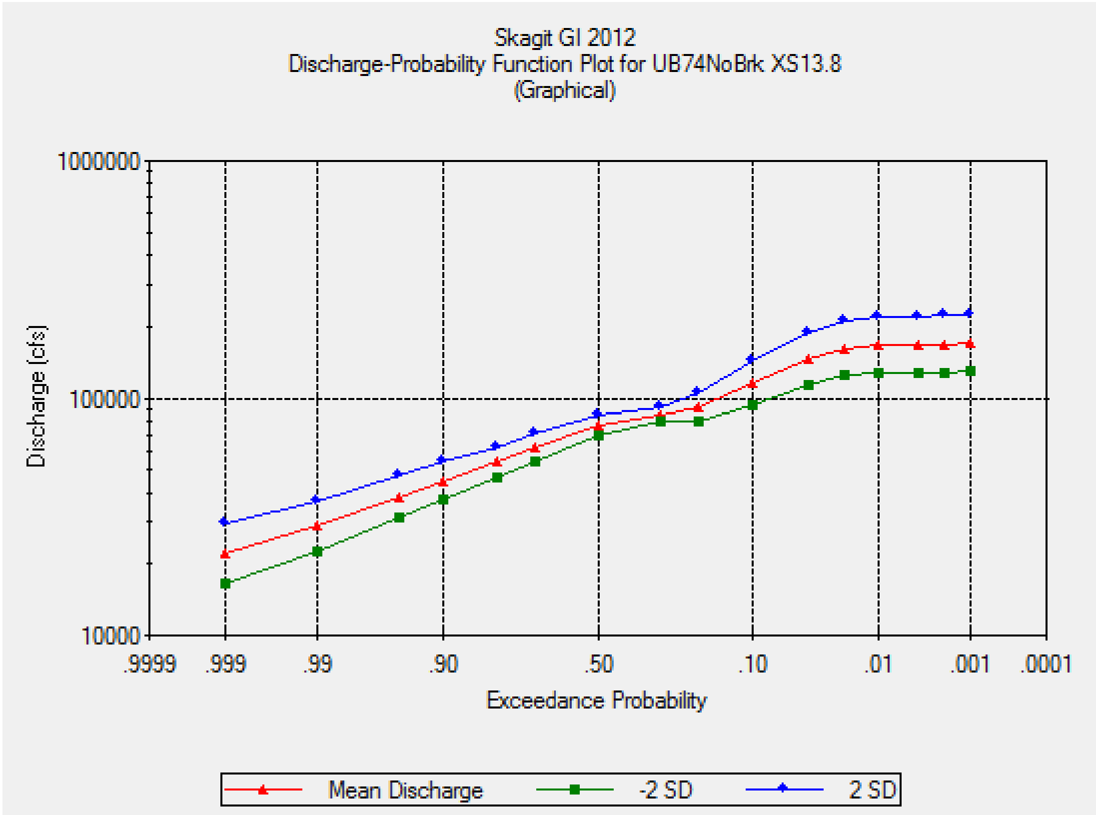


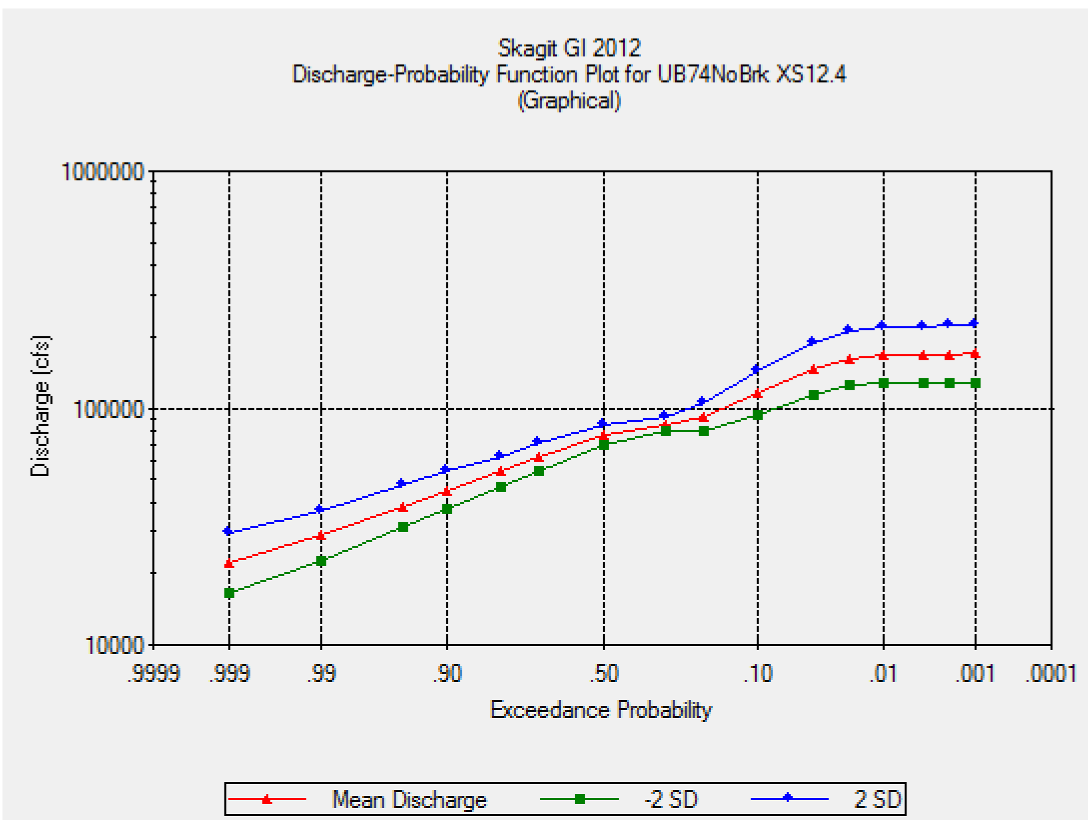

Skagit GI 2012
Discharge-Probability Function Plot for UB74NoBrk XS22.2
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

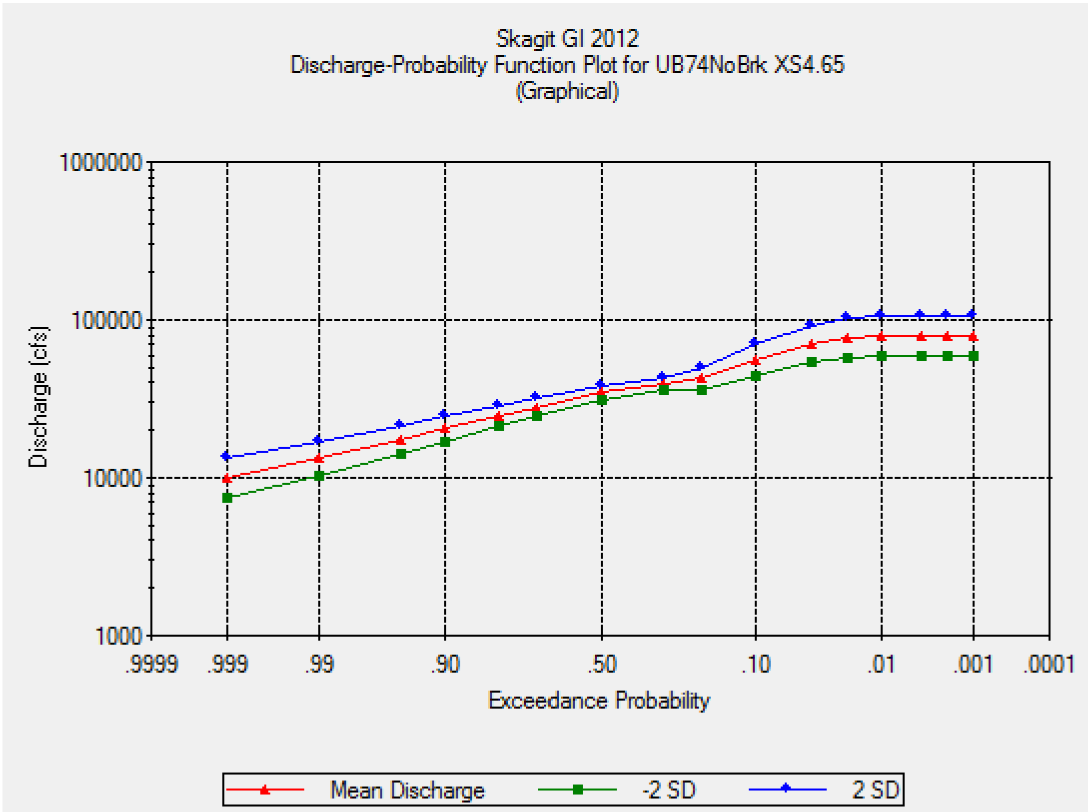

Skagit GI 2012
Discharge-Probability Function Plot for UB74 NoBrkXS21.6
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

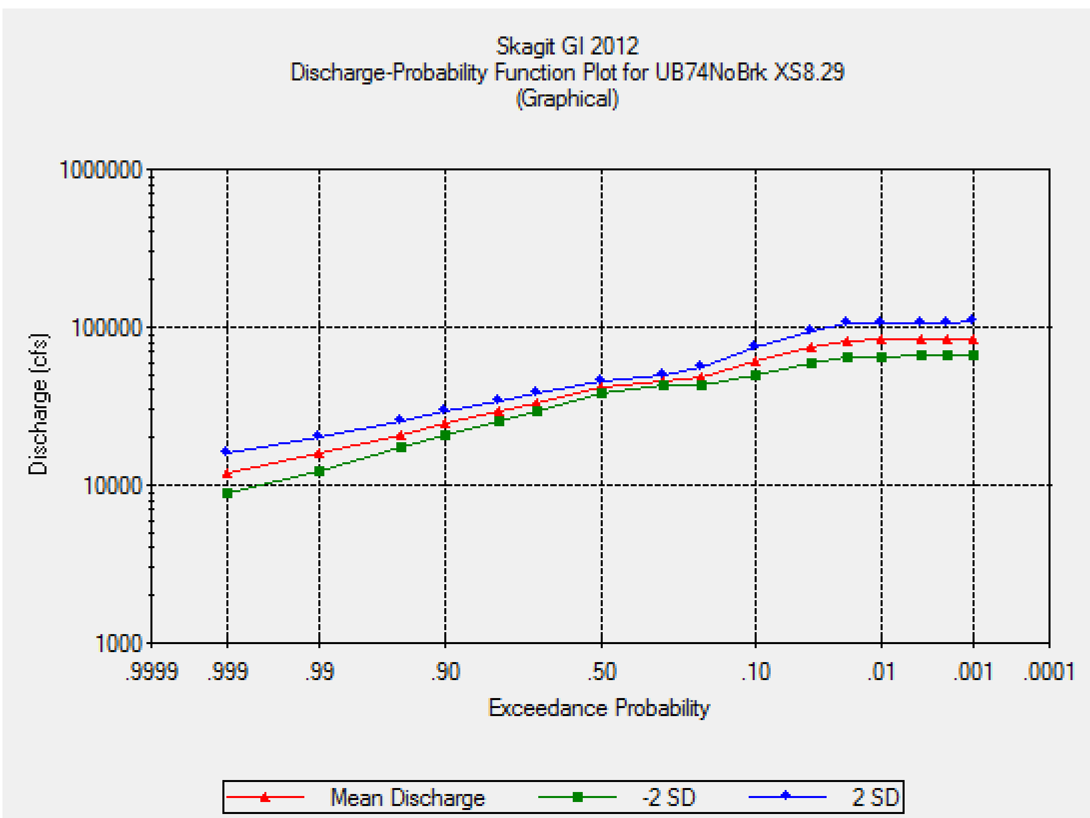

Skagit GI 2012
Discharge-Probability Function Plot for UB74NoBrk XS17.9
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

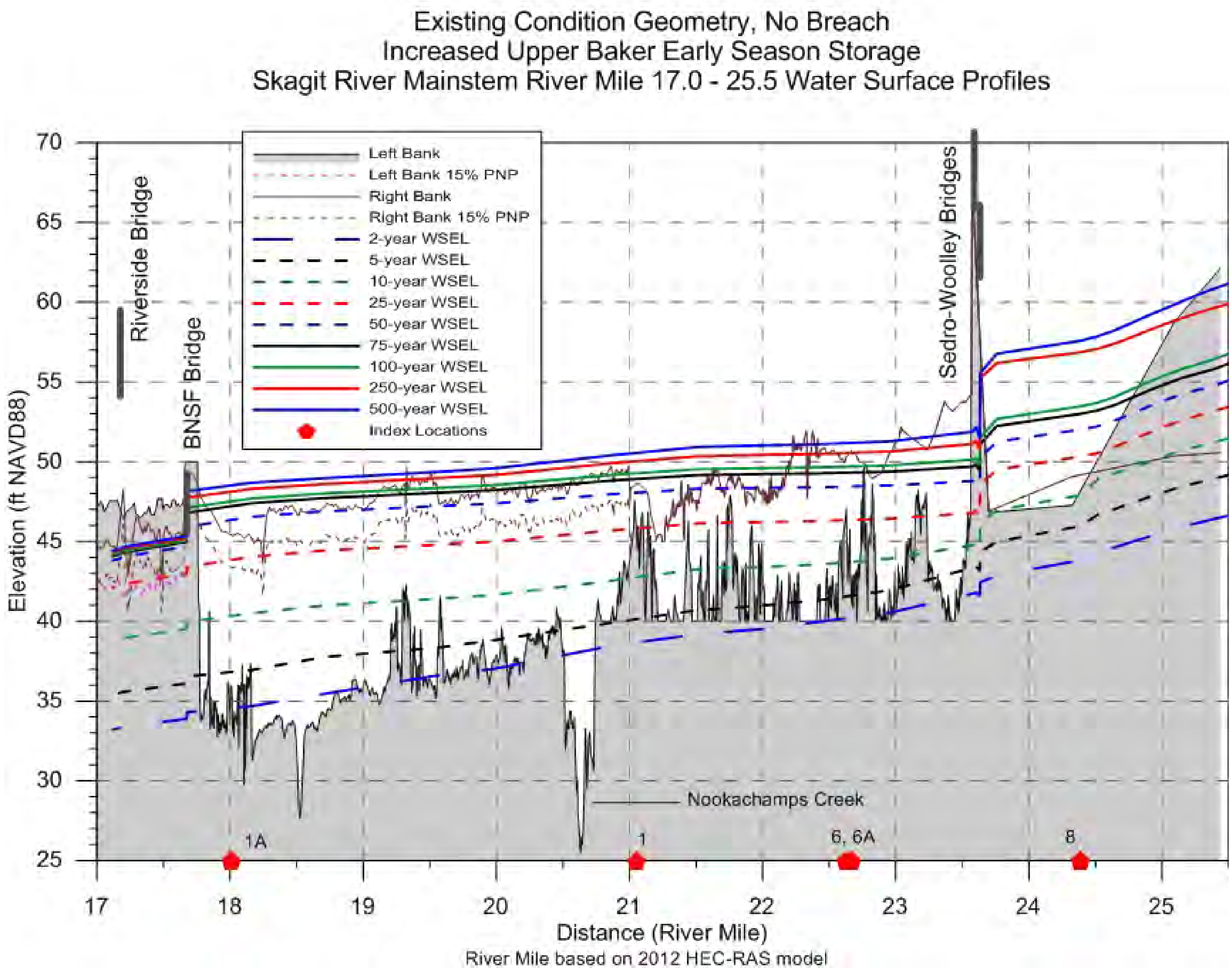

Skagit GI 2012
Discharge-Probability Function Plot for UB74NoBrkXS16.78
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

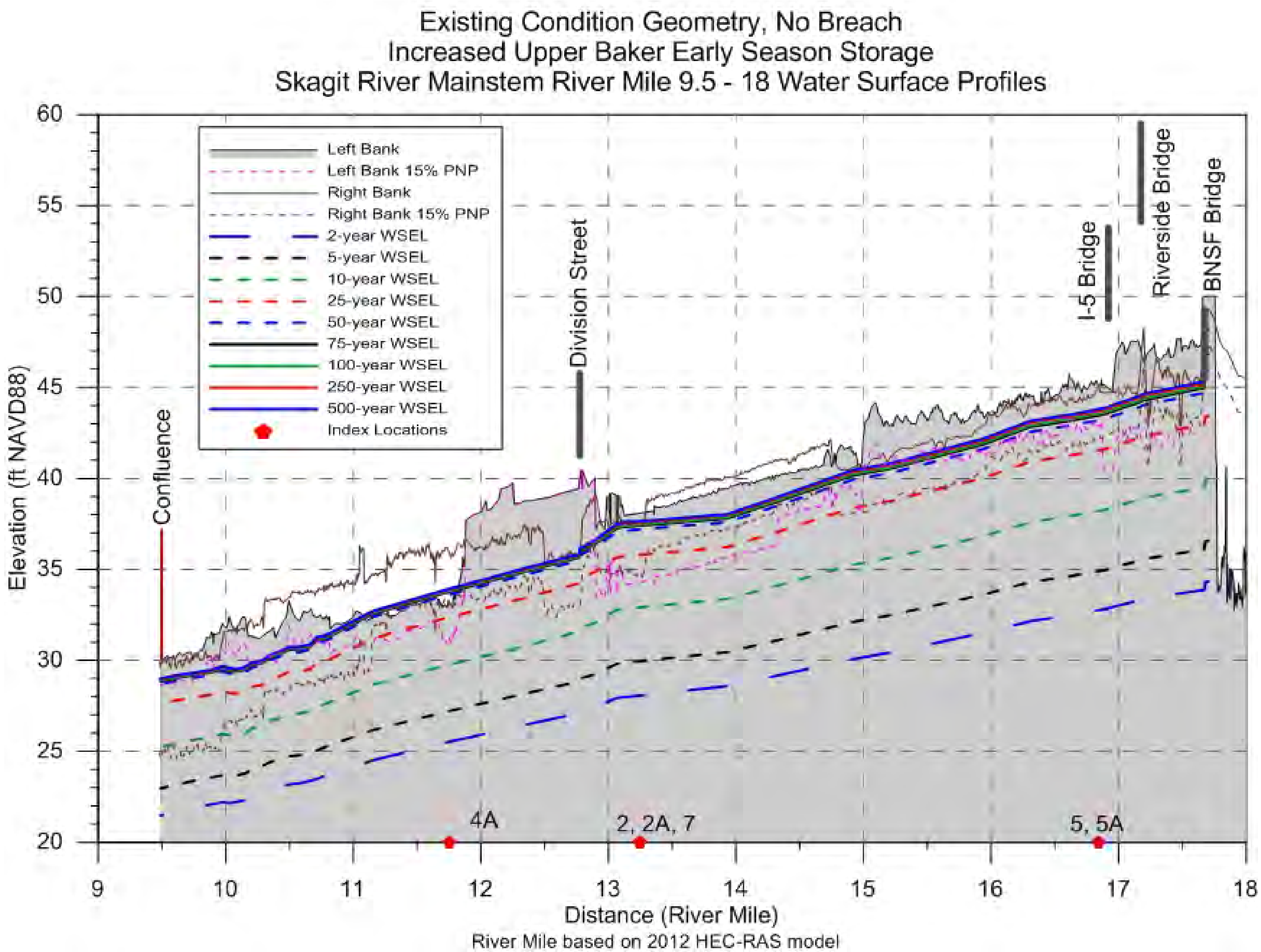

Skagit GI 2012
Discharge-Probability Function Plot for UB74NoBrk XS13.8
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

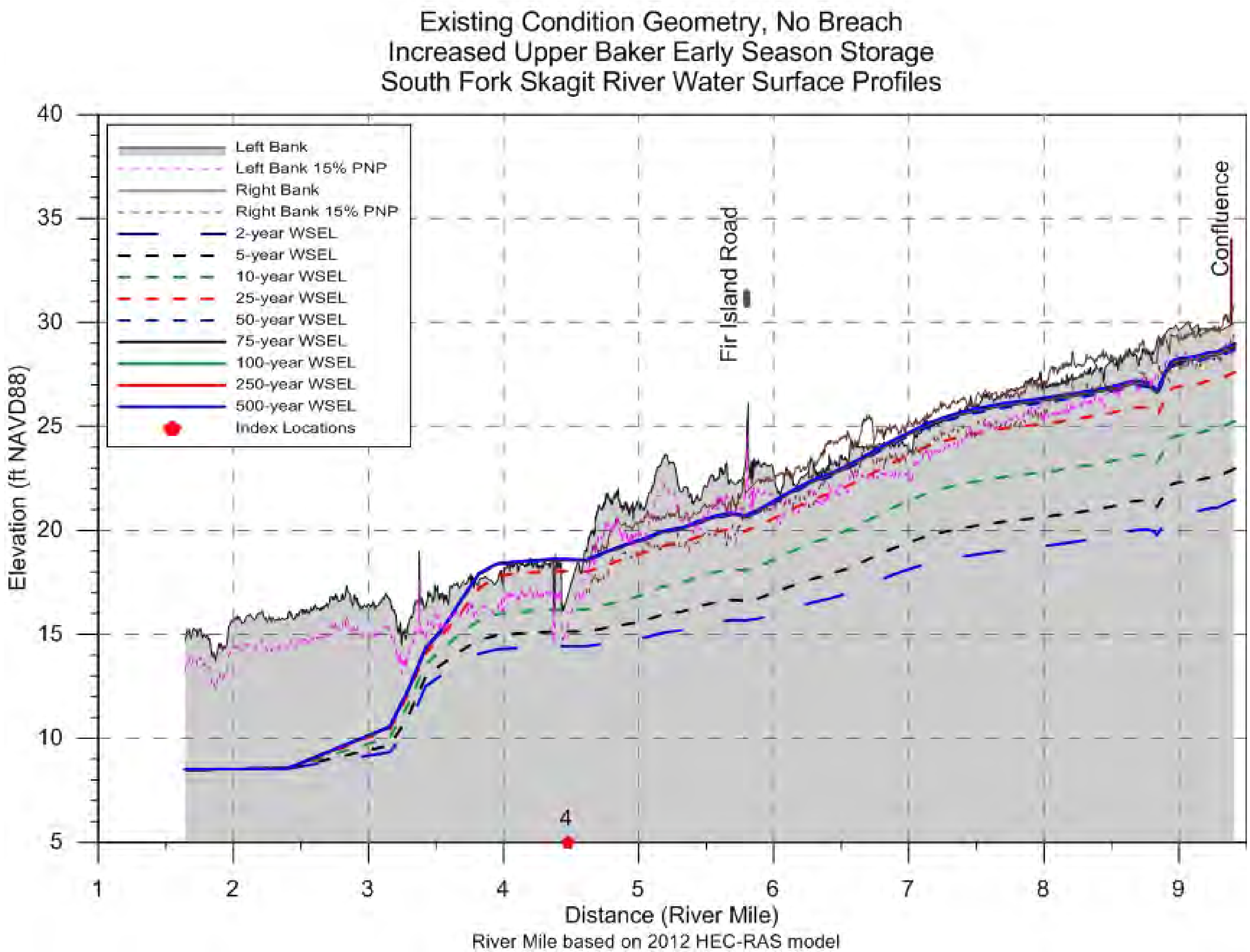

Skagit GI 2012
Discharge-Probability Function Plot for UB74NoBrk XS12.4
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

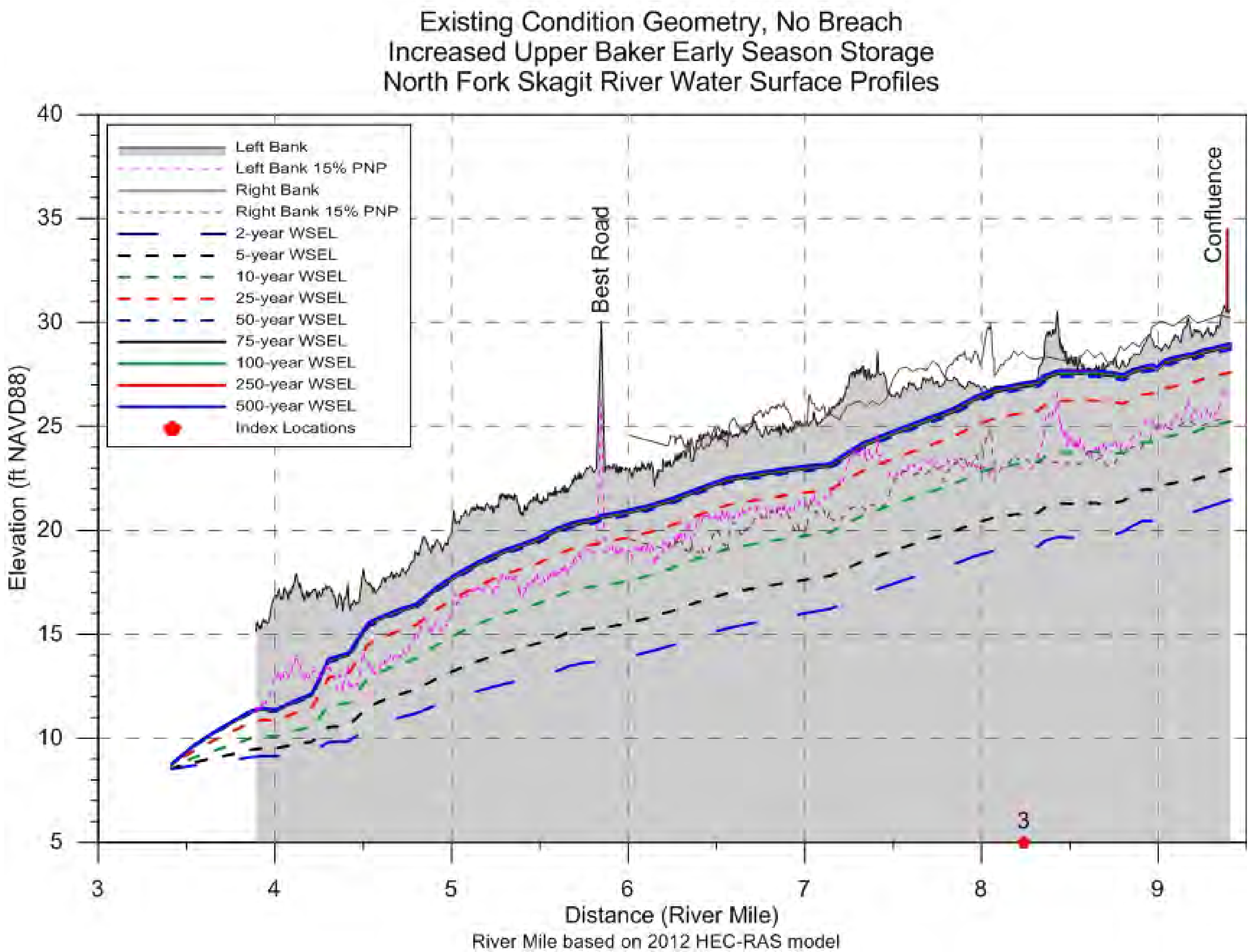

Skagit GI 2012
Discharge-Probability Function Plot for UB74NoBrk XS4.65
(Graphical)
1000000
100000
10000
1000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Skagit GI 2012
Discharge-Probability Function Plot for UB74NoBrk XS8.29
(Graphical)
1000000
100000
10000
1000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Existing Condition Geometry, No Breach
Increased Upper Baker Early Season Storage
Skagit River Mainstem River Mile 17.0 - 25.5 Water Surface Profiles
Left Bank
Left Bank 15% PNP
Right Bank
Right Bank 15% PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Index Locations
Riverside Bridge
BNSF Bridge
Sedro-Woolley Bridges
Nookachamps Creek
1A
1
6, 6A
8
Elevation (ft NAVD88)
70
65
60
55
50
45
40
35
30
25
17
18
19
20
21
22
23
24
25
Distance (River Mile)
River Mile based on 2012 HEC-RAS model


Existing Condition Geometry, No Breach
Increased Upper Baker Early Season Storage
Skagit River Mainstem River Mile 9.5 - 18 Water Surface Profiles
Left Bank
Left Bank 15% PNP
Right Bank
Right Bank 15% PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Index Locations
Confluence
Division Street
I-5 Bridge
Riverside Bridge
BNSF Bridge
4A
2, 2A, 7
5, 5A
Elevation (ft NAVD88)
Distance (River Mile)
River Mile based on 2012 HEC-RAS model


Existing Condition Geometry, No Breach
Increased Upper Baker Early Season Storage
South Fork Skagit River Water Surface Profiles
Left Bank
Left Bank 15% PNP
Right Bank
Right Bank 15% PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Index Locations
Fir Island Road
Confluence
Elevation (ft NAVD88)
40
35
30
25
20
15
10
5
4
1
2
3
4
5
6
7
8
9
Distance (River Mile)
River Mile based on 2012 HEC-RAS model


Existing Condition Geometry, No Breach
Increased Upper Baker Early Season Storage
North Fork Skagit River Water Surface Profiles
Left Bank
Left Bank 15% PNP
Right Bank
Right Bank 15% PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Index Locations
Best Road
Confluence
3
Elevation (ft NAVD88)
40
35
30
25
20
15
10
5
3
4
5
6
7
8
9
Distance (River Mile)
River Mile based on 2012 HEC-RAS model

APPENDIX 5-3

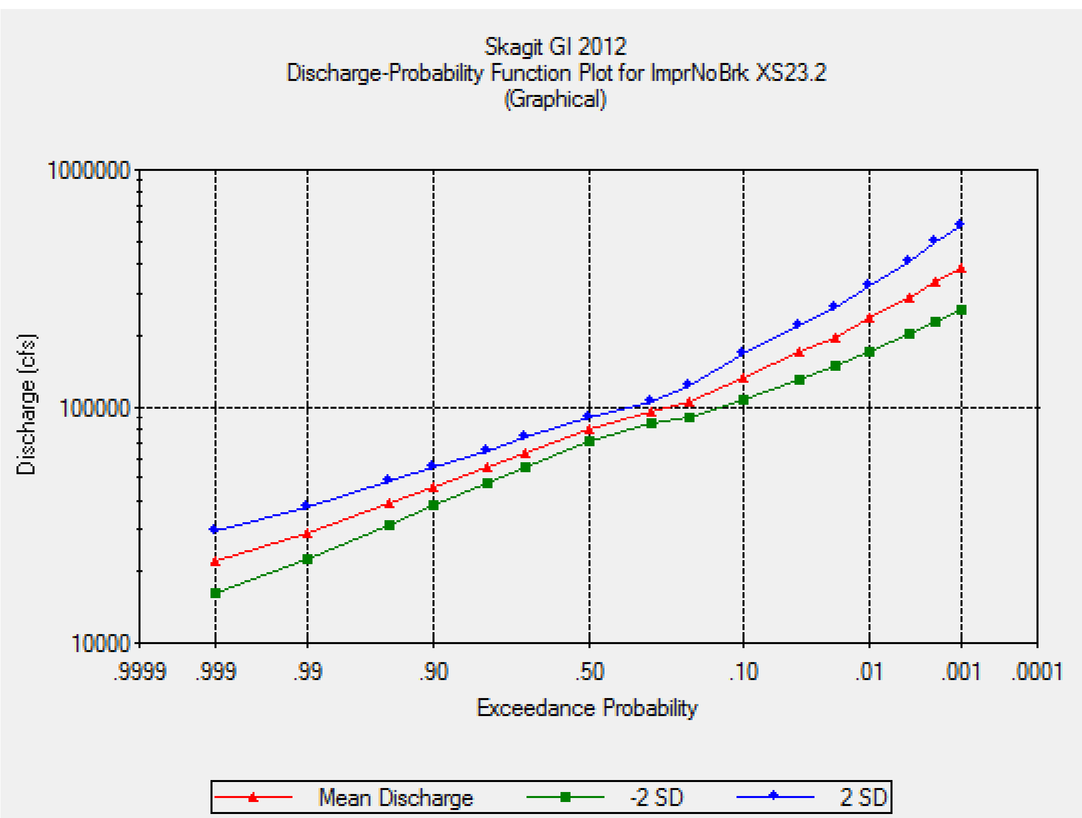
Improved Levee No-Breach Simulation Results

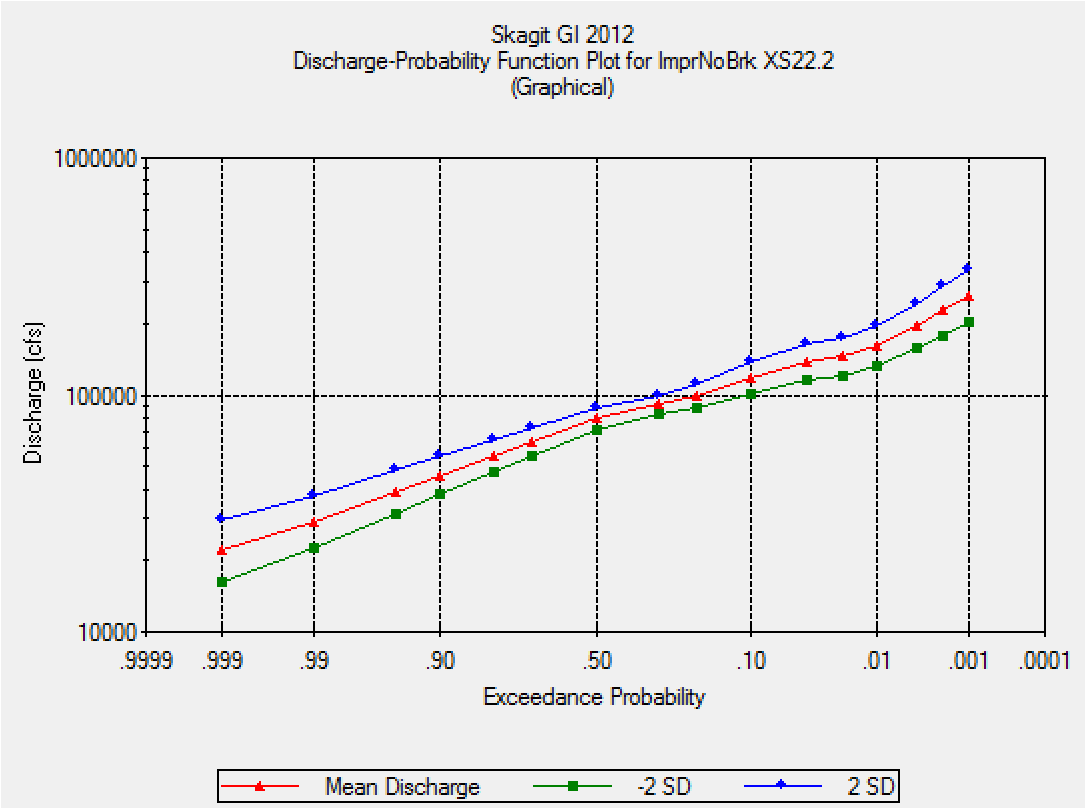
Improved Levee No-Breach Simulation Results

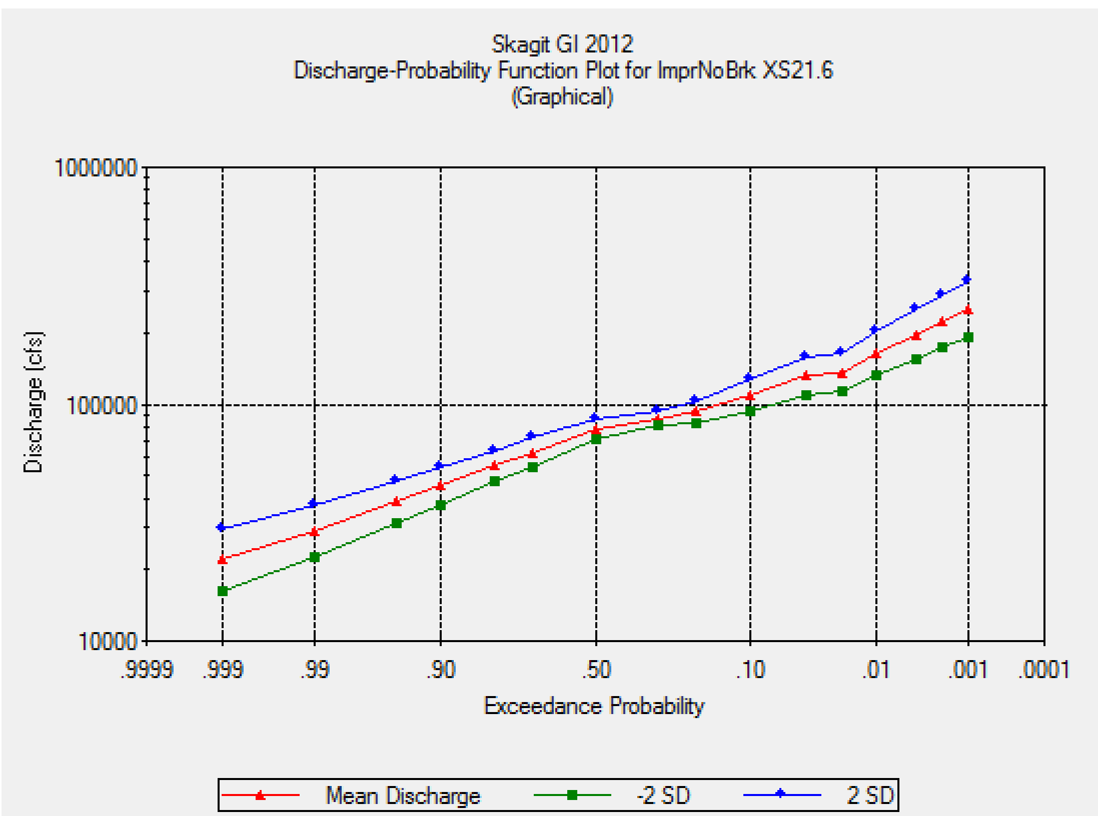
Flood quantiles (cfs), no breach, improved levees, existing regulation

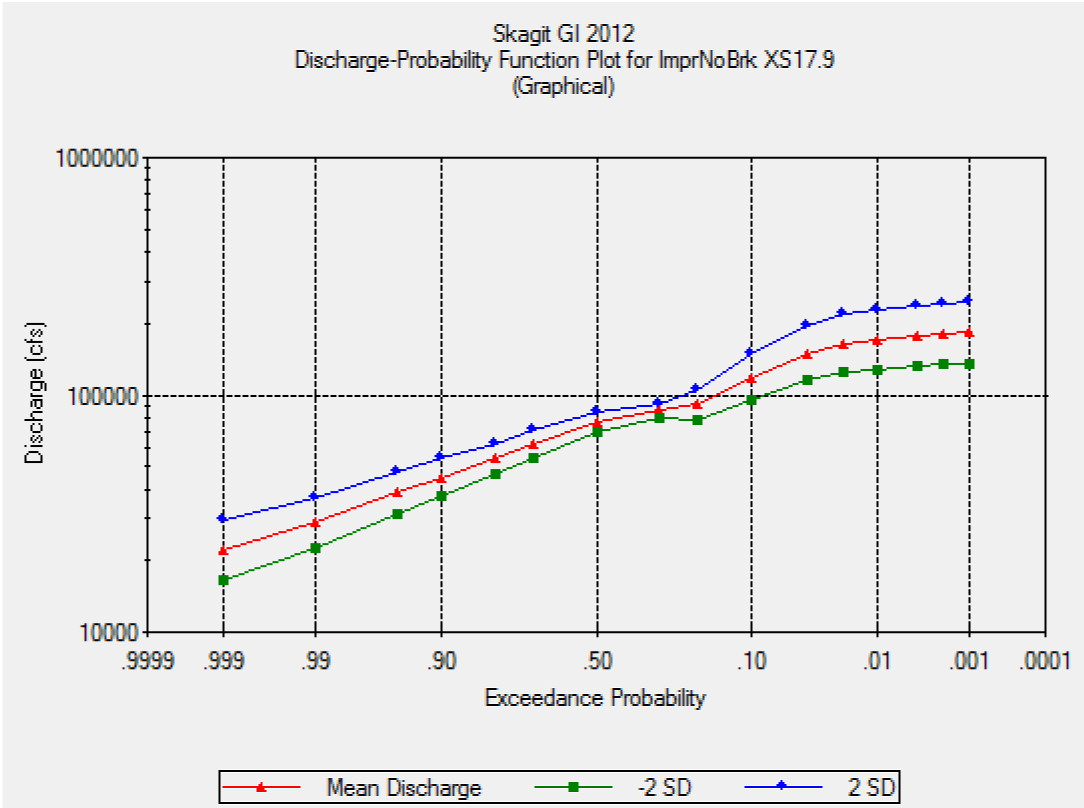
Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)								
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	80,400	79,300	78,500	77,000	77,000	76,900	76,800	34,900	41,700
5	105,200	99,000	92,600	91,300	91,300	91,300	91,300	42,400	48,900
10	132,700	118,000	109,600	118,500	118,400	118,400	118,400	56,400	61,800
25	169,800	138,900	131,700	149,400	149,300	149,300	149,200	72,000	76,900
50	197,400	144,700	135,400	164,900	163,800	163,800	163,800	78,100	82,700
75	220,000	156,500	151,200	169,500	165,600	165,600	165,600	78,600	83,200
100	235,700	160,600	163,400	171,700	166,300	166,300	166,300	78,800	83,400
250	289,900	194,800	197,100	176,800	167,700	167,700	167,700	79,200	83,800
500	337,500	226,700	224,200	180,400	168,600	168,600	168,600	79,300	84,100

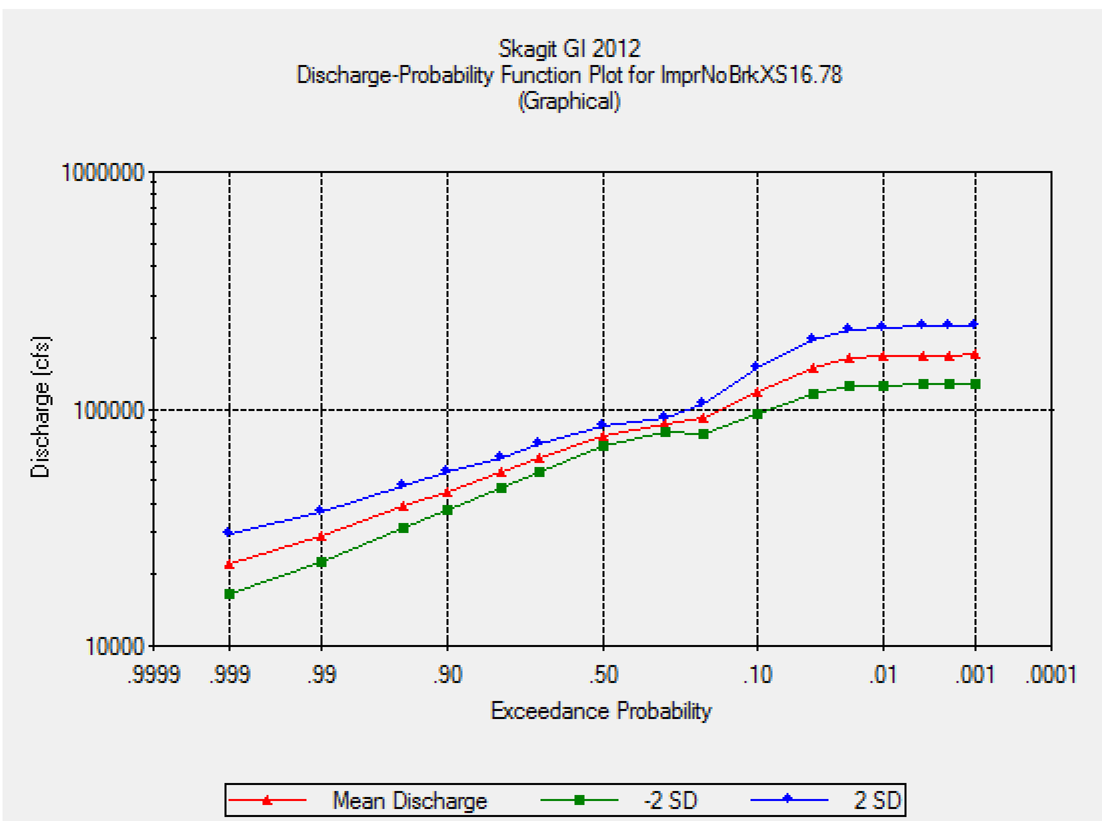
Indicates possible overestimation of flow due to questionable HEC-RAS model behavior - data should not be used without further evaluation

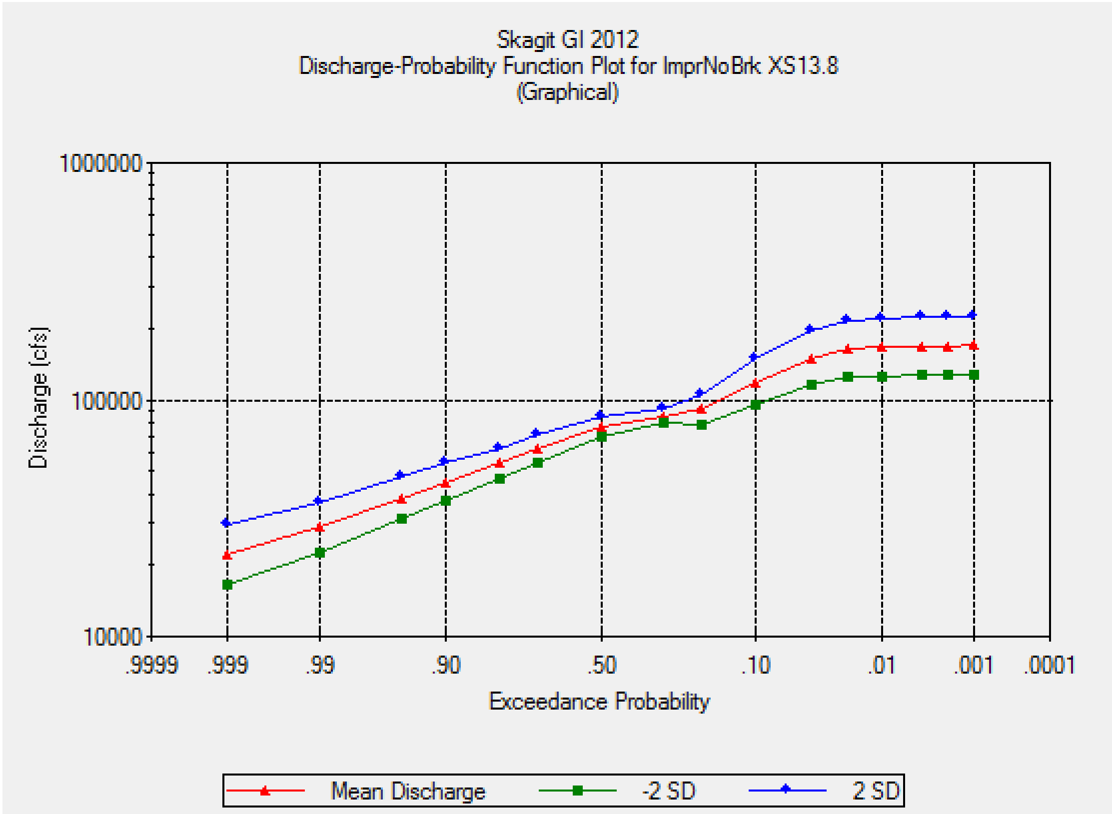


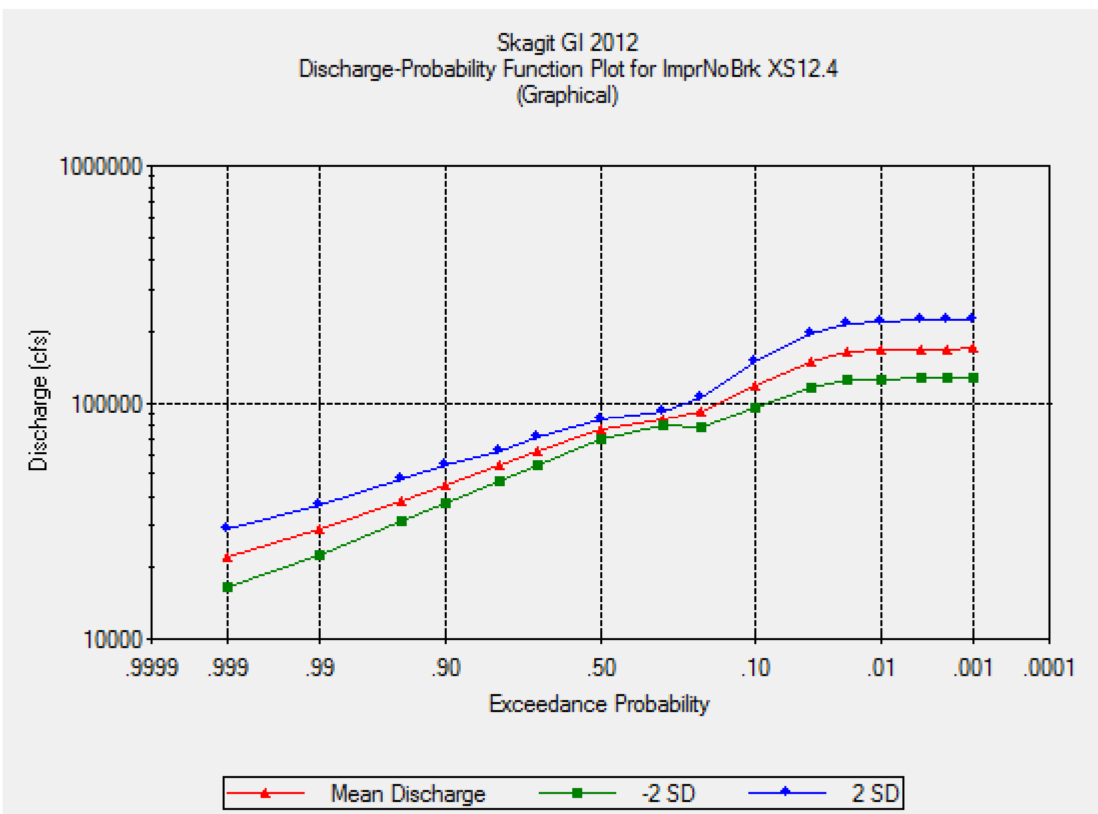

Skagit GI 2012
Discharge-Probability Function Plot for ImprNoBrk XS22.2
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

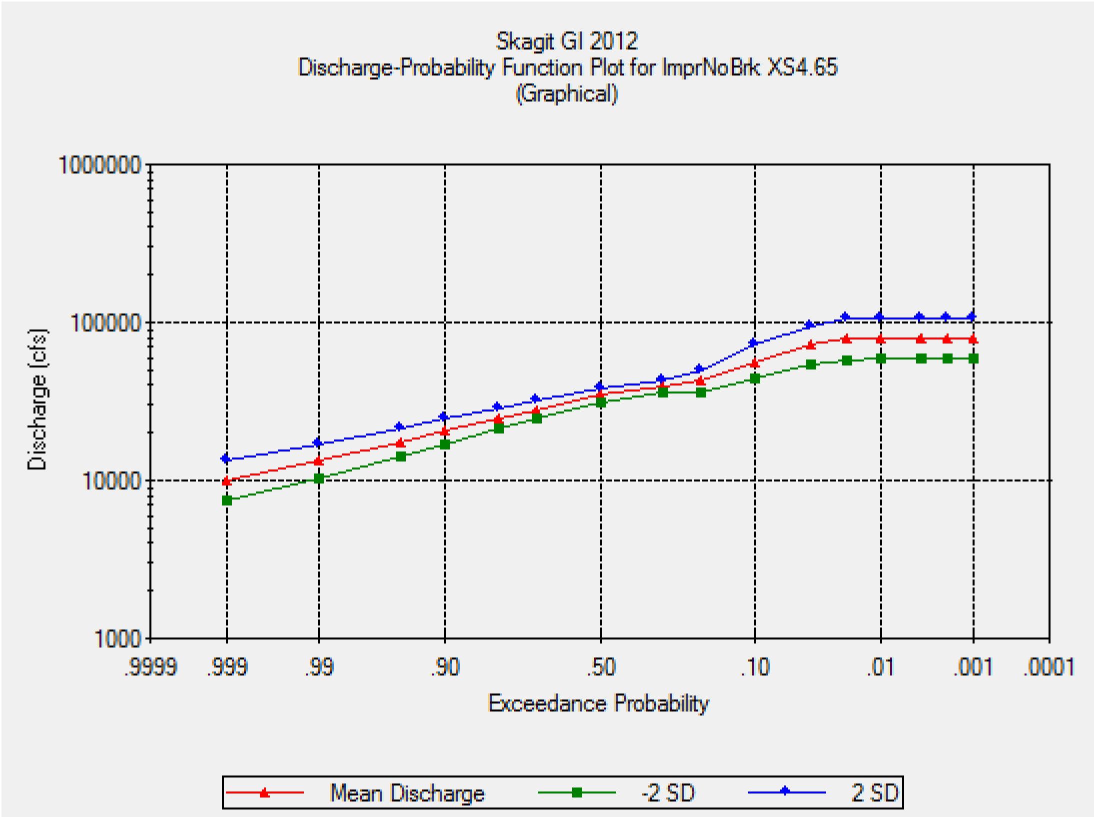

Skagit GI 2012
Discharge-Probability Function Plot for ImprNoBrk XS21.6
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

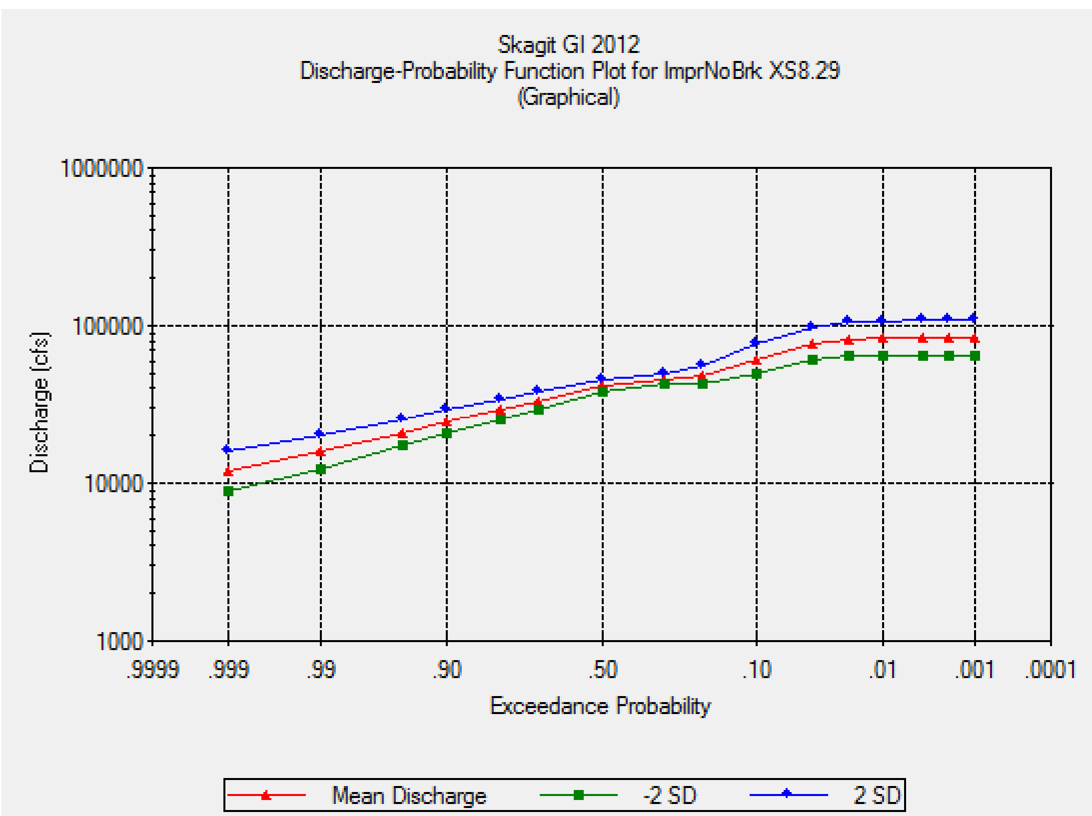

Skagit GI 2012
Discharge-Probability Function Plot for ImprNoBrk XS17.9
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

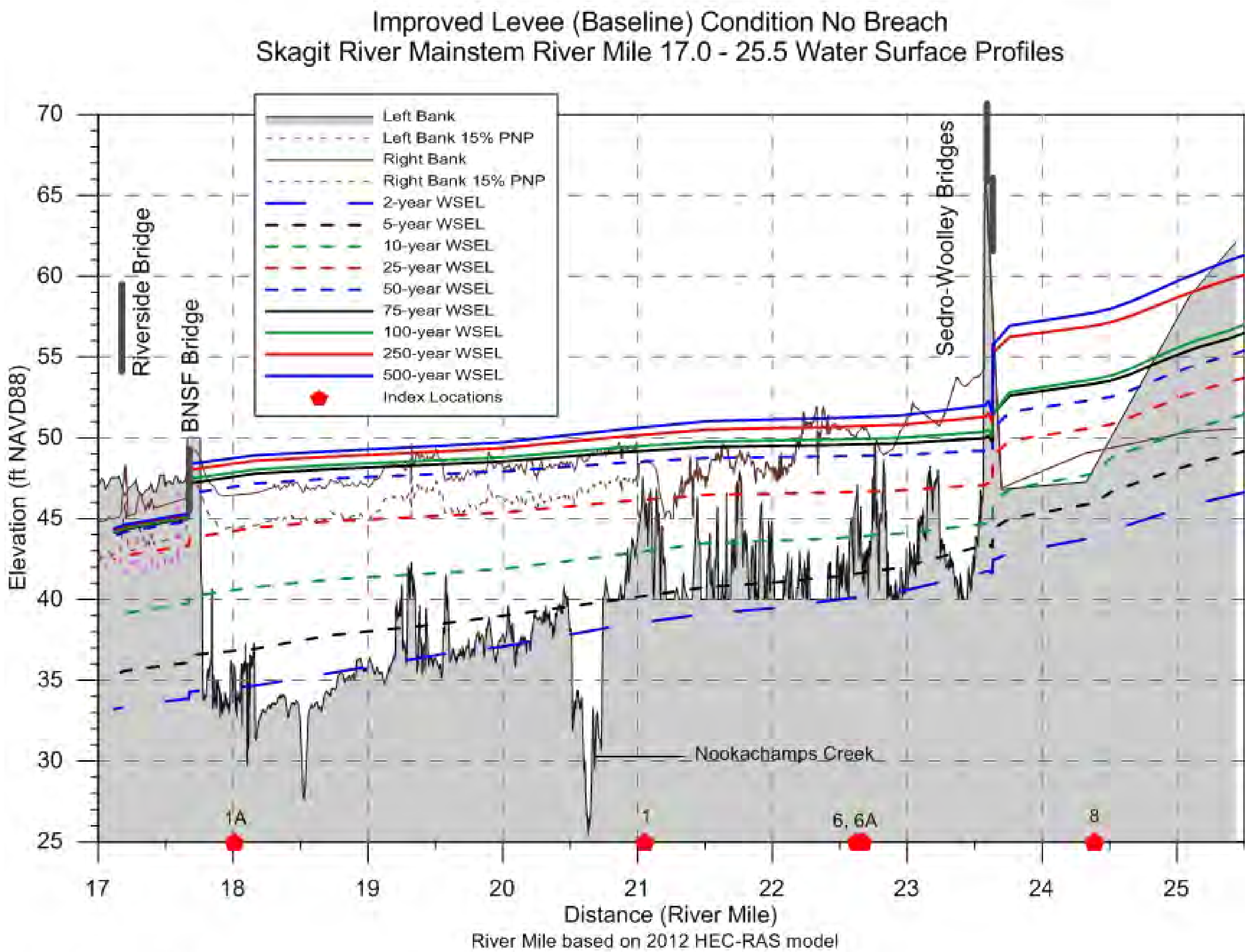

Skagit GI 2012
Discharge-Probability Function Plot for ImprNoBrkXS16.78
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Skagit GI 2012
Discharge-Probability Function Plot for ImprNoBrk XS13.8
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

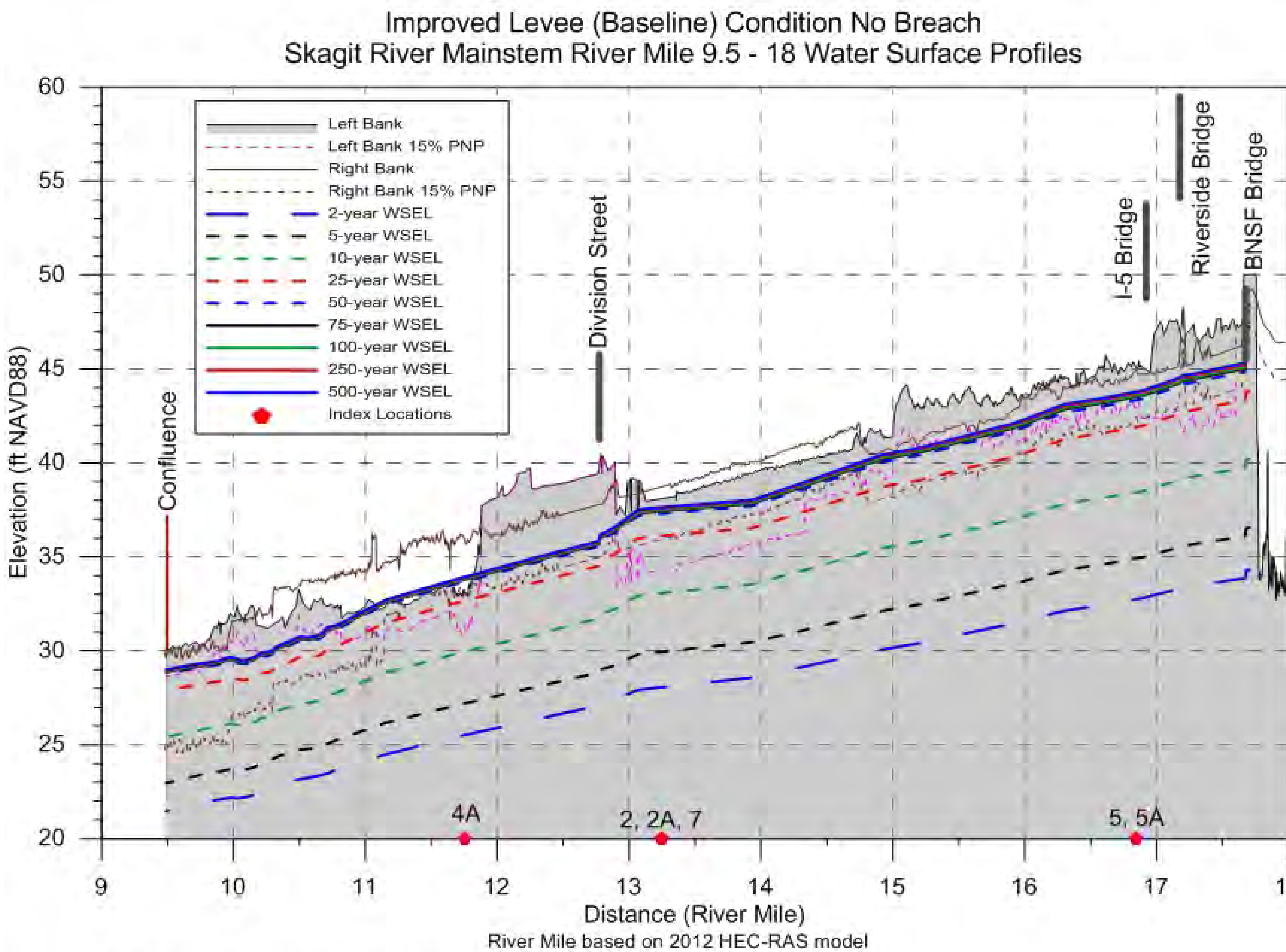

Skagit GI 2012
Discharge-Probability Function Plot for ImprNoBrk XS12.4
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Skagit GI 2012
Discharge-Probability Function Plot for ImprNoBrk XS4.65
(Graphical)
1000000
100000
10000
1000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

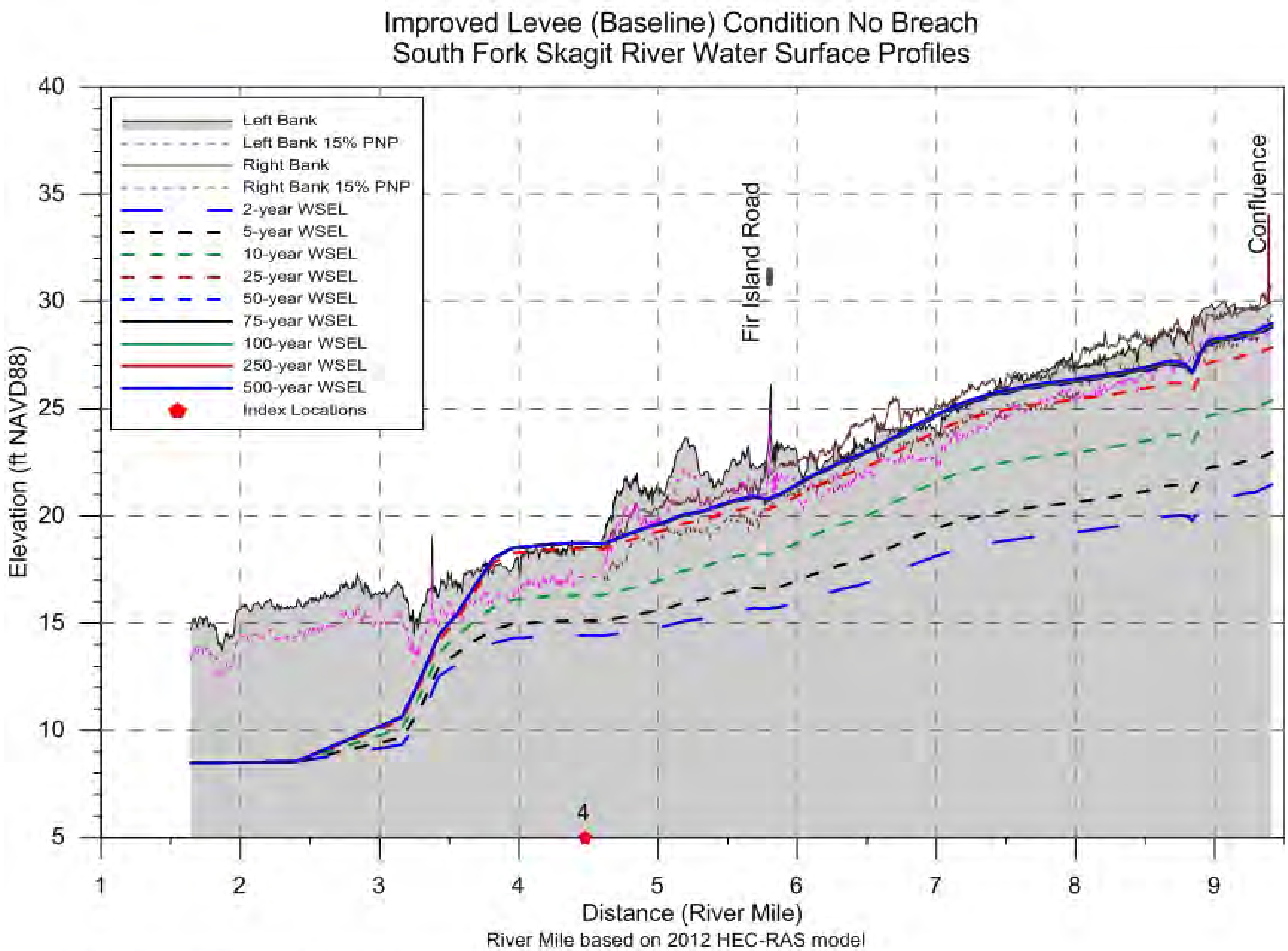

Skagit GI 2012
Discharge-Probability Function Plot for ImprNoBrk XS8.29
(Graphical)
1000000
100000
10000
1000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD



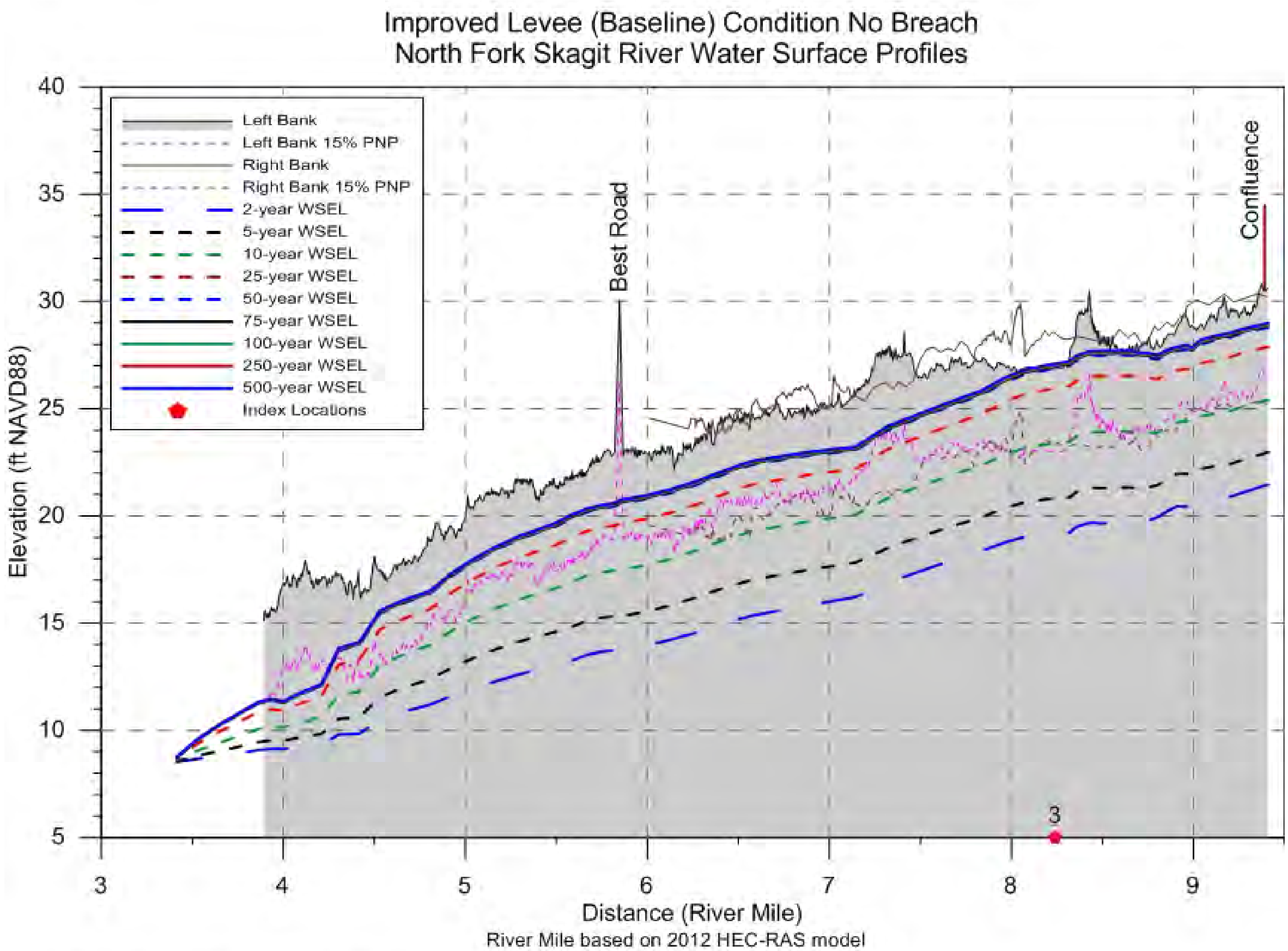
Note: Bank profiles and PNPs reflect improved levee ("baseline") condition



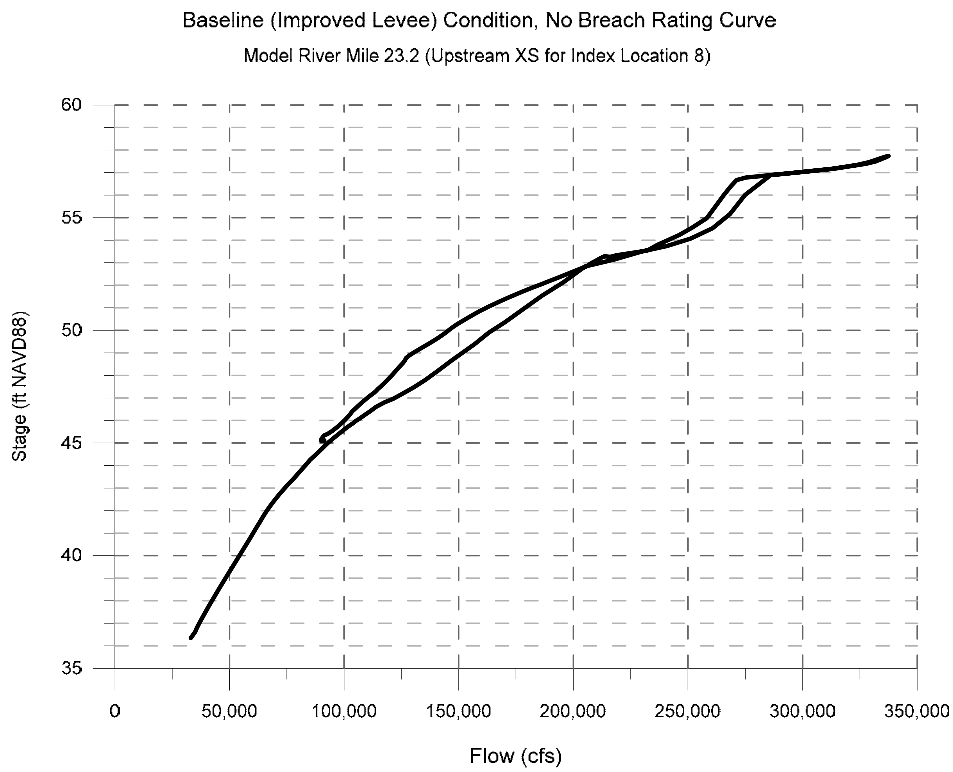
Note: Bank profiles and PNPs reflect improved levee ("baseline") condition

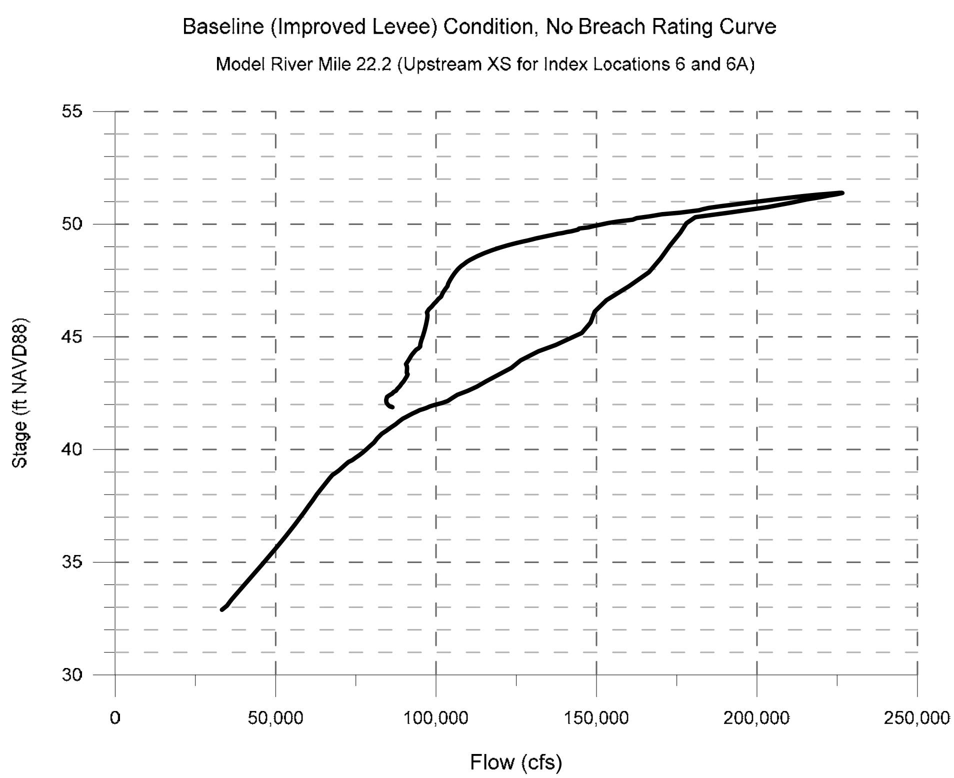


Note: Bank profiles and PNPs reflect improved levee ("baseline") condition



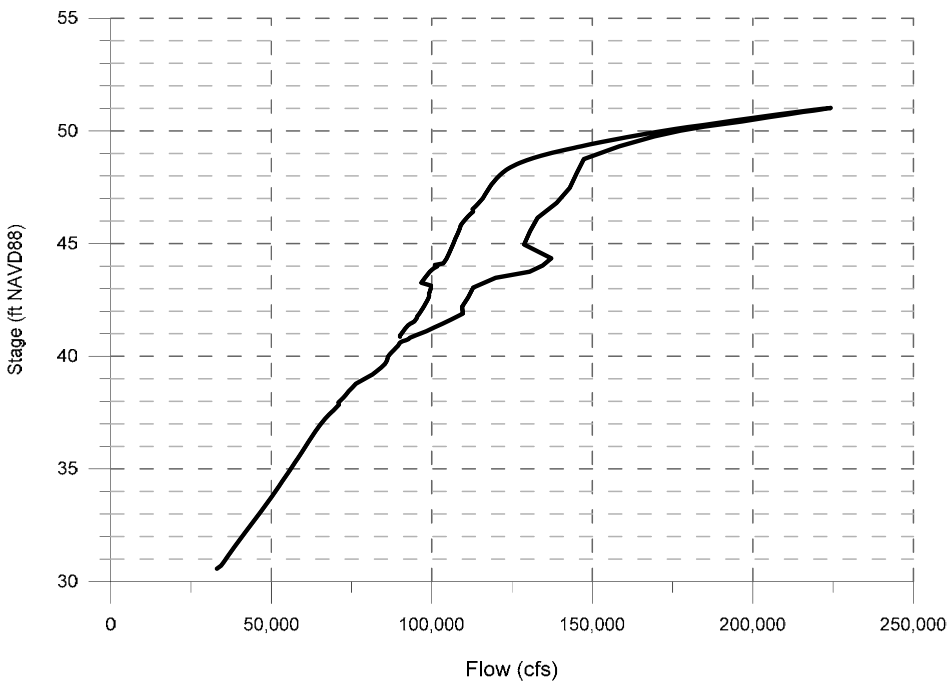
Note: Bank profiles and PNPs reflect improved levee ("baseline") condition


Baseline (Improved Levee) Condition, No Breach Rating Curve
Model River Mile 23.2 (Upstream XS for Index Location 8)
Stage (ft NAVD88)
60
55
50
45
40
35
0
50,000
100,000
150,000
200,000
250,000
300,000
350,000
Flow (cfs)


Baseline (Improved Levee) Condition, No Breach Rating Curve
Model River Mile 22.2 (Upstream XS for Index Locations 6 and 6A)
Stage (ft NAVD88)
55
50
45
40
35
30
0
50,000
100,000
150,000
200,000
250,000
Flow (cfs)

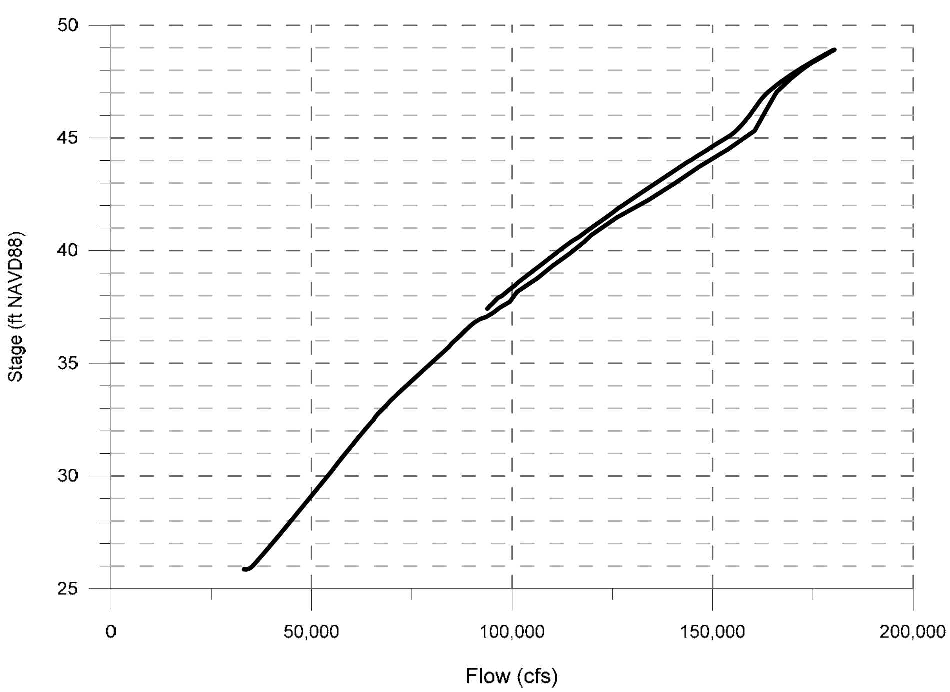
Baseline (Improved Levee) Condition, No Breach Rating Curve

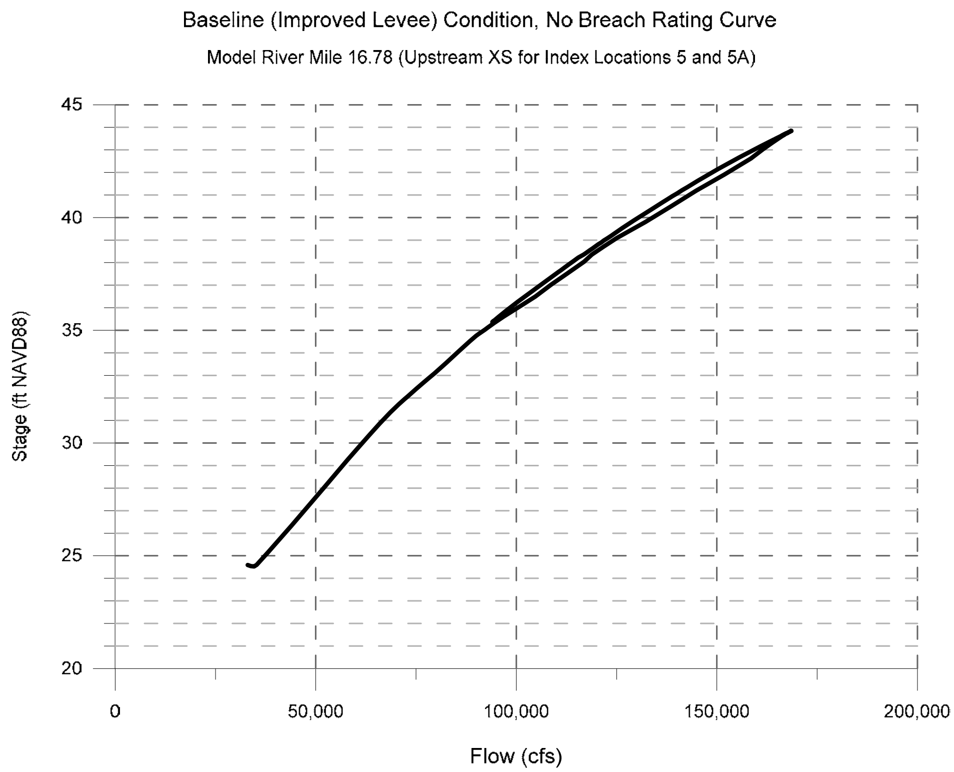
Model River Mile 21.6 (Upstream XS for Index Location 1)



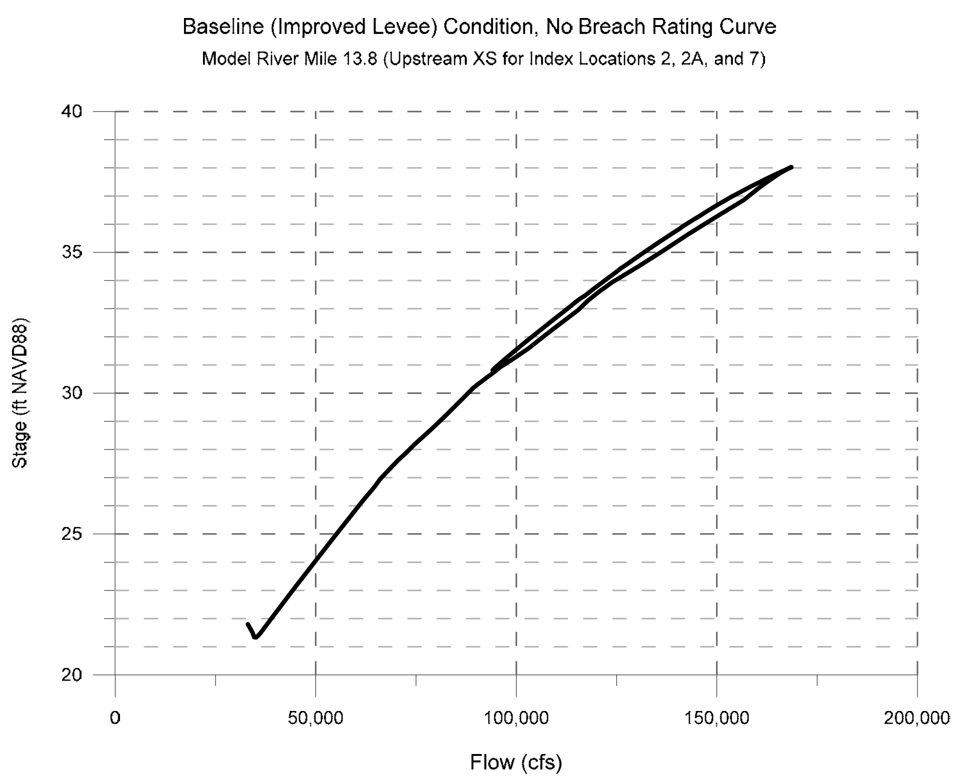
Baseline (Improved Levee) Condition, No Breach Rating Curve

Model River Mile 17.9 (Upstream XS for Index Location 1A)





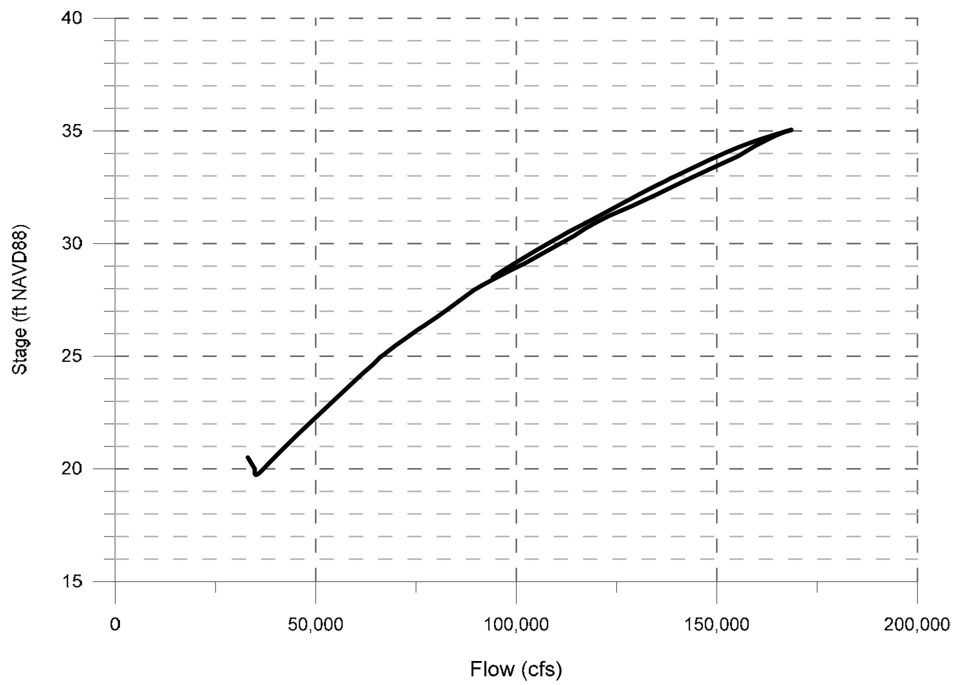
Baseline (Improved Levee) Condition, No Breach Rating Curve
Model River Mile 16.78 (Upstream XS for Index Locations 5 and 5A)
Stage (ft NAVD88)
45
40
35
30
25
20
0
50,000
100,000
150,000
200,000
Flow (cfs)



Baseline (Improved Levee) Condition, No Breach Rating Curve
Model River Mile 13.8 (Upstream XS for Index Locations 2, 2A, and 7)
Stage (ft NAVD88)
40
35
30
25
20
0
50,000
100,000
150,000
200,000
Flow (cfs)

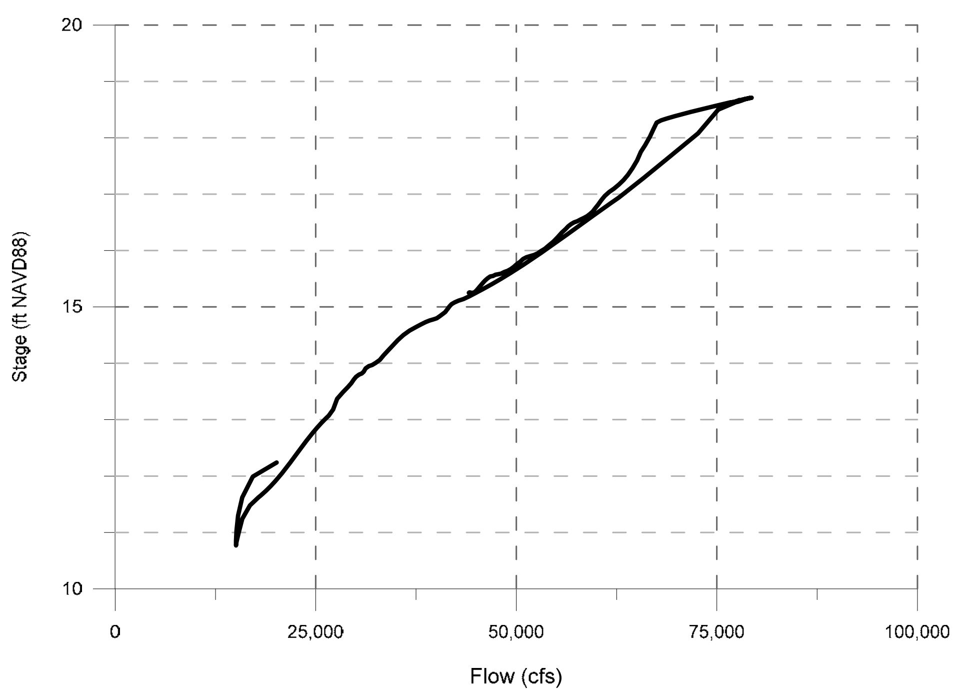
Baseline (Improved Levee) Condition, No Breach Rating Curve

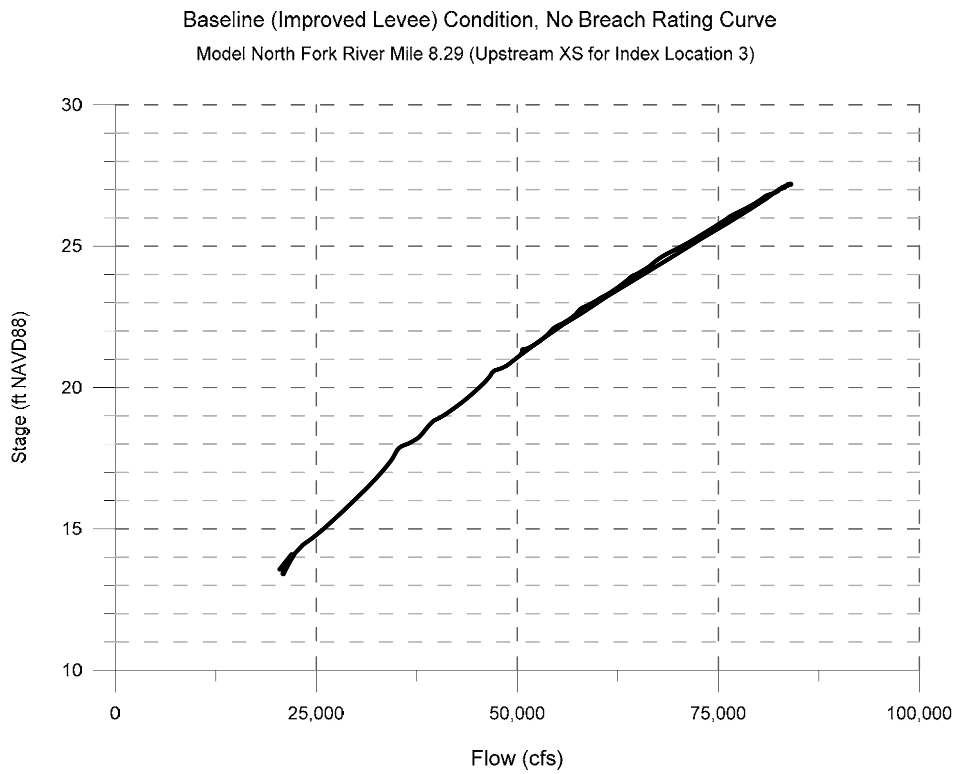
Model River Mile 12.4 (Upstream XS for Index Location 4A)



Baseline (Improved Levee) Condition, No Breach Rating Curve

Model South Fork River Mile 4.65 (Upstream XS for Index Location 4)




Baseline (Improved Levee) Condition, No Breach Rating Curve
Model North Fork River Mile 8.29 (Upstream XS for Index Location 3)
Stage (ft NAVD88)
30
25
20
15
10
0
25,000
50,000
75,000
100,000
Flow (cfs)

APPENDIX 6-1

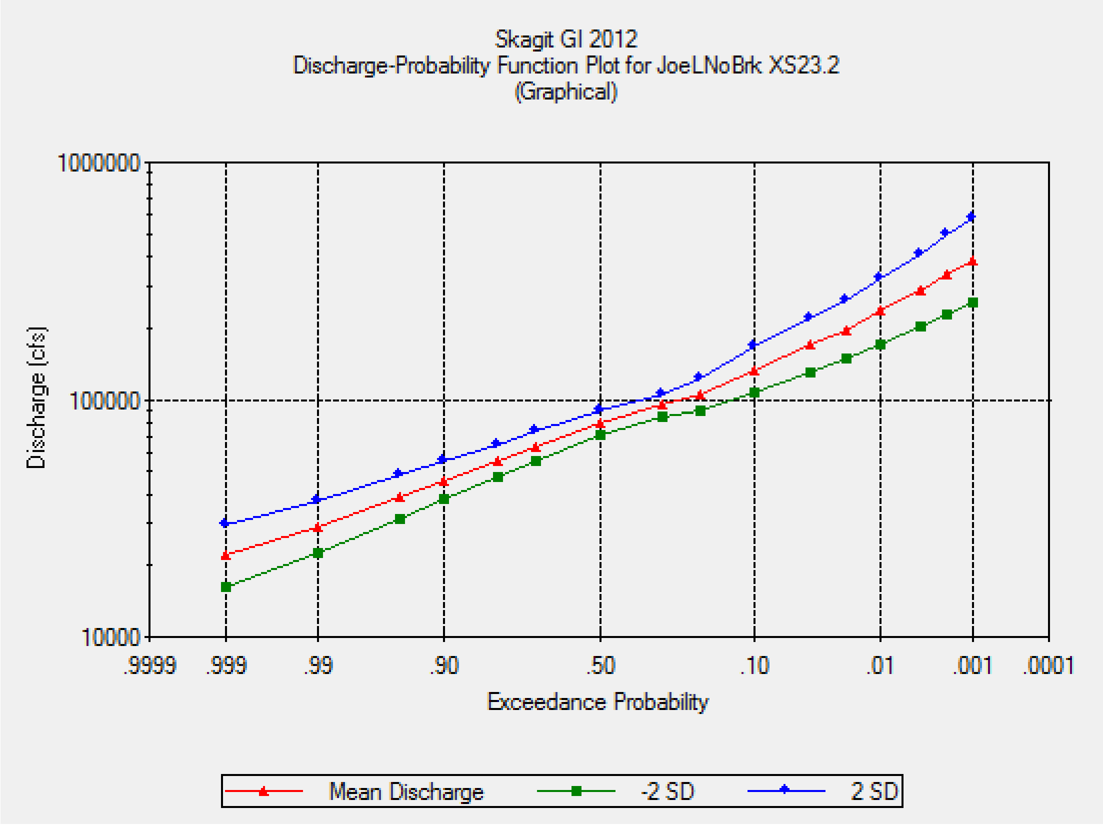
Joe Leary Slough Flood Bypass No-Breach Simulation Results

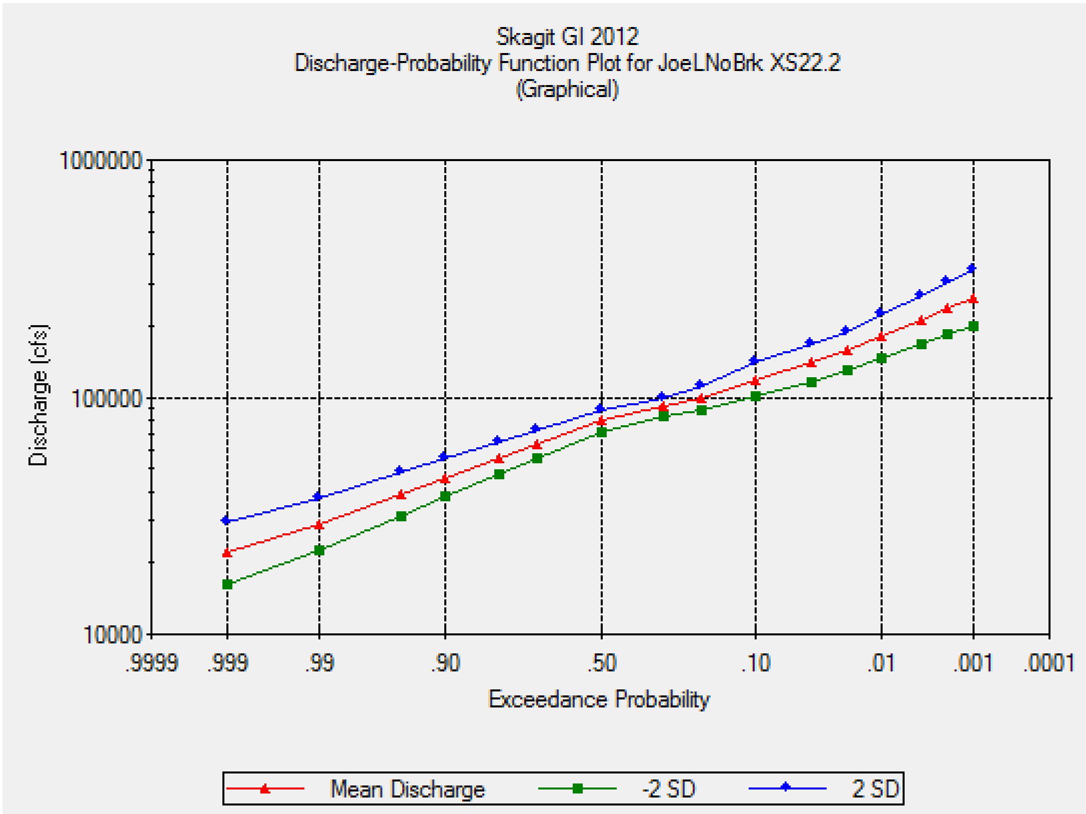
Joe Leary Slough Flood Bypass, Wide Variant, No-Breach Simulation Results

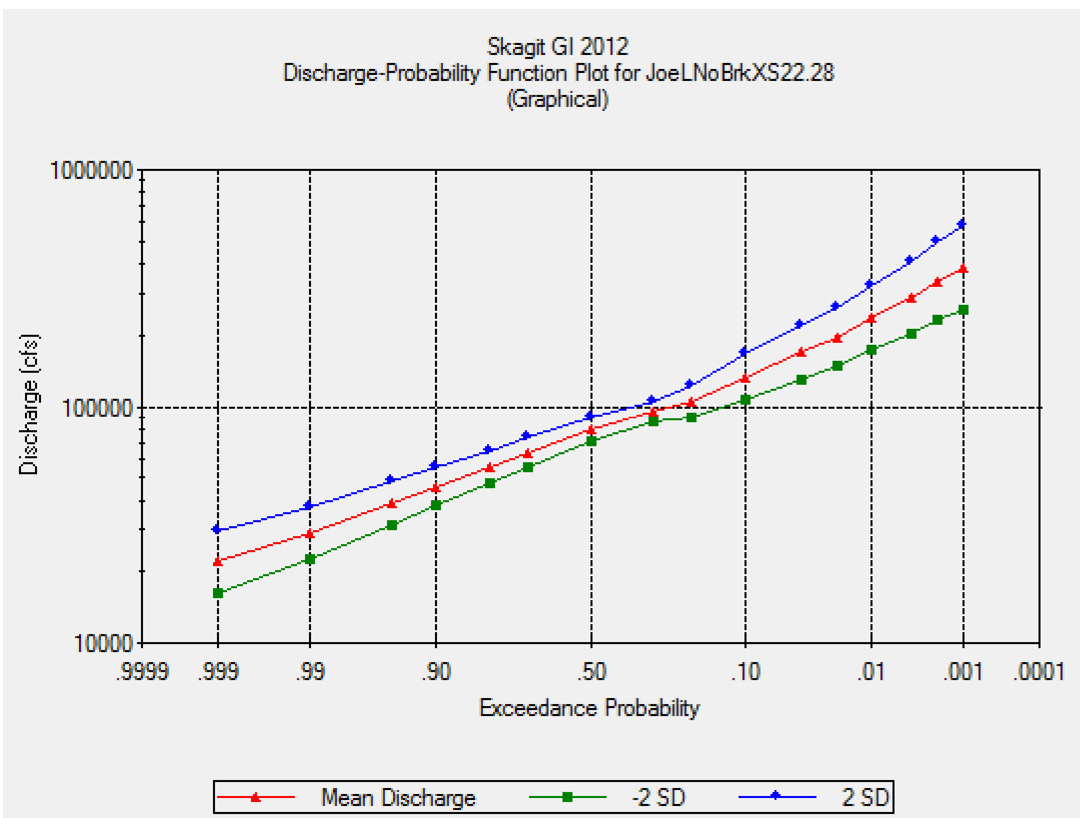
Flood quantiles (cfs), no breach, Joe Leary Slough Flood Bypass, wide variant, existing regulation

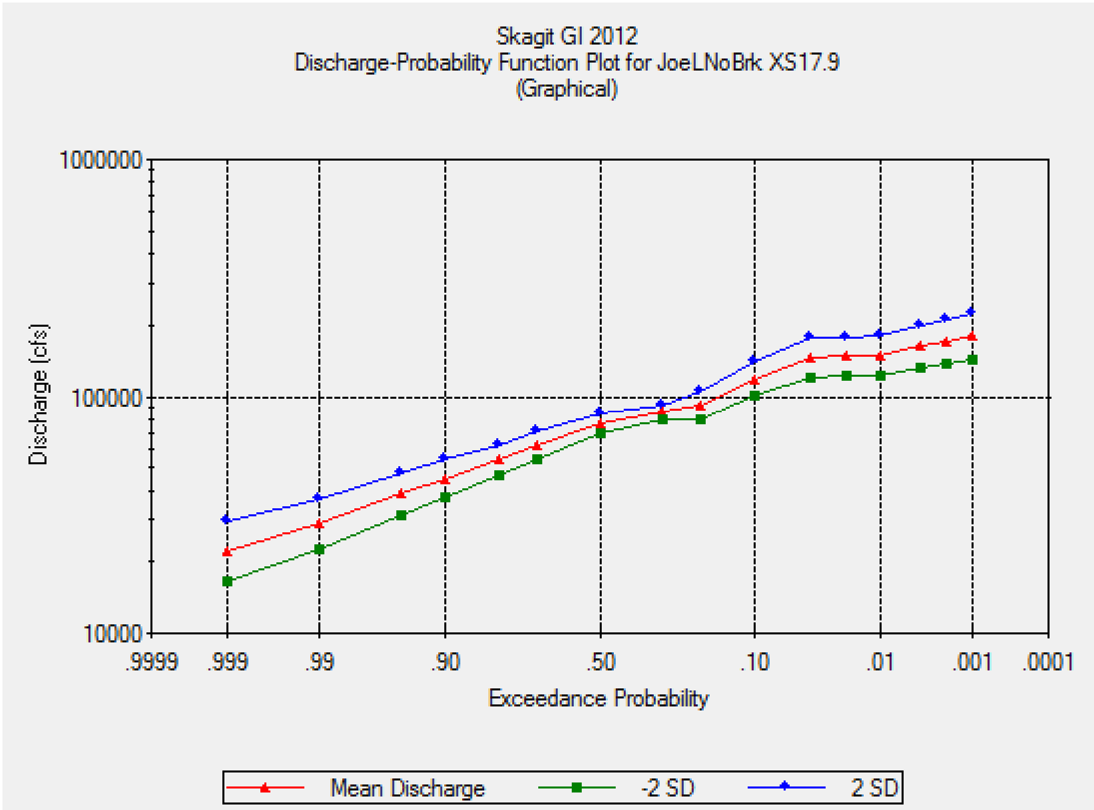
Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)										Joe Leary Bypass
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 22.28) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5A)	RM 13.3 (XS 13.8) (5)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)	
2	80,400	79,500	80,500	77,000	77,000	76,900	76,900	76,900	34,900	41,800	0
5	105,200	99,100	105,300	91,400	91,400	91,400	91,400	91,400	42,500	49,000	0
10	132,800	118,900	132,800	118,100	118,000	118,000	118,000	118,000	56,200	61,600	0
25	169,800	139,500	169,700	145,500	145,500	145,400	145,400	145,200	69,000	74,300	63,500
50	197,000	156,400	197,200	147,600	147,200	147,000	147,000	146,600	69,700	74,600	75,900
75	220,000	172,000	220,400	148,200	148,100	147,700	147,700	147,400	70,500	75,700	82,500
100	235,700	181,600	236,100	149,800	149,400	149,400	149,400	149,300	72,400	77,000	87,400
250	289,900	212,100	290,300	163,800	163,200	163,200	163,200	163,200	78,000	82,500	117,000
500	337,500	236,400	337,800	171,600	166,300	166,300	166,300	166,300	78,800	83,400	134,200

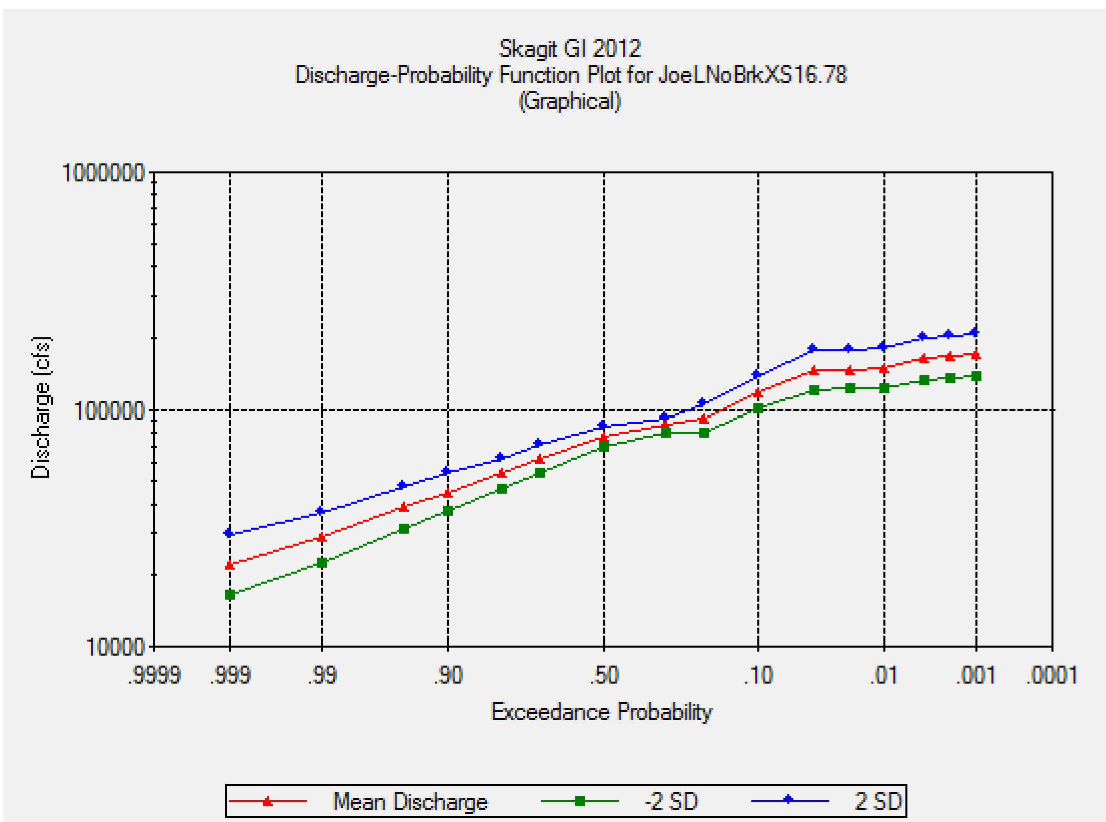
Index Point location for Damage Reaches 1 and 5 for Joe Leary Bypass are different from those assumed for Existing Condition and Improved Levee Condition

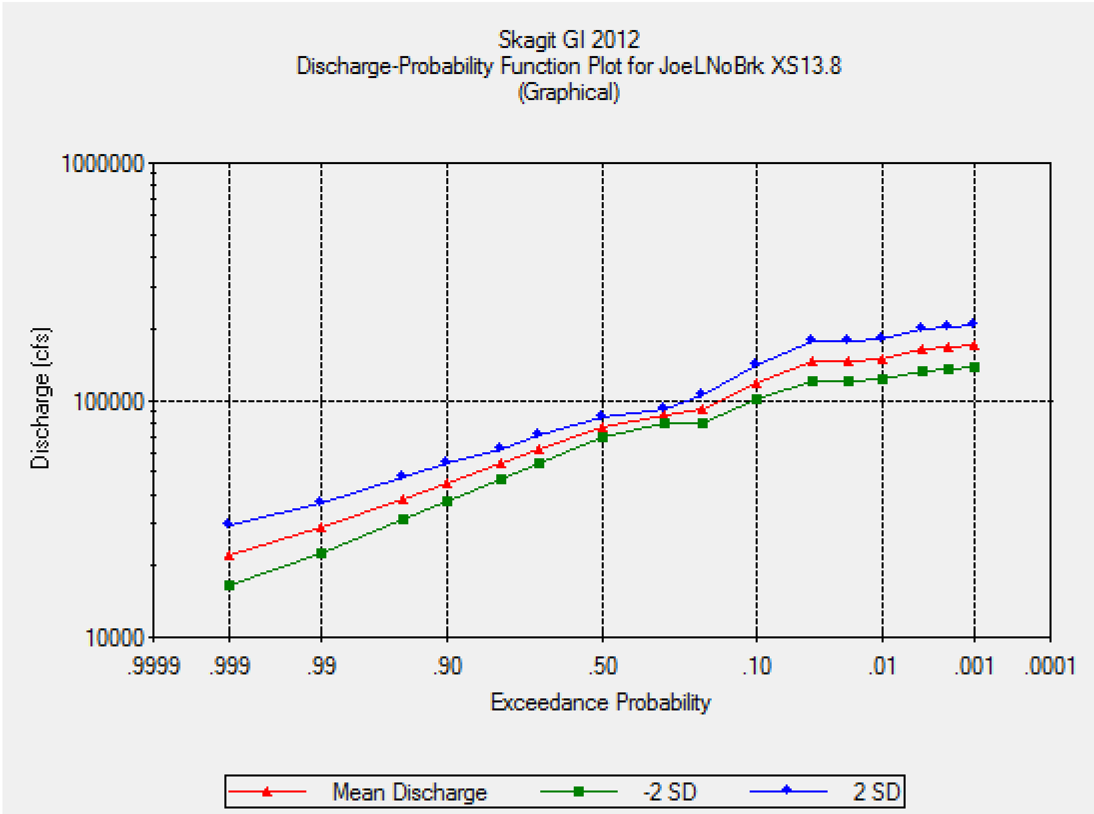


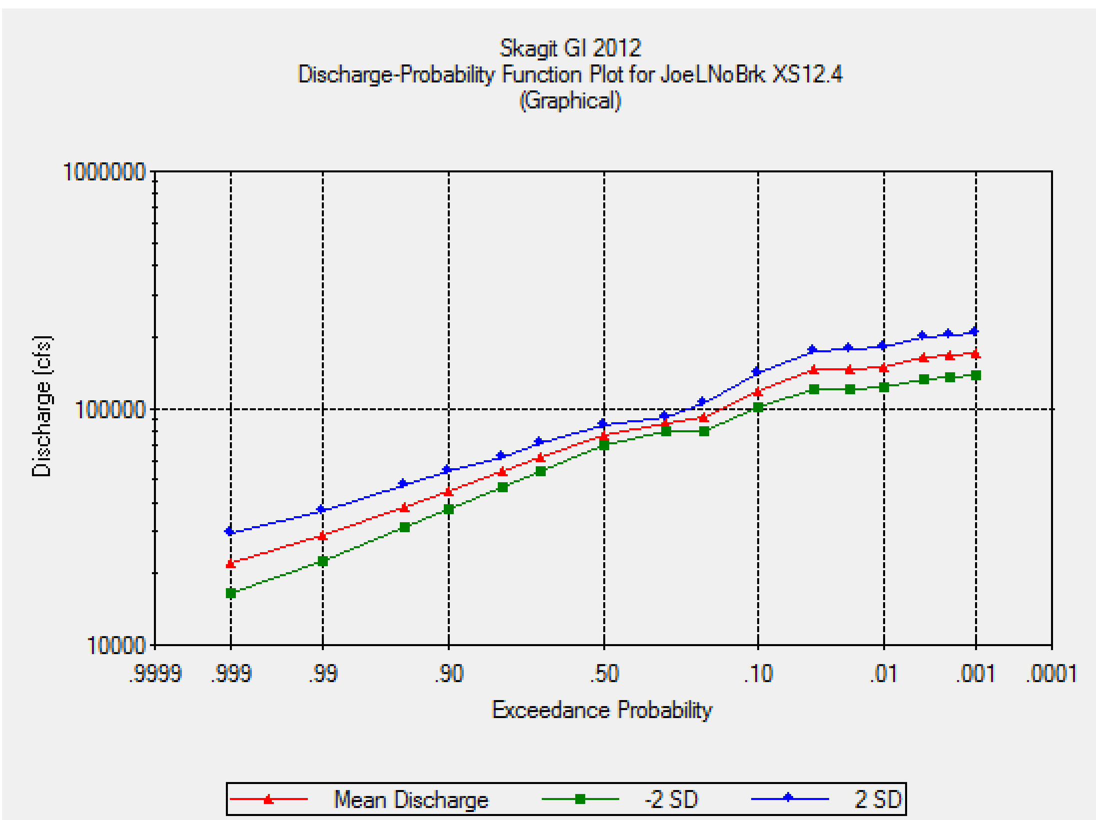

Skagit GI 2012
Discharge-Probability Function Plot for JoeLNoBrk XS22.2
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

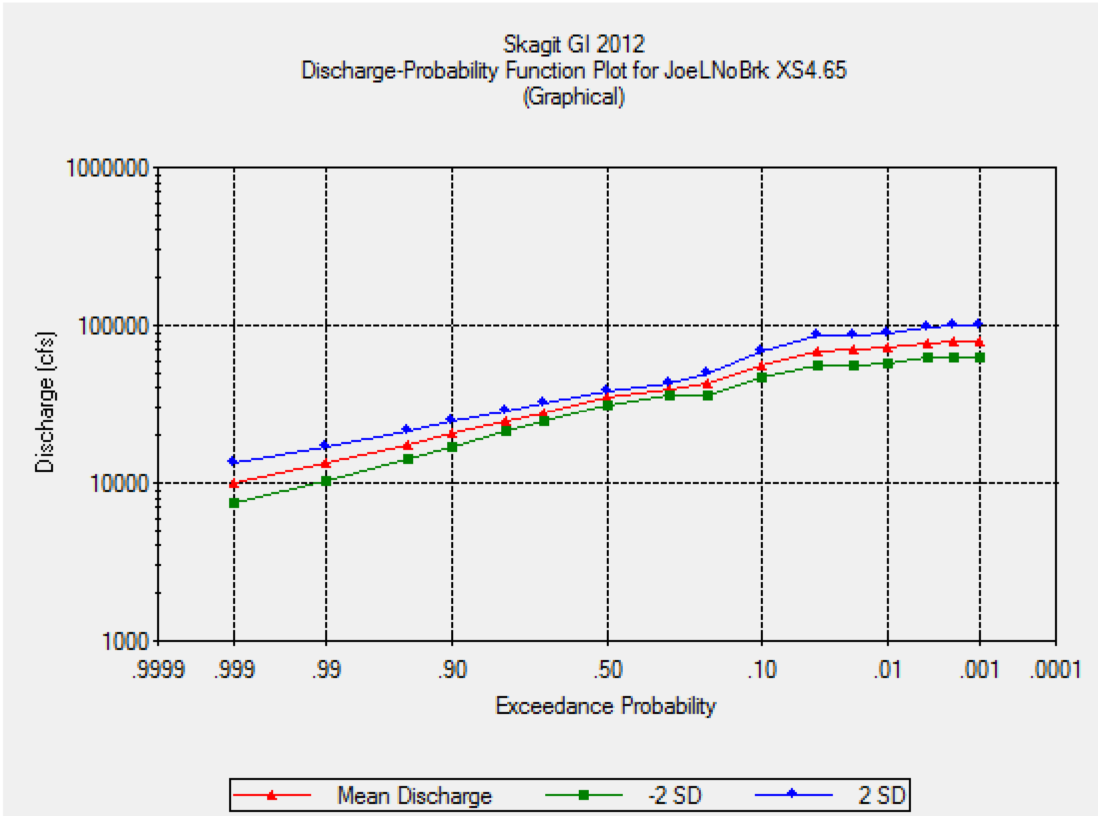

Skagit GI 2012
Discharge-Probability Function Plot for JoeLNoBrkXS22.28
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

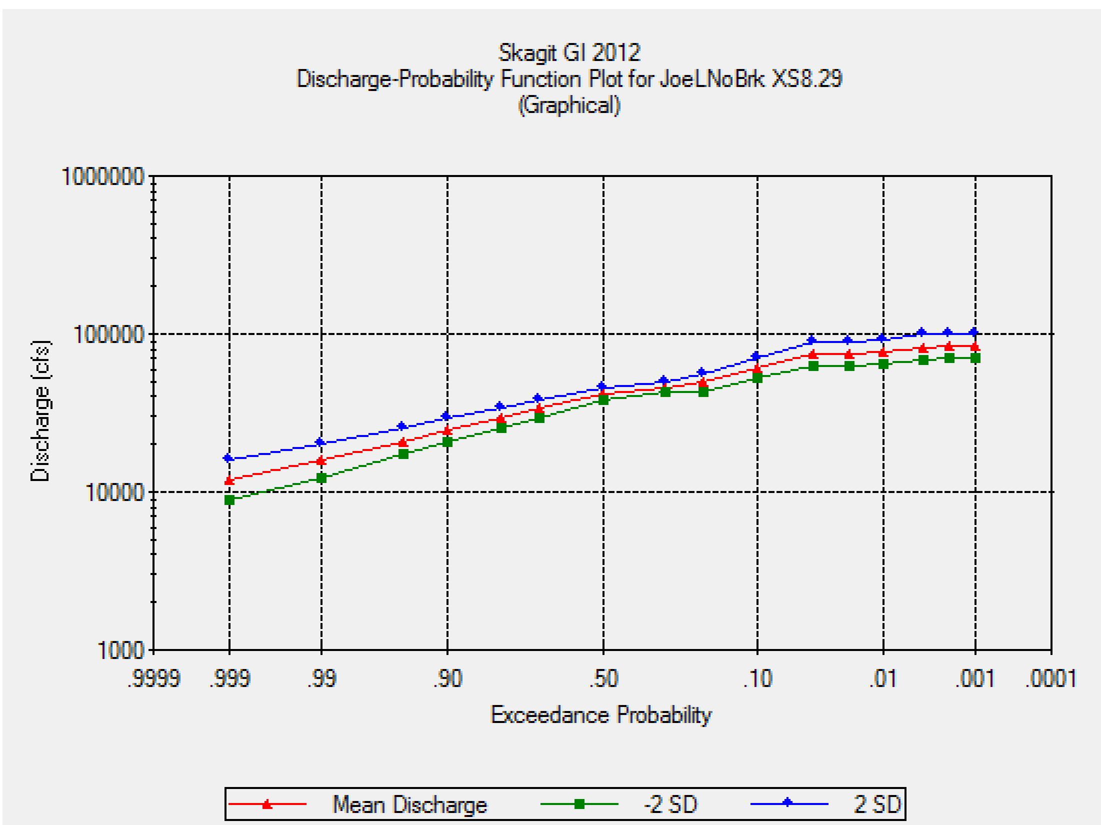

Skagit GI 2012
Discharge-Probability Function Plot for JoeLNoBrk XS17.9
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Skagit GI 2012
Discharge-Probability Function Plot for JoeLNoBrkXS16.78
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

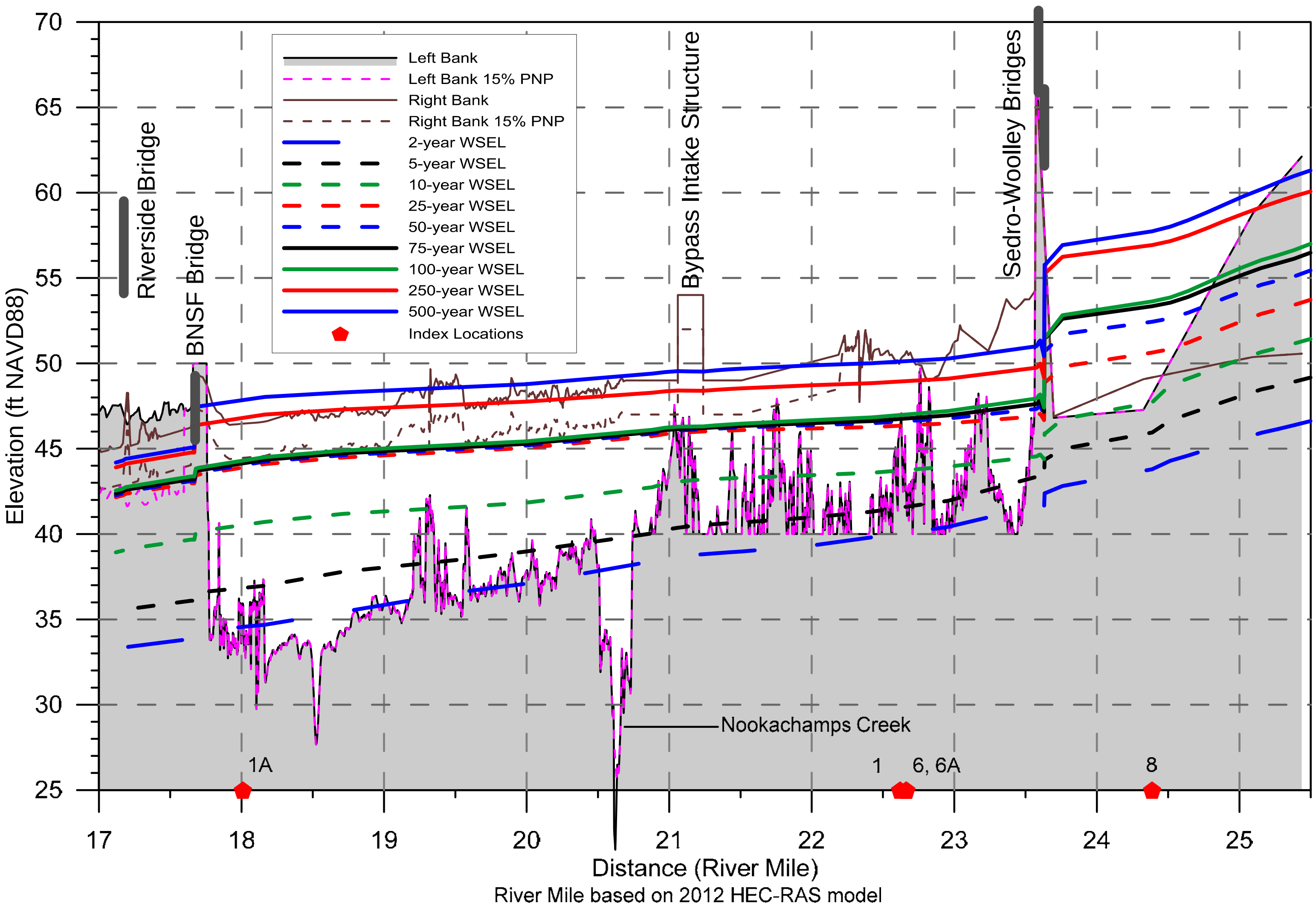

Skagit GI 2012
Discharge-Probability Function Plot for JoeLNoBrk XS13.8
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Skagit GI 2012
Discharge-Probability Function Plot for JoeLNoBrk XS12.4
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

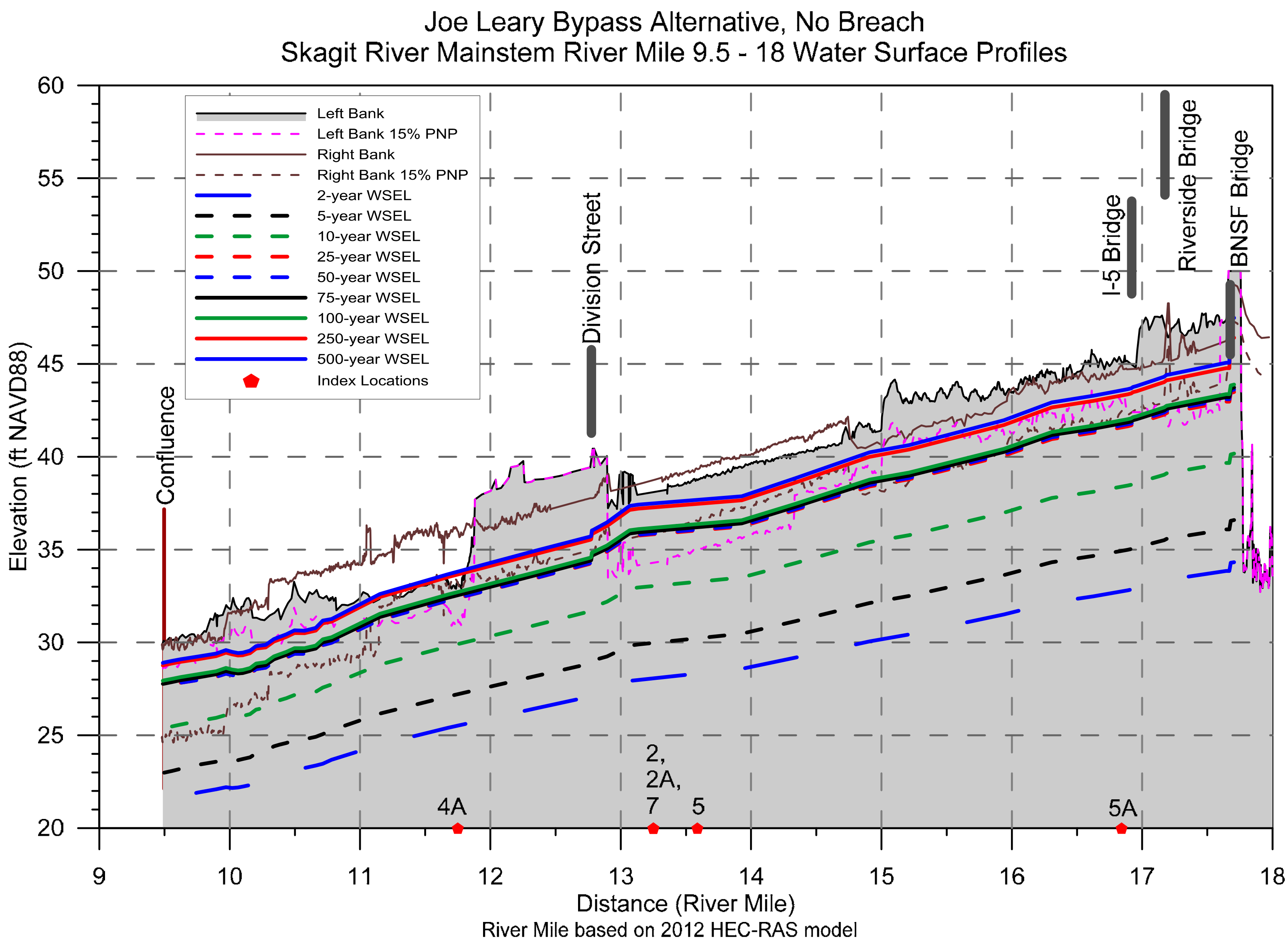

Skagit GI 2012
Discharge-Probability Function Plot for JoeLNoBrk XS4.65
(Graphical)
Discharge (cfs)
1000000
100000
10000
1000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Skagit GI 2012
Discharge-Probability Function Plot for JoeLNoBrk XS8.29
(Graphical)
Discharge (cfs)
1000000
100000
10000
1000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

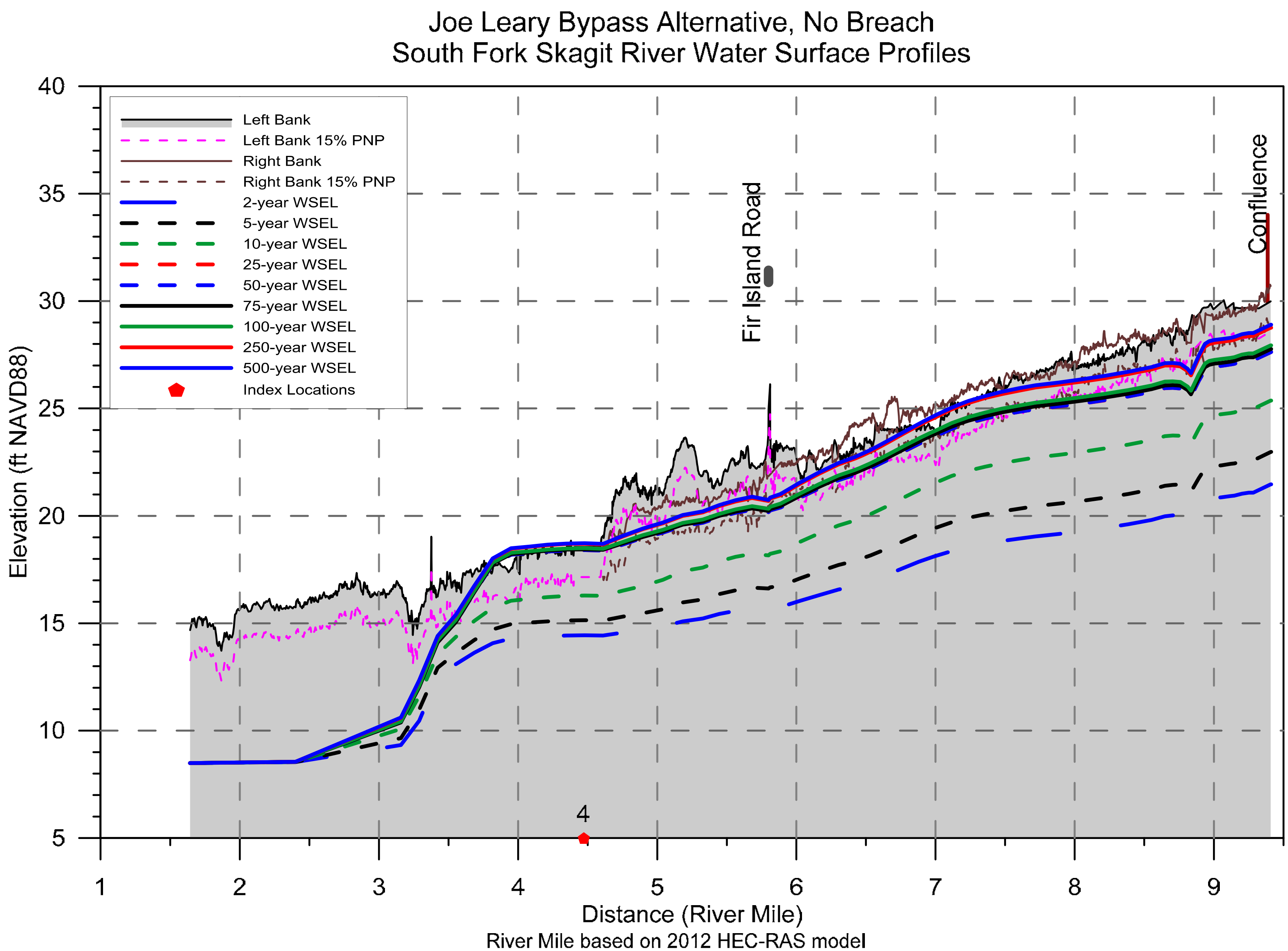
Joe Leary Bypass Alternative, No Breach
Skagit River Mainstem River Mile 17 - 25.5 Water Surface Profiles



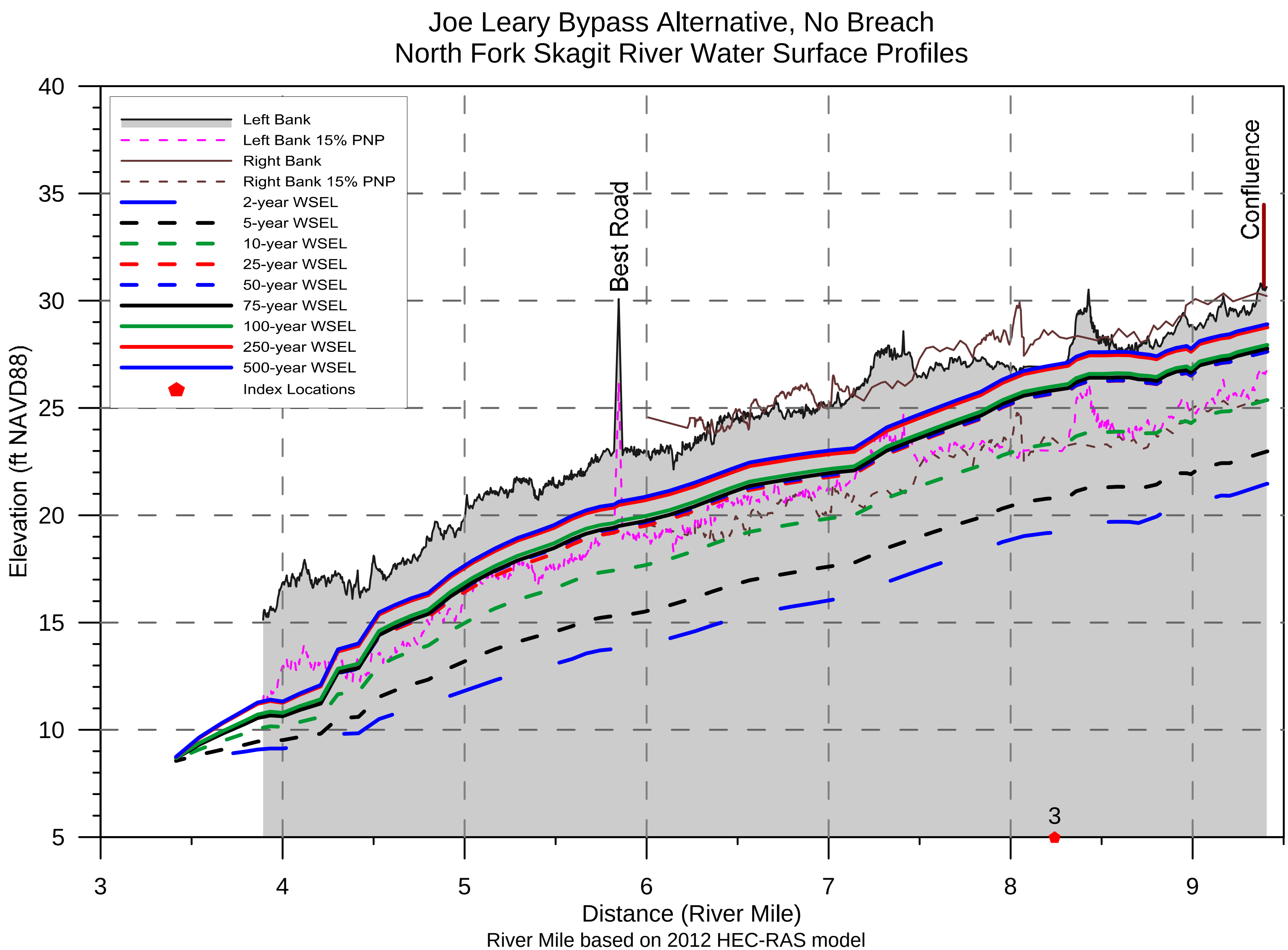
Note: Bank elevation profiles represent the "with alternative" condition.



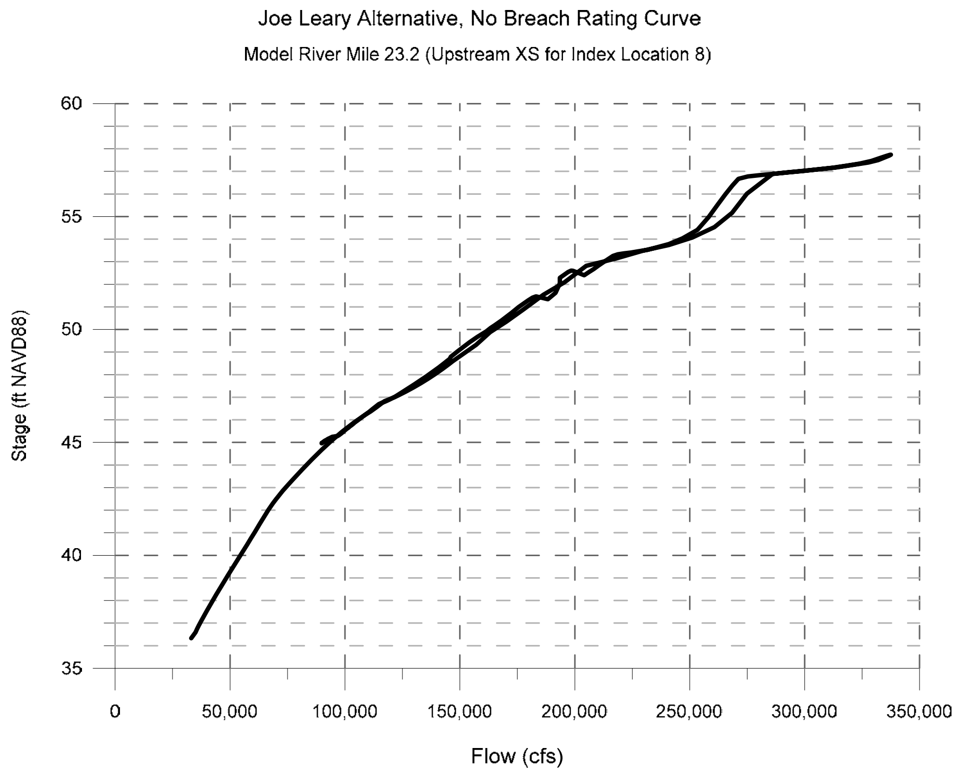
Note: Bank elevation profiles represent the "with alternative" condition.



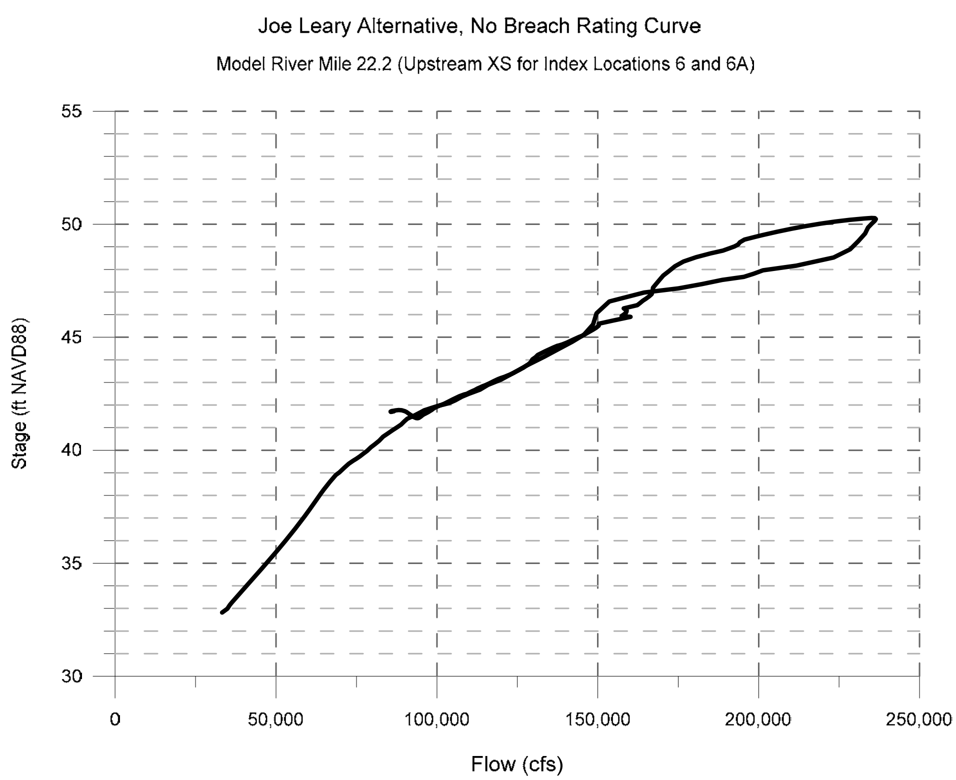
Note: Bank elevation profiles represent the "with alternative" condition.



Note: Bank elevation profiles represent the "with alternative" condition.



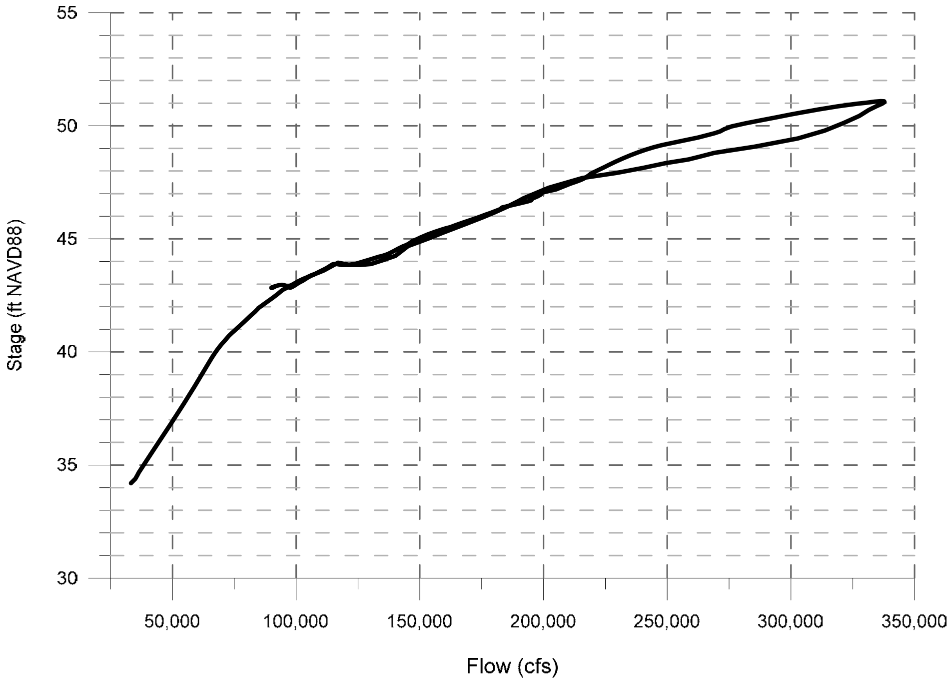
Joe Leary Alternative, No Breach Rating Curve
Model River Mile 23.2 (Upstream XS for Index Location 8)
Stage (ft NAVD88)
60
55
50
45
40
35
0
50,000
100,000
150,000
200,000
250,000
300,000
350,000
Flow (cfs)



Joe Leary Alternative, No Breach Rating Curve
Model River Mile 22.2 (Upstream XS for Index Locations 6 and 6A)
Stage (ft NAVD88)
55
50
45
40
35
30
0
50,000
100,000
150,000
200,000
250,000
Flow (cfs)

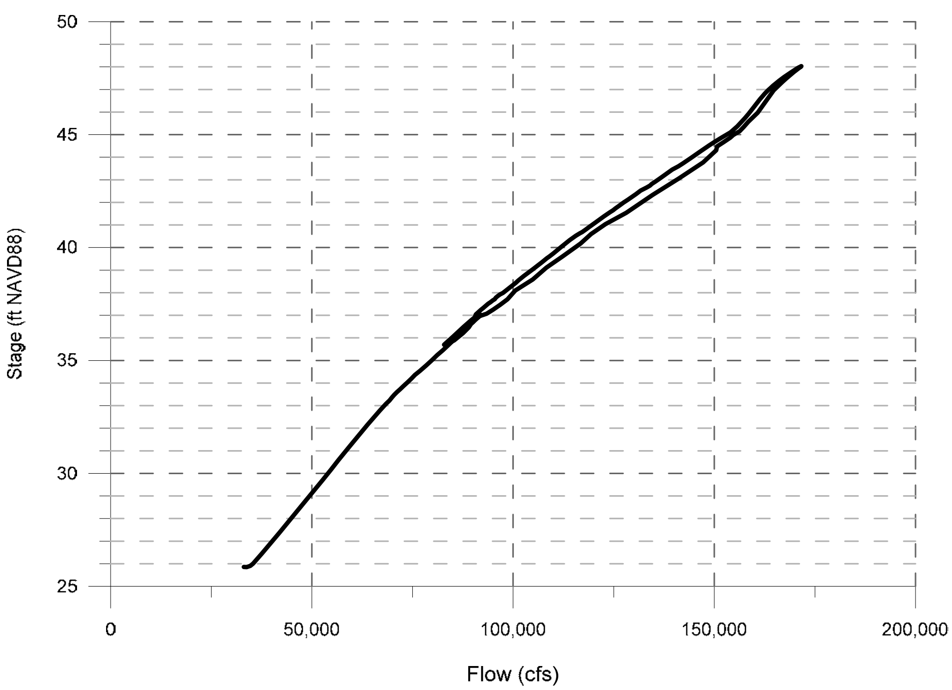
Joe Leary Alternative, No Breach Rating Curve

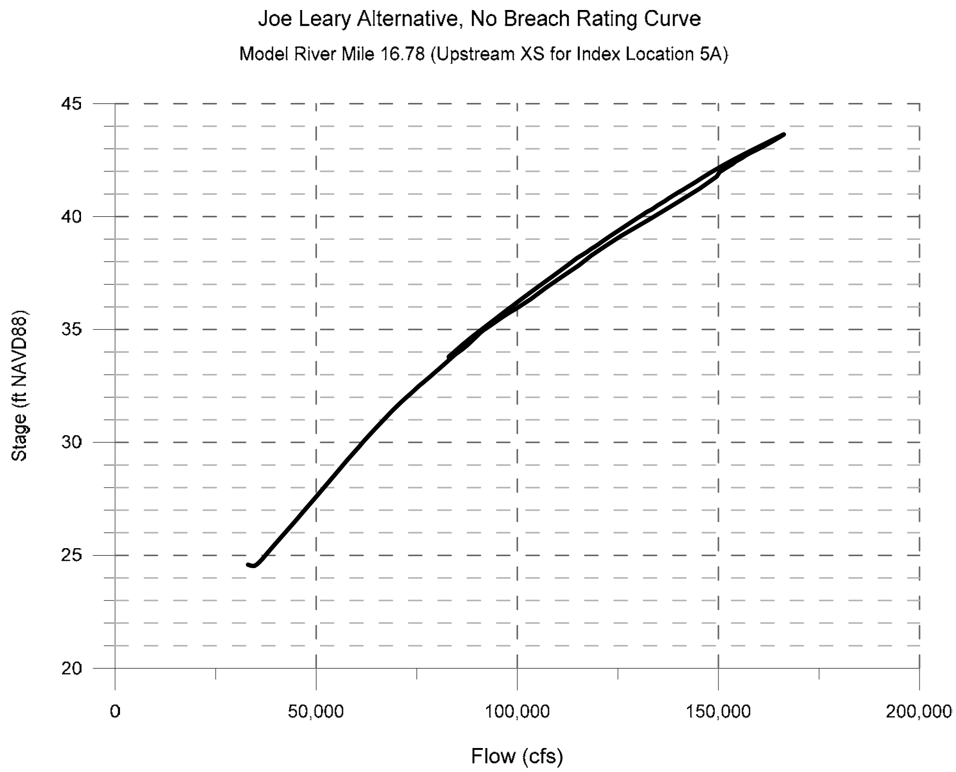
Model River Mile 22.27 (Upstream XS for Index Location 1- Joe Leary Alternative)

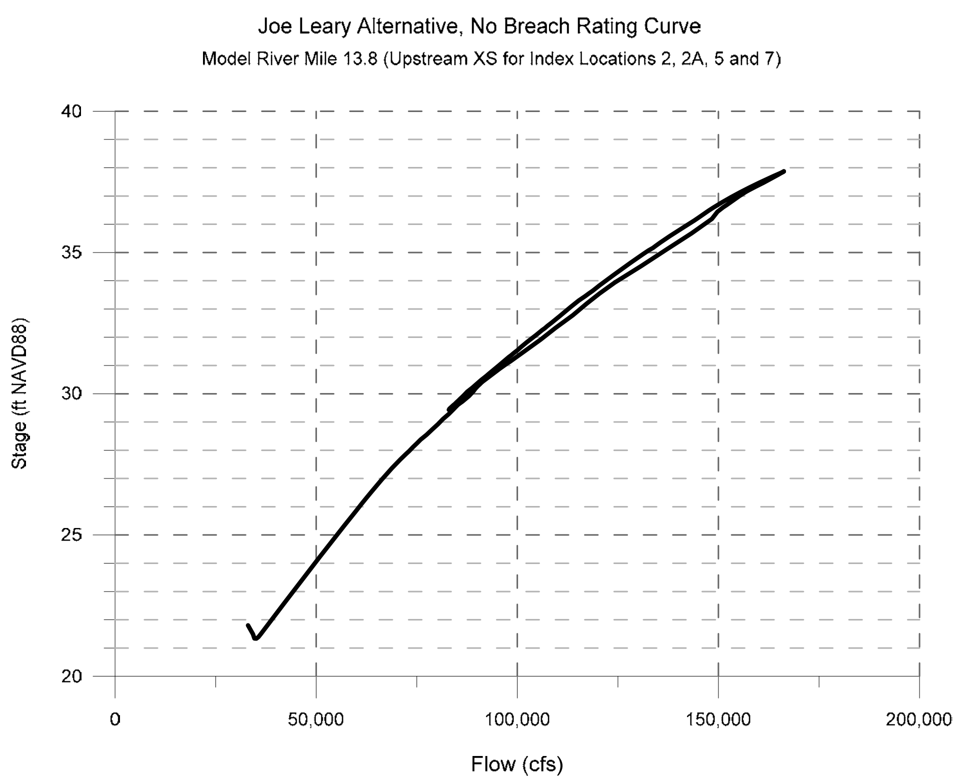


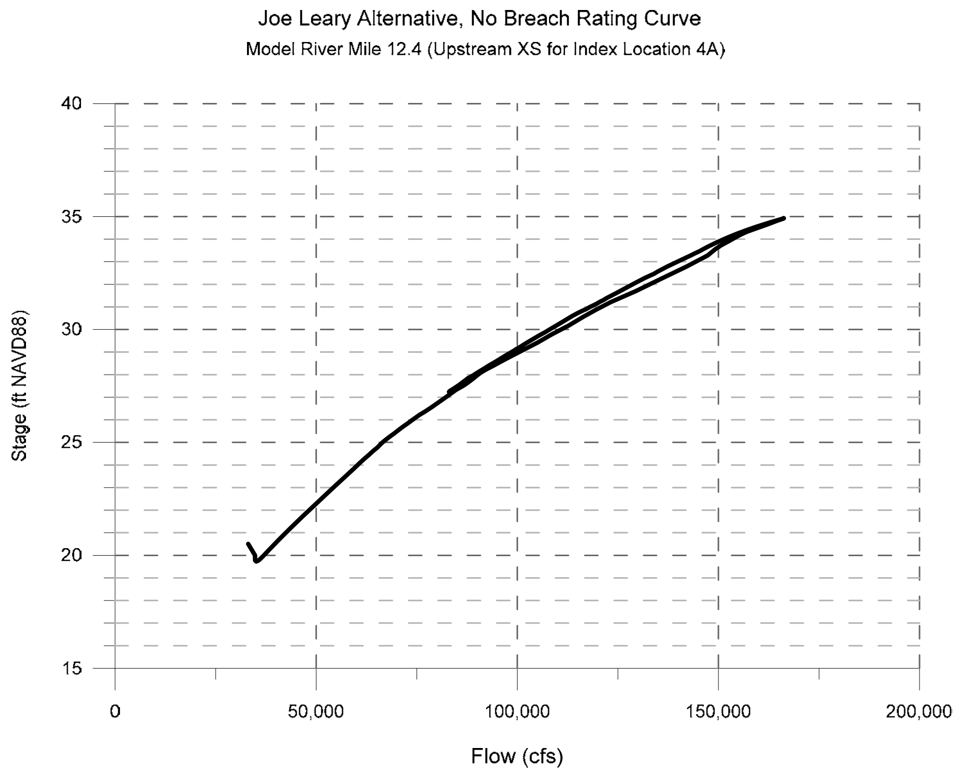
Joe Leary Alternative, No Breach Rating Curve

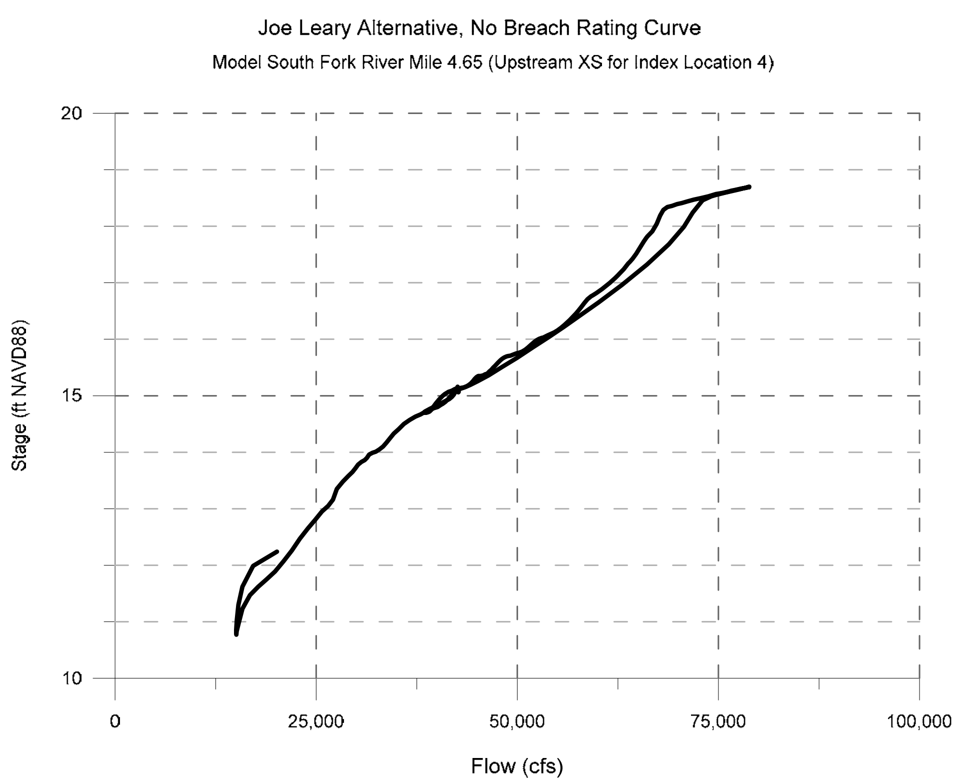
Model River Mile 17.9 (Upstream XS for Index Location 1A)

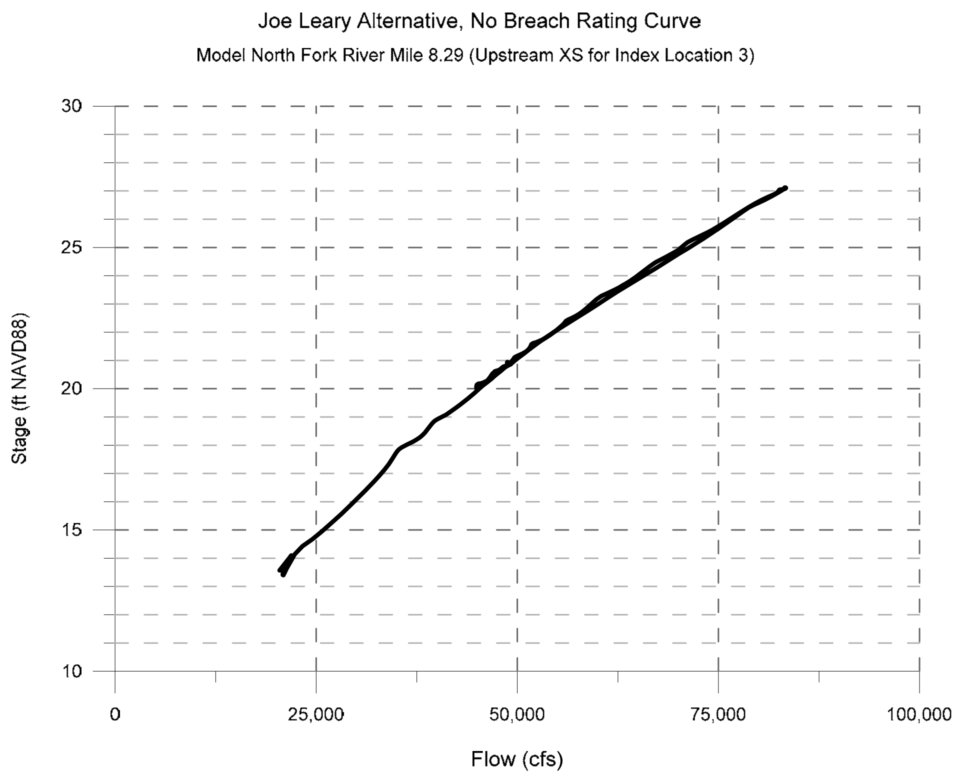



Joe Leary Alternative, No Breach Rating Curve
Model River Mile 16.78 (Upstream XS for Index Location 5A)
Stage (ft NAVD88)
45
40
35
30
25
20
0
50,000
100,000
150,000
200,000
Flow (cfs)


Joe Leary Alternative, No Breach Rating Curve
Model River Mile 13.8 (Upstream XS for Index Locations 2, 2A, 5 and 7)
Stage (ft NAVD88)
40
35
30
25
20
0
50,000
100,000
150,000
200,000
Flow (cfs)


Joe Leary Alternative, No Breach Rating Curve
Model River Mile 12.4 (Upstream XS for Index Location 4A)
Stage (ft NAVD88)
40
35
30
25
20
15
0
50,000
100,000
150,000
200,000
Flow (cfs)


Joe Leary Alternative, No Breach Rating Curve
Model South Fork River Mile 4.65 (Upstream XS for Index Location 4)
Stage (ft NAVD88)
20
15
10
0
25,000
50,000
75,000
100,000
Flow (cfs)


Joe Leary Alternative, No Breach Rating Curve
Model North Fork River Mile 8.29 (Upstream XS for Index Location 3)
30
25
20
15
10
Stage (ft NAVD88)
0
25,000
50,000
75,000
100,000
Flow (cfs)

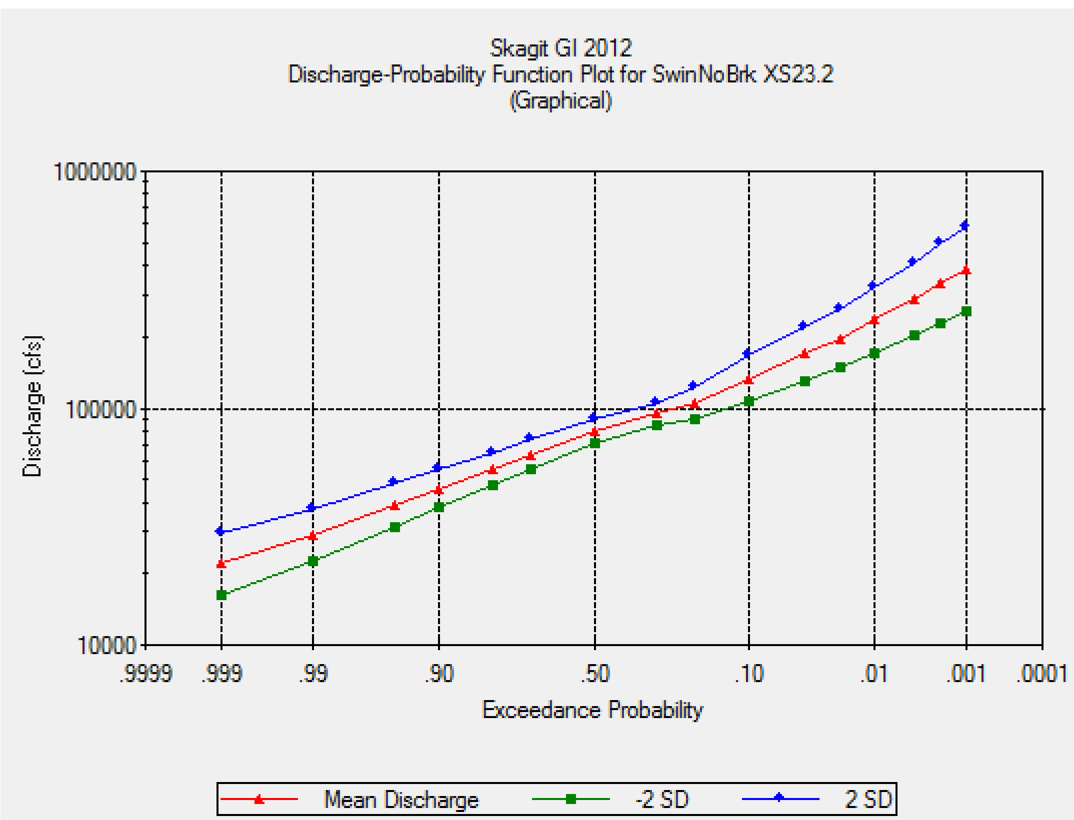
APPENDIX 6-2

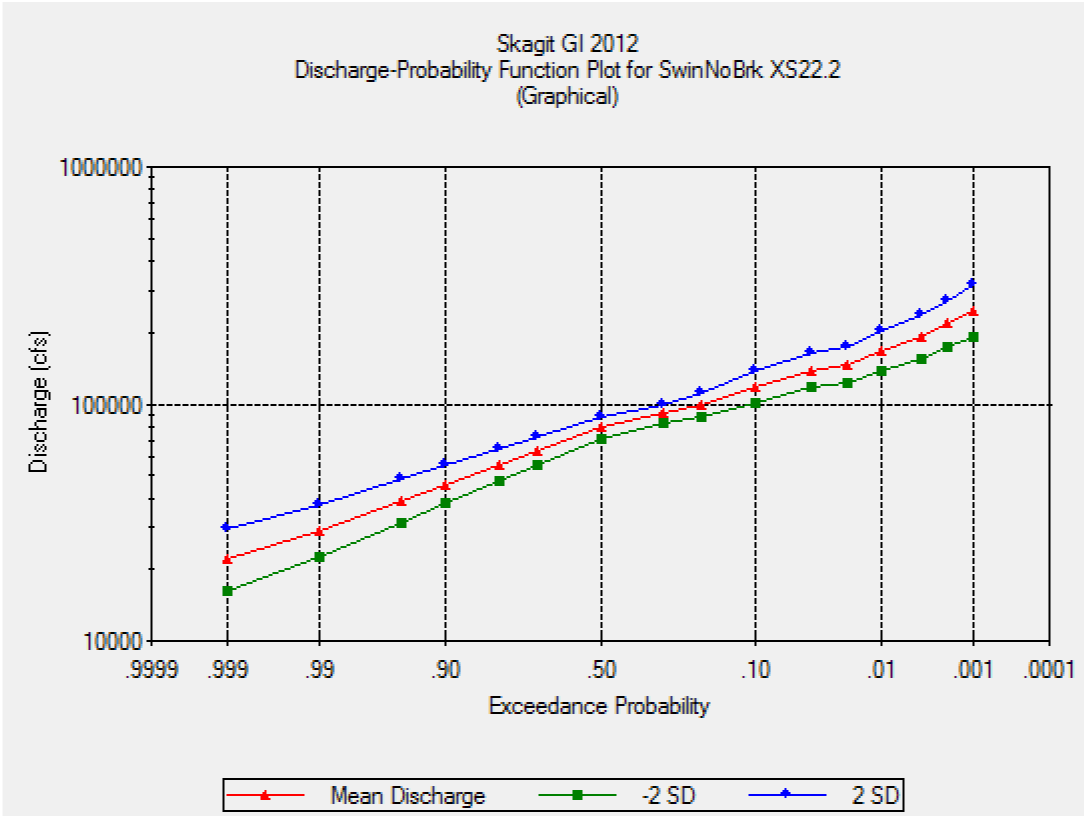
Swinomish Flood Bypass No-Breach Simulation Results

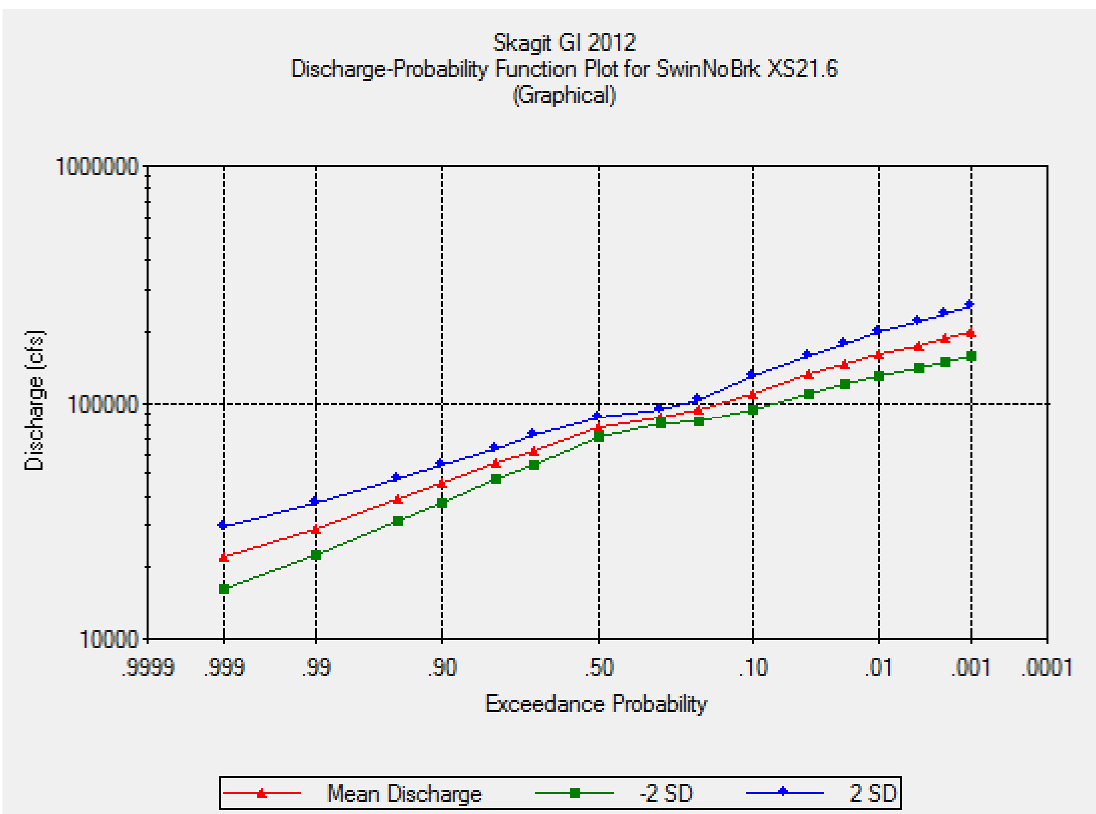
Swinomish Flood Bypass (Wide Configuration) No-Breach Simulation Results

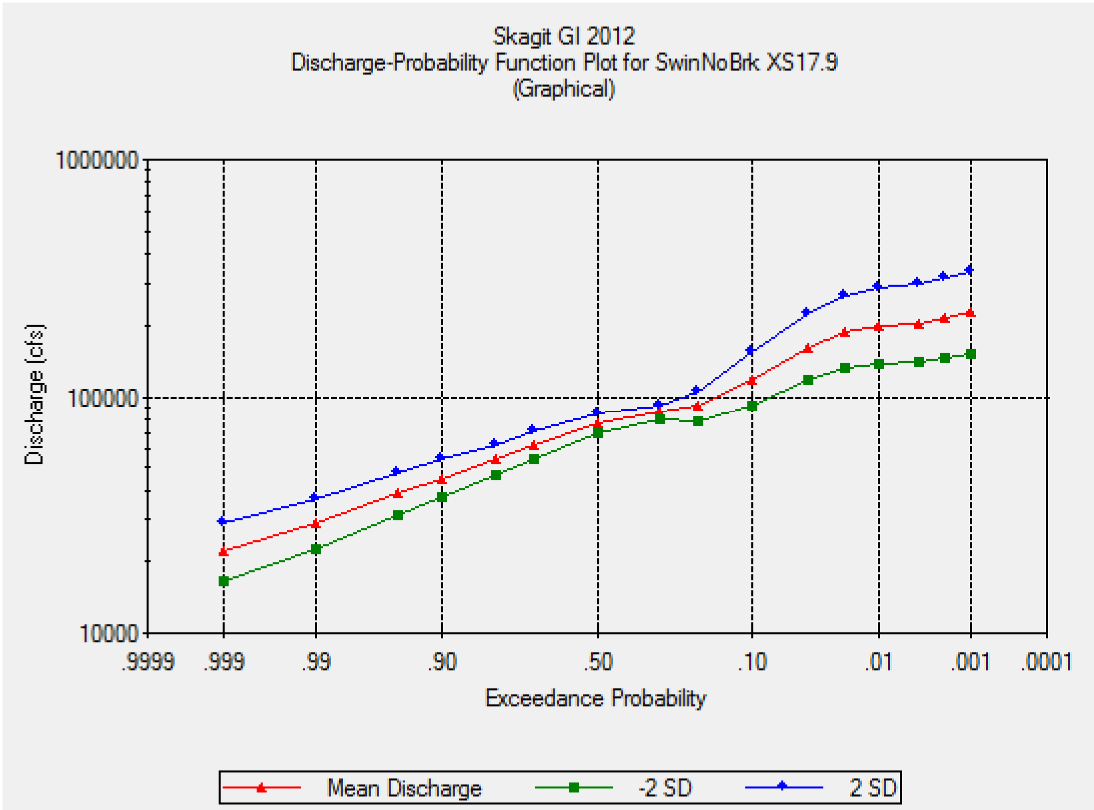
Flood quantiles (cfs), no breach, Swinomish Wide Bypass, existing regulation

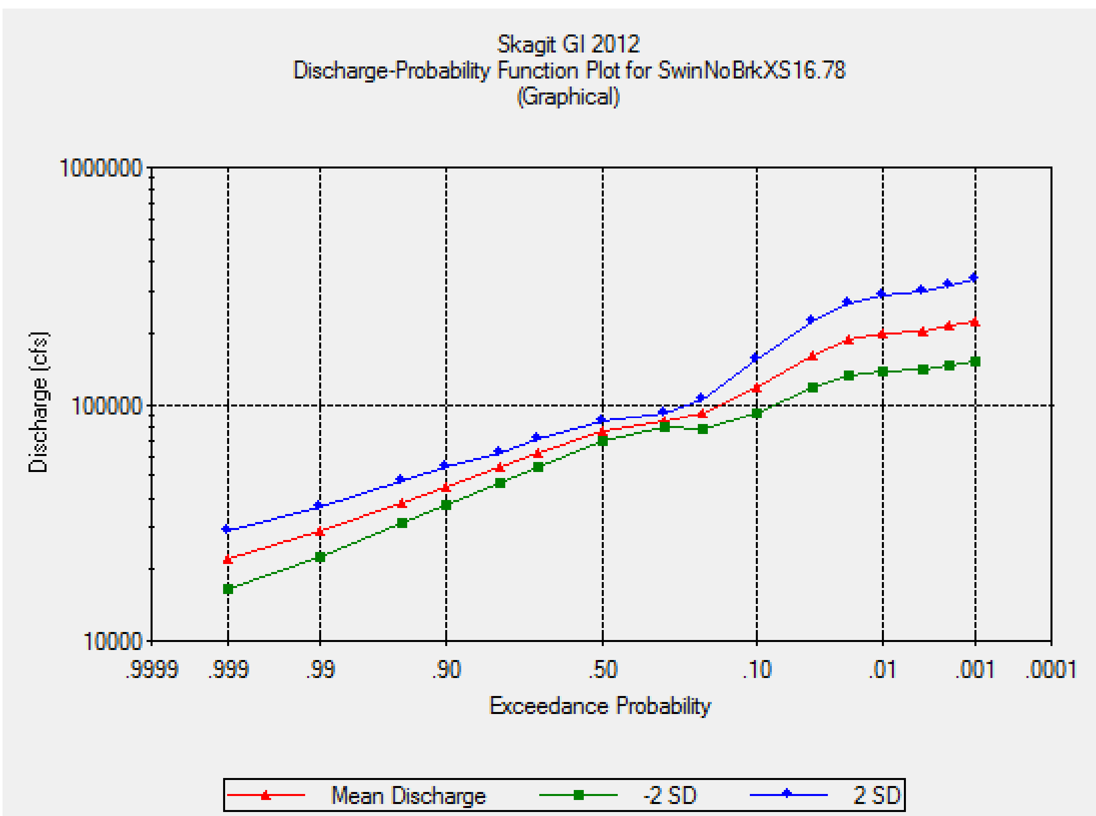
Recurrence Interval (years)	Index Point (associated XS)(associated Damage Reach)								
	RM 23.3 (XS 23.2) (8)	RM 22.1 (XS 22.2) (6, 6A)	RM 21.3 (XS 21.6) (1)	RM 18.1 (XS 17.9) (1A)	RM 16.8 (XS 16.78) (5, 5A)	RM 13.1 (XS 13.8) (2, 2A,7)	RM 12.7 (XS 12.4) (4A)	So Fk RM 4.5 (XS 465) (4)	No Fk RM 8.3 (XS 829) (3)
2	80,400	79,300	78,500	77,000	76,900	76,900	76,800	34,900	41,600
5	105,200	99,000	92,600	91,300	91,300	91,200	91,200	42,500	48,700
10	132,700	117,900	109,800	118,400	118,300	118,300	118,300	56,600	61,600
25	169,800	138,900	131,600	161,300	161,300	133,900	133,300	63,300	68,000
50	197,200	146,500	146,500	187,100	187,000	135,800	135,500	63,600	68,800
75	220,000	158,200	155,500	196,000	195,900	136,600	135,900	64,500	69,300
100	235,700	166,000	160,800	199,300	199,200	137,200	136,900	65,500	70,200
250	289,900	190,900	174,600	204,800	204,700	138,700	138,700	66,900	71,600
500	337,500	217,800	187,300	216,100	215,000	144,200	144,200	69,600	74,200

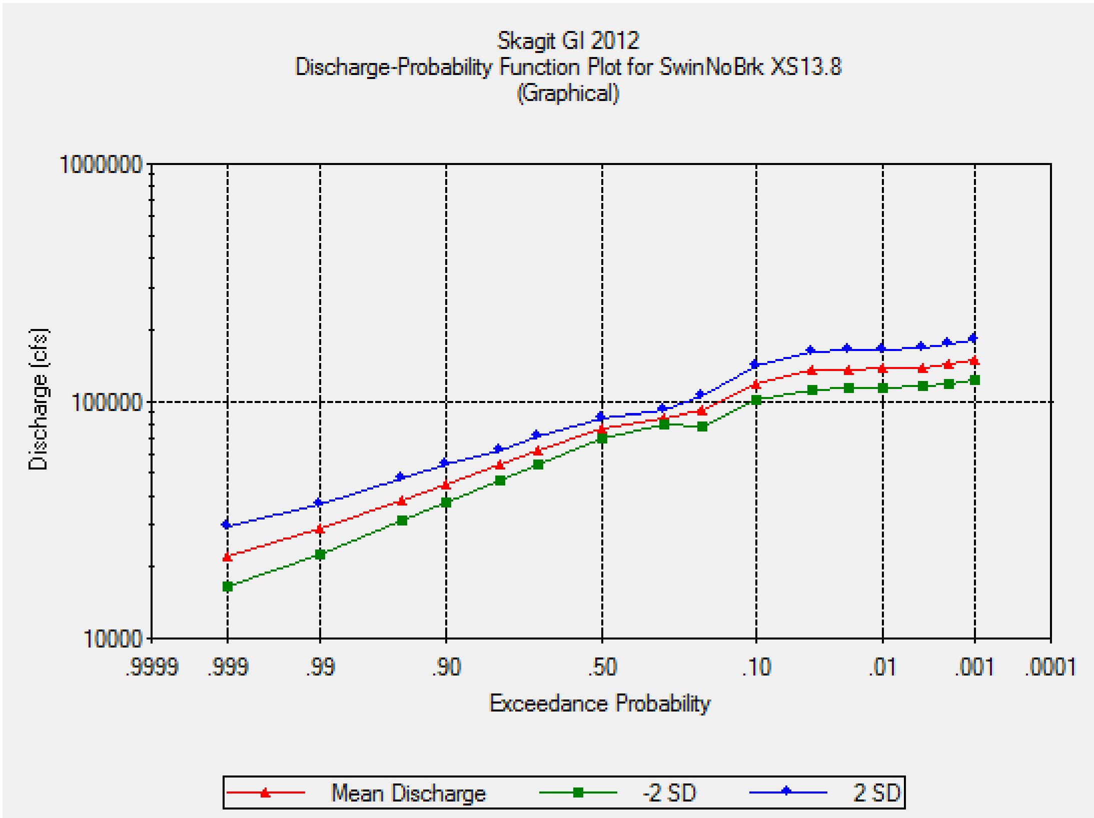


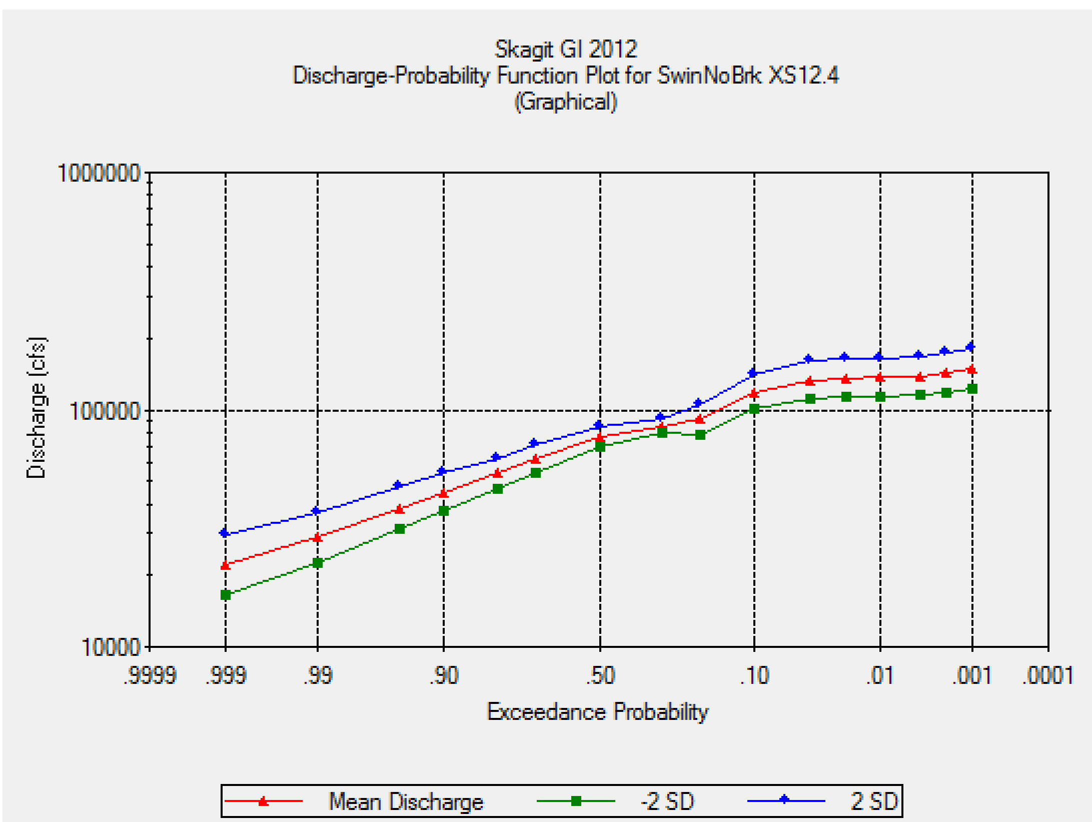

Skagit GI 2012
Discharge-Probability Function Plot for SwinNoBrk XS22.2
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

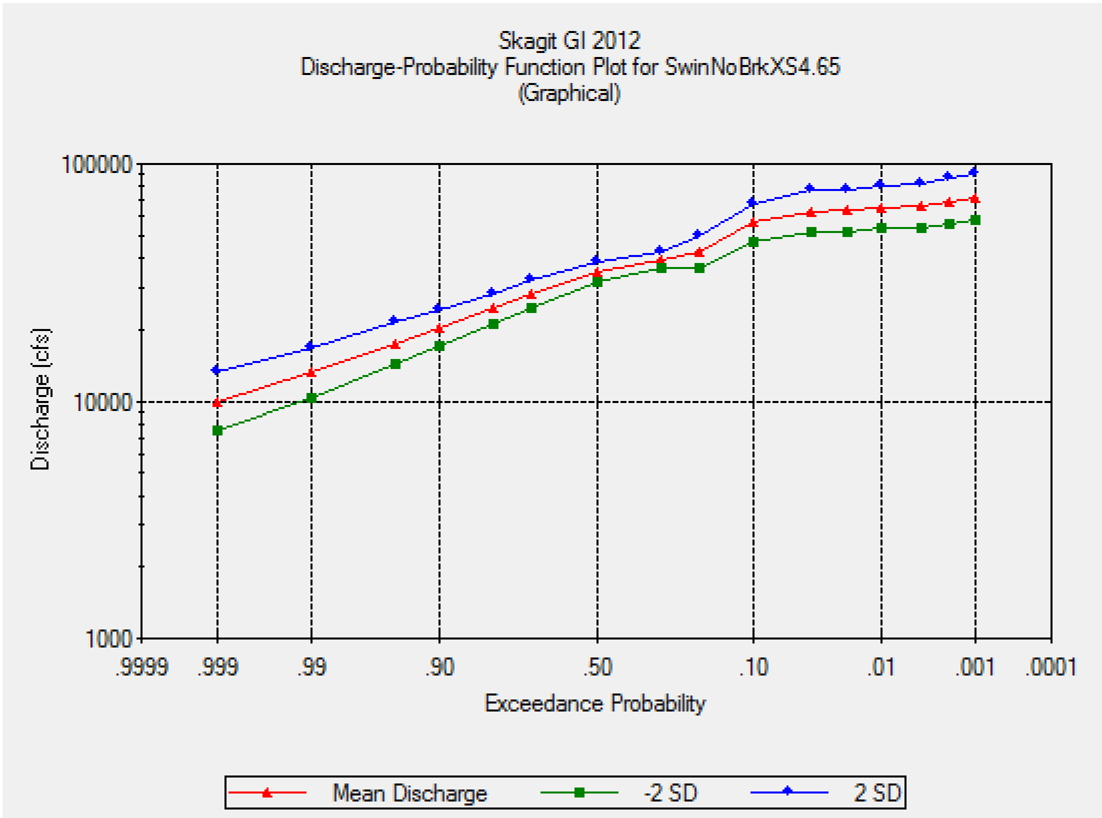

Skagit GI 2012
Discharge-Probability Function Plot for SwinNoBrk XS21.6
(Graphical)
Discharge (cfs)
1000000
100000
10000
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Skagit GI 2012
Discharge-Probability Function Plot for SwinNoBrk XS17.9
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

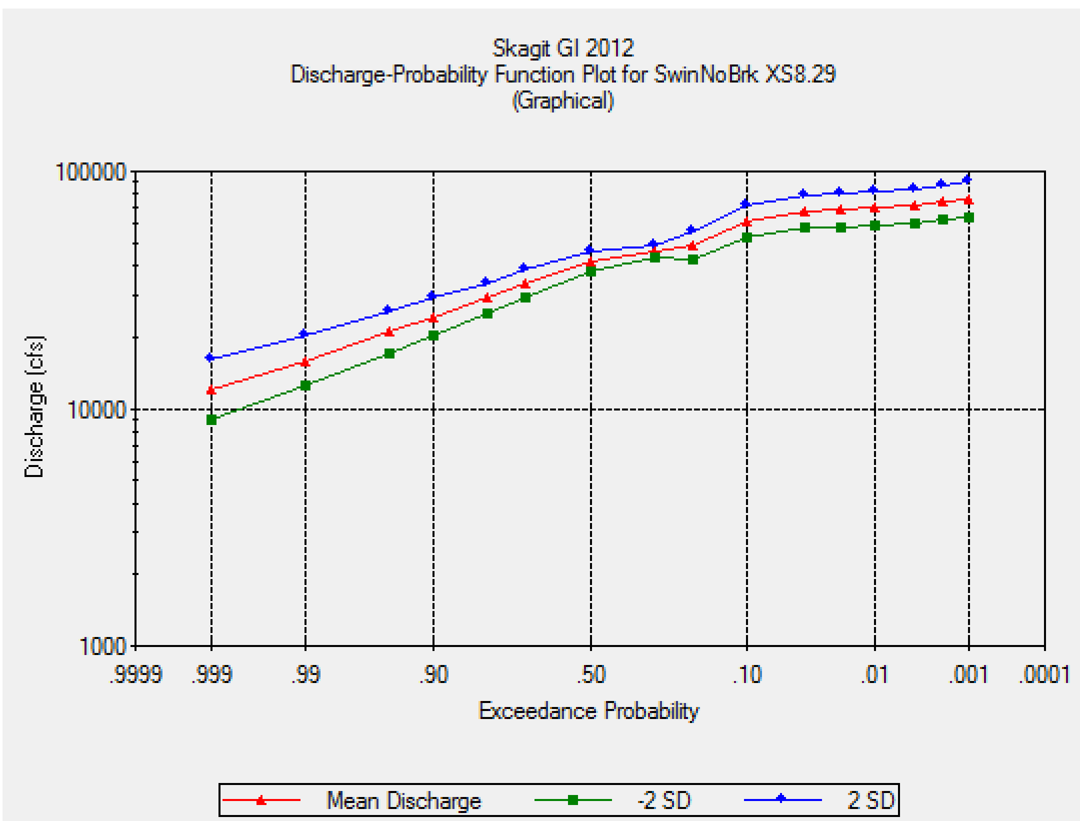

Skagit GI 2012
Discharge-Probability Function Plot for SwinNoBrkXS16.78
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD


Skagit GI 2012
Discharge-Probability Function Plot for SwinNoBrk XS13.8
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD

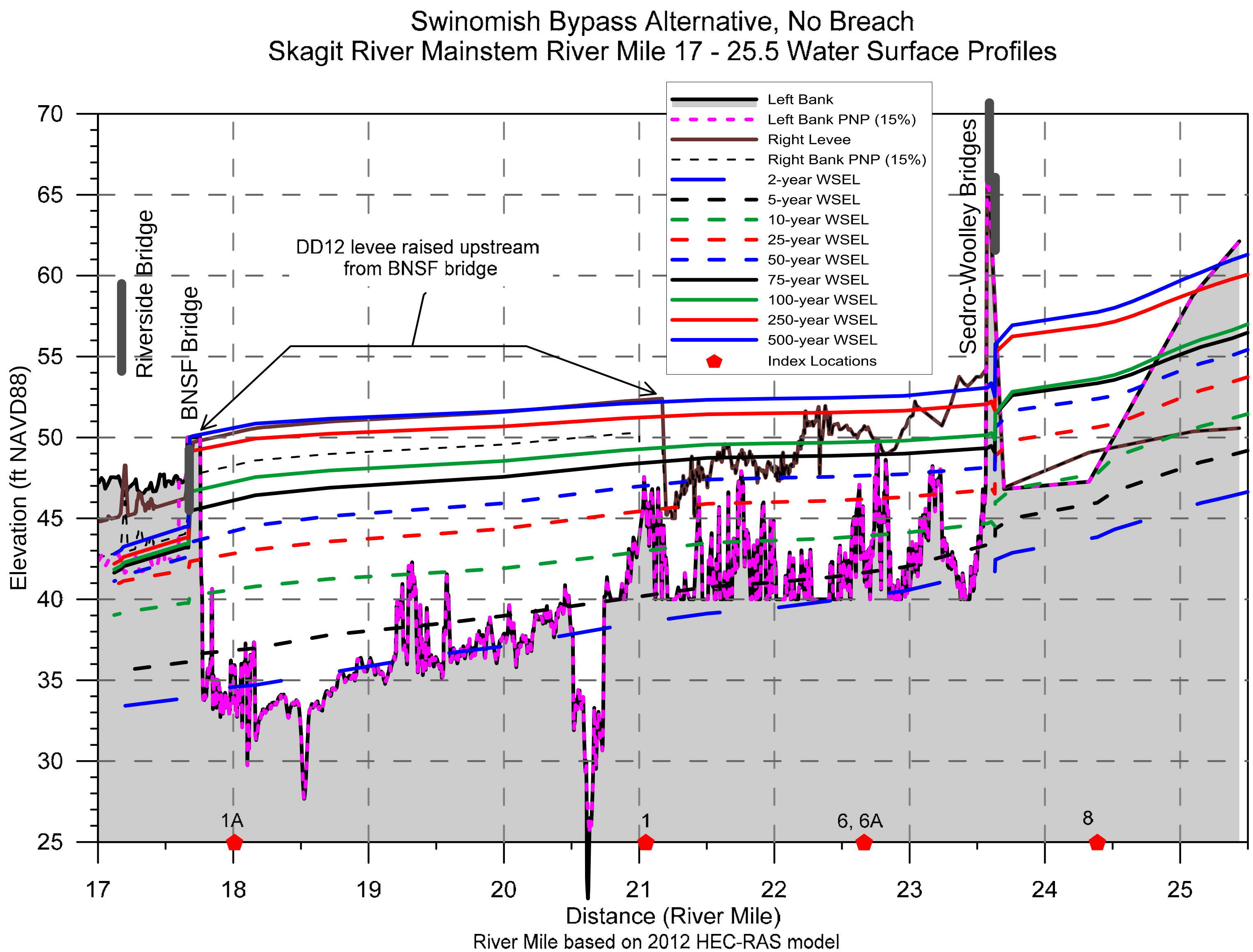

Skagit GI 2012
Discharge-Probability Function Plot for SwinNoBrk XS12.4
(Graphical)
1000000
100000
10000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD



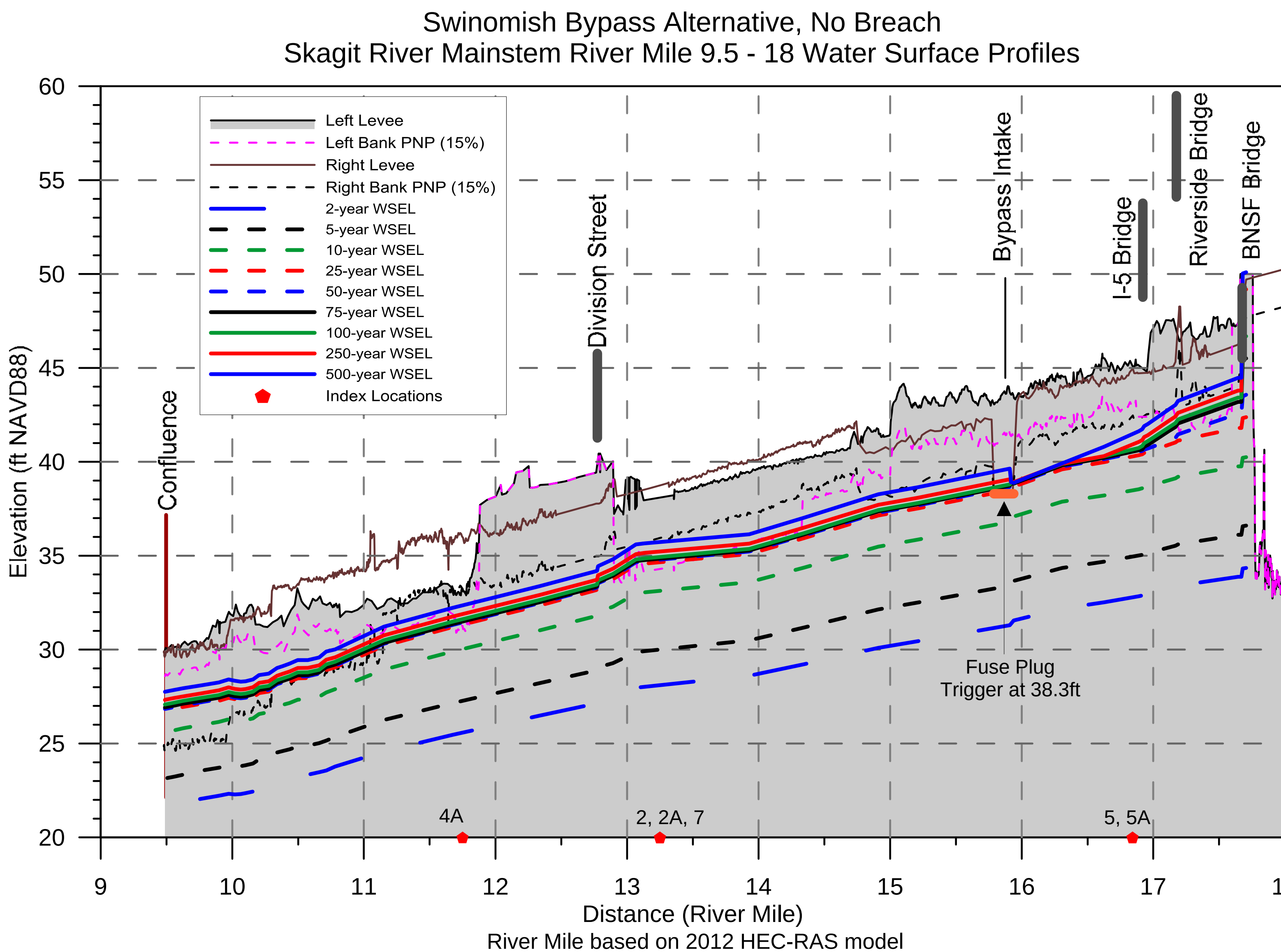
Skagit GI 2012
Discharge-Probability Function Plot for SwinNoBrkXS4.65
(Graphical)
100000
10000
1000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD



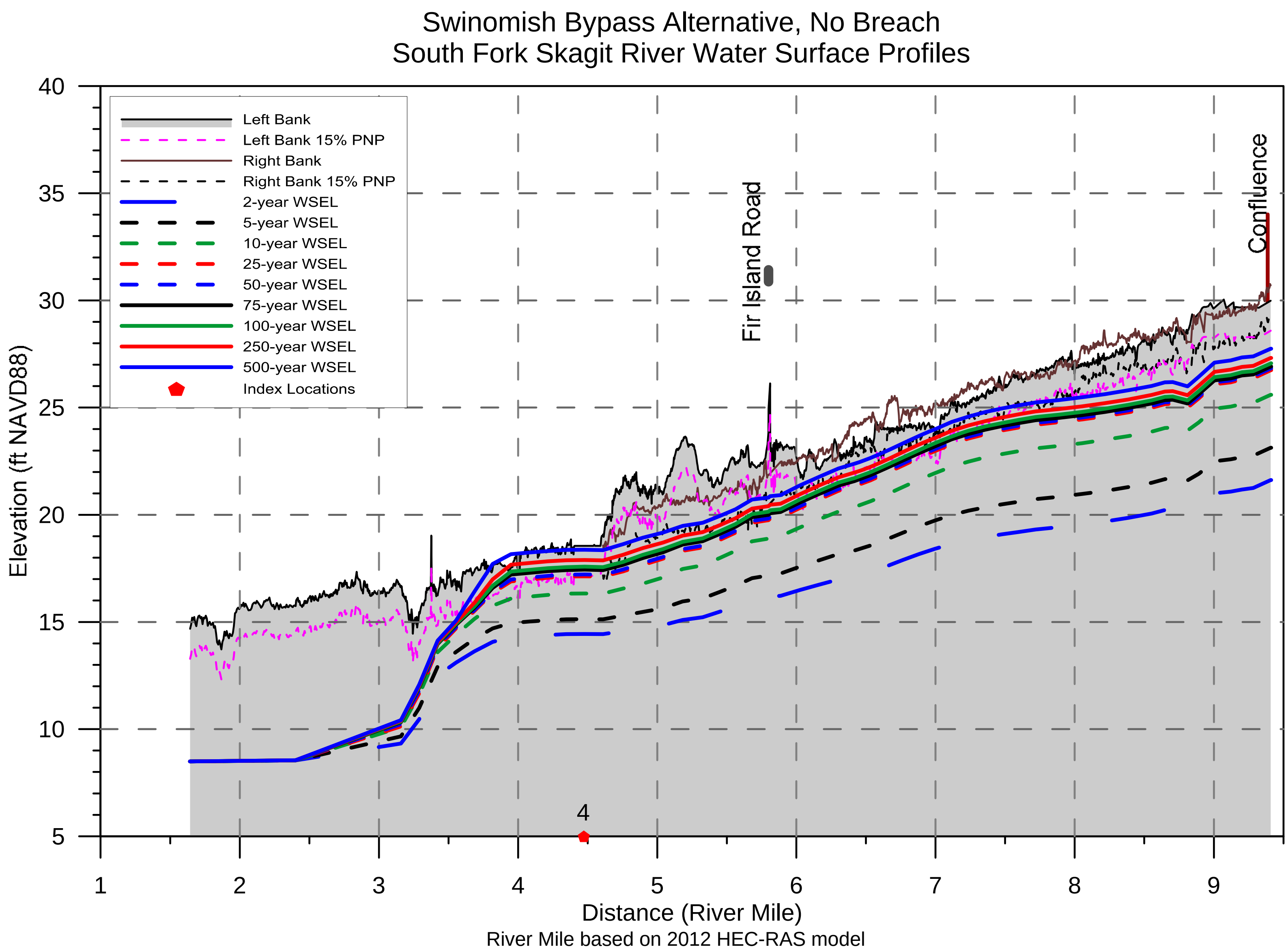
Skagit GI 2012
Discharge-Probability Function Plot for SwinNoBrk XS8.29
(Graphical)
100000
10000
1000
Discharge (cfs)
.9999
.999
.99
.90
.50
.10
.01
.001
.0001
Exceedance Probability
Mean Discharge
-2 SD
2 SD



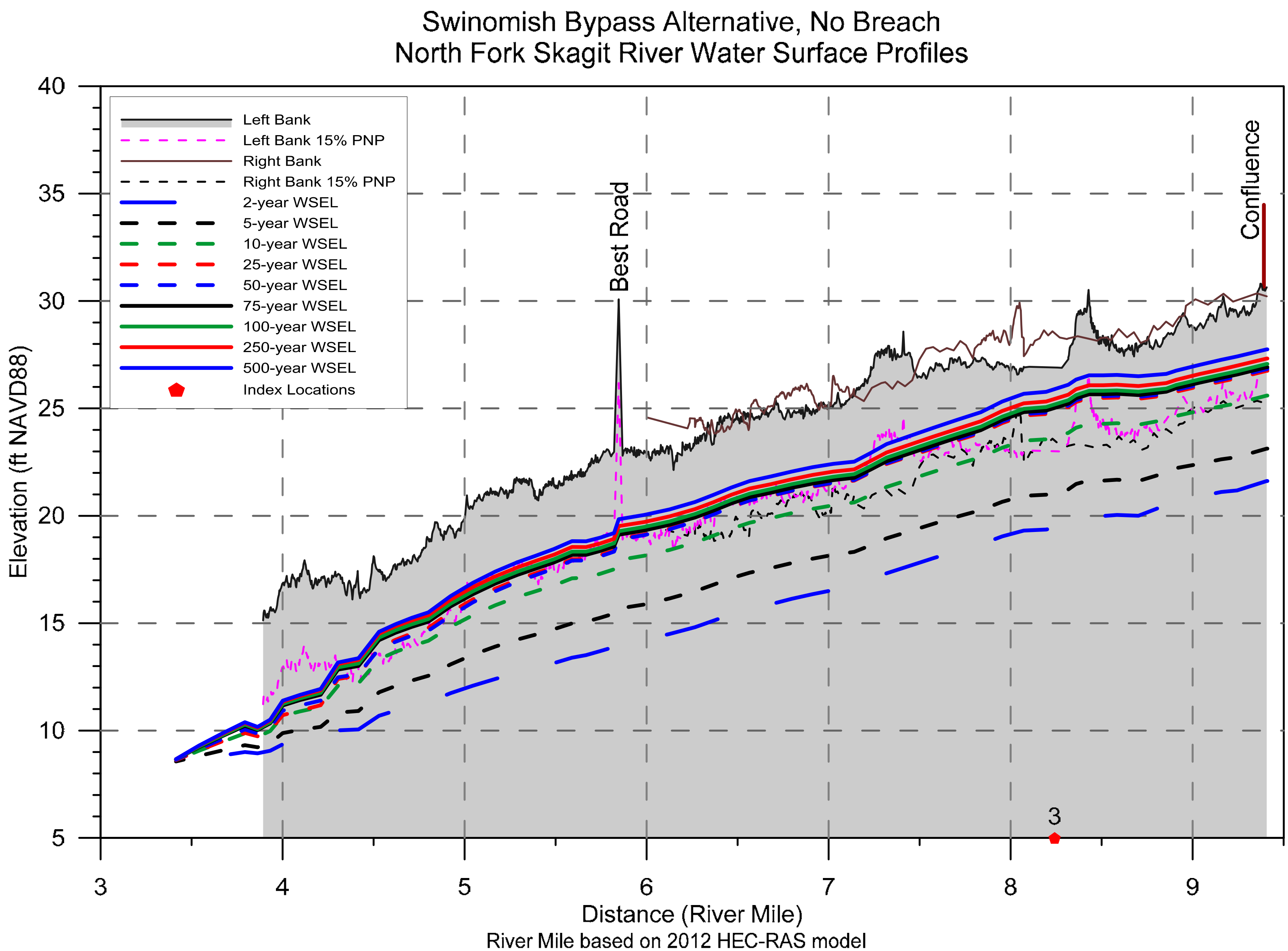
Notes:
1) Levee elevations and PNP profiles represent the "with alternative" condition.



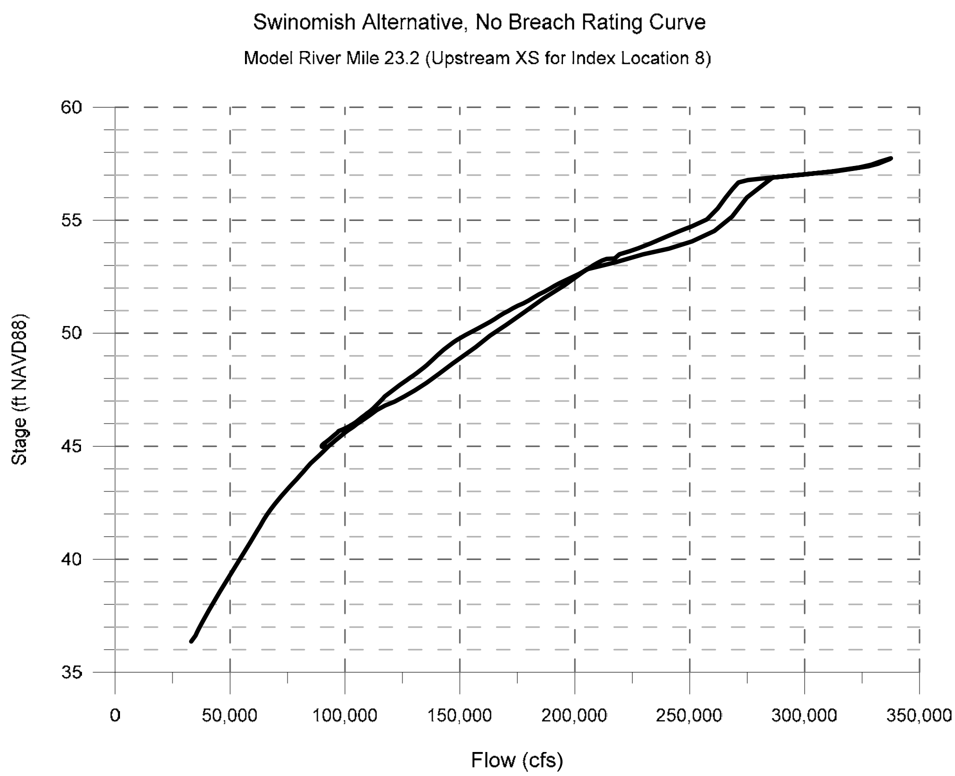
Notes:
1) Levee elevations and PNP profiles represent the "with alternative" condition.

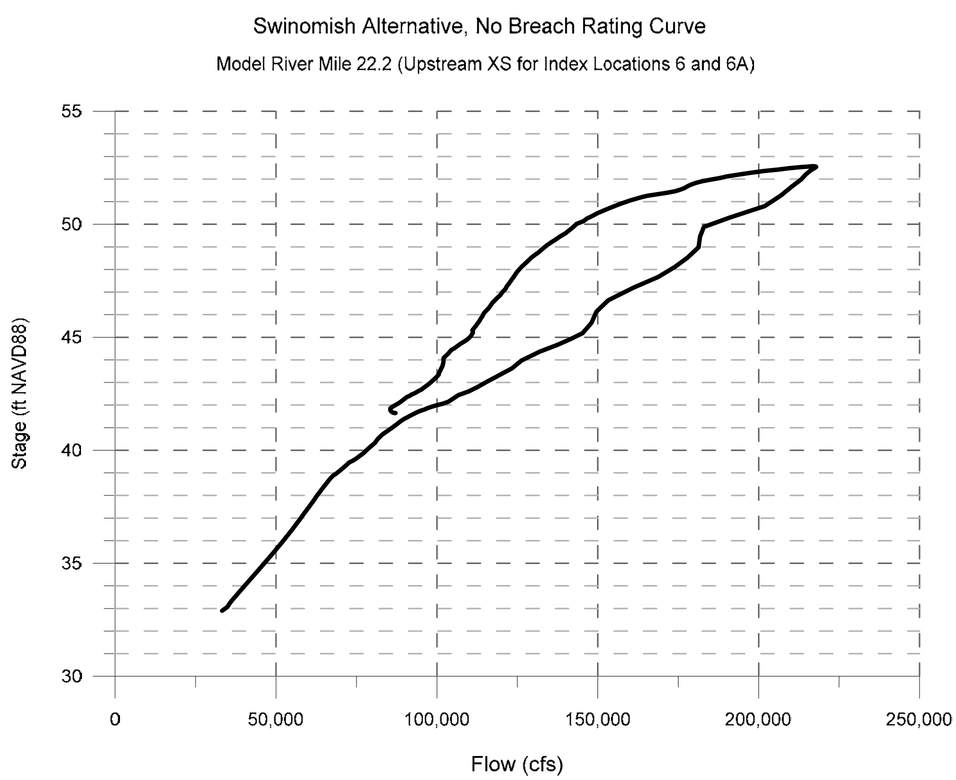


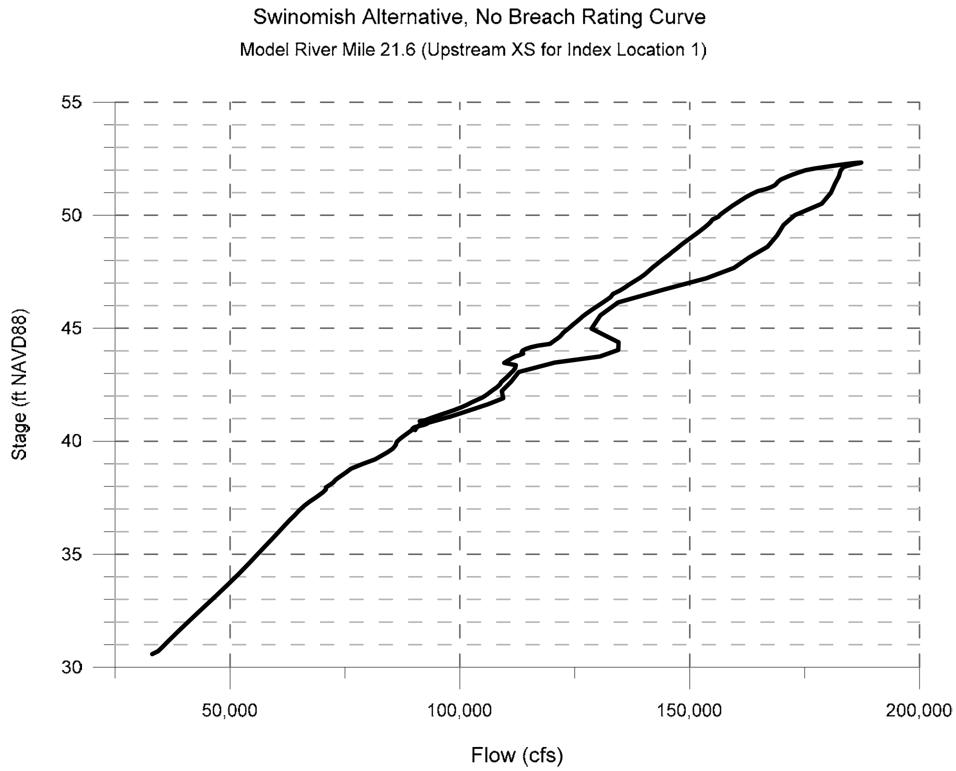
Note:
1) Levee elevation and PNP profiles represent the "with alternative" condition.

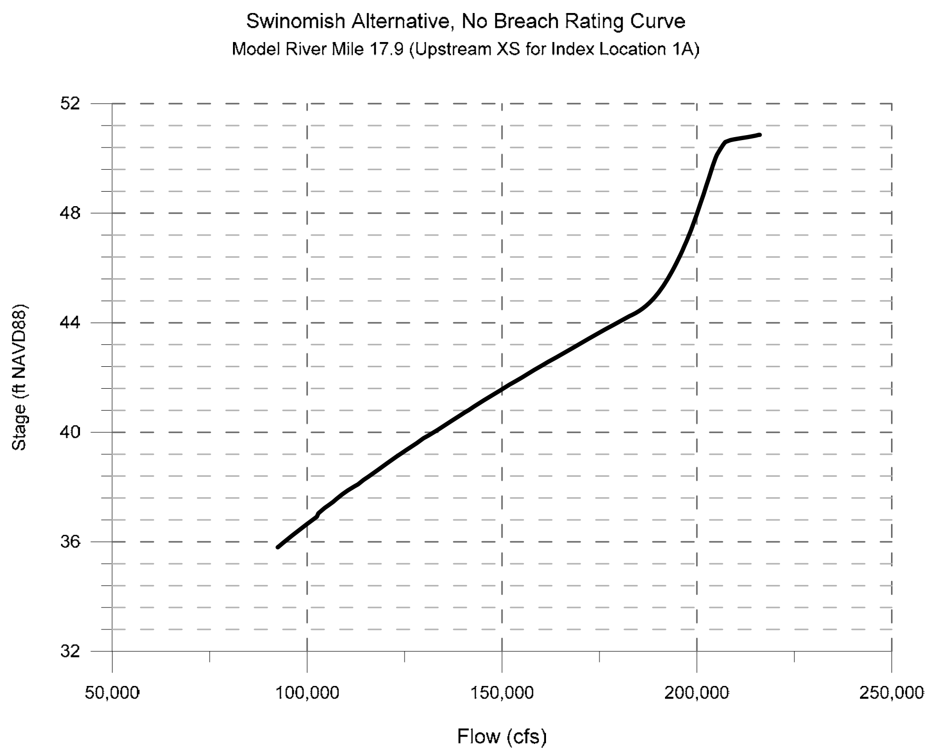


Note:
1) Levee elevation and PNP profiles represent the "with alternative" condition.


Swinomish Alternative, No Breach Rating Curve
Model River Mile 23.2 (Upstream XS for Index Location 8)
Stage (ft NAVD88)
60
55
50
45
40
35
0
50,000
100,000
150,000
200,000
250,000
300,000
350,000
Flow (cfs)


Swinomish Alternative, No Breach Rating Curve
Model River Mile 22.2 (Upstream XS for Index Locations 6 and 6A)
Stage (ft NAVD88)
55
50
45
40
35
30
0
50,000
100,000
150,000
200,000
250,000
Flow (cfs)

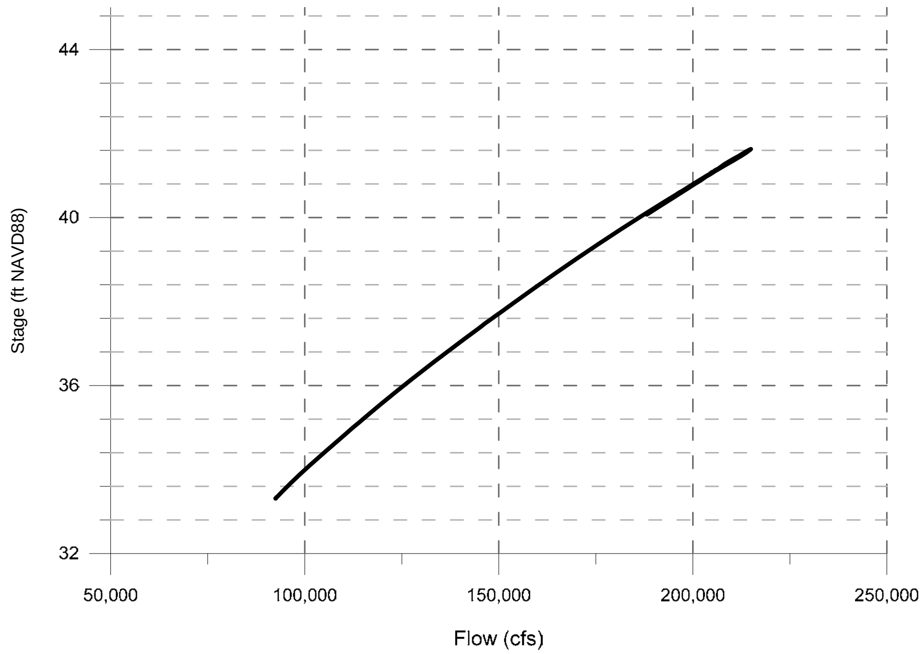




Note: Rating curve reflects hydraulic conditions post activation of the "fuse plug"

Swinomish Alternative, No Breach Rating Curve

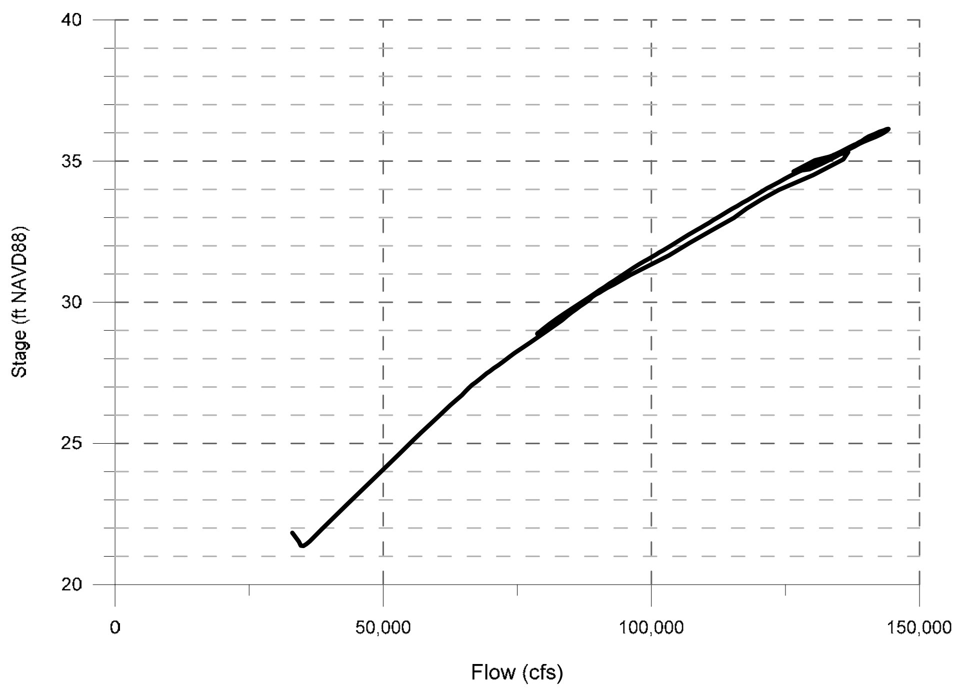
Model River Mile 16.78 (Upstream XS for Index Locations 5 and 5A)

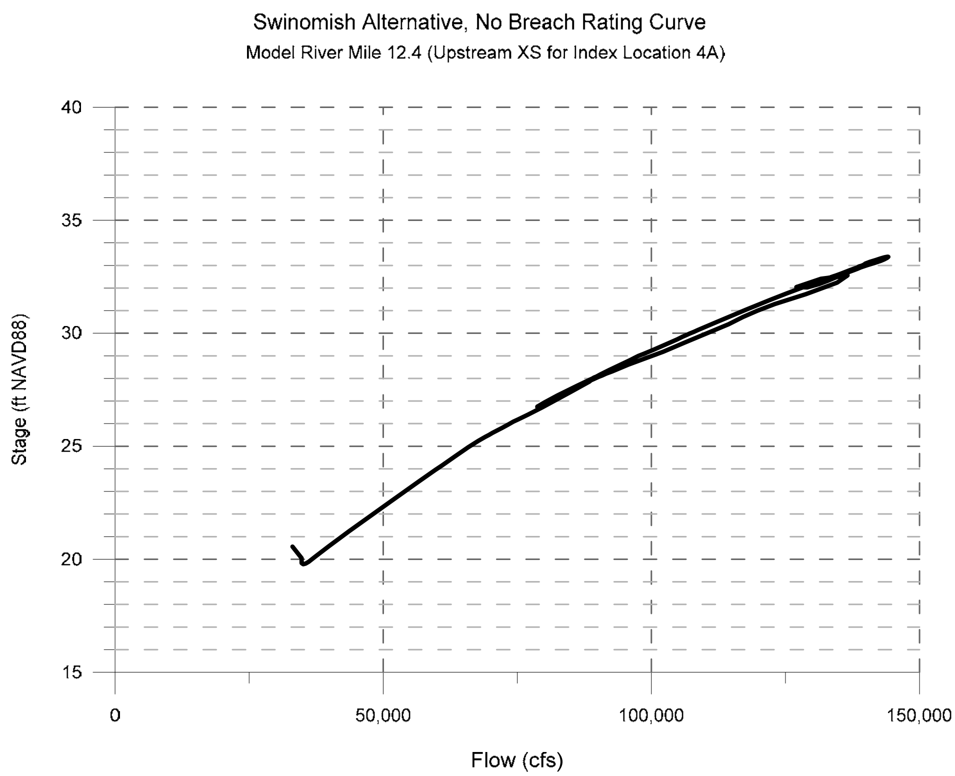


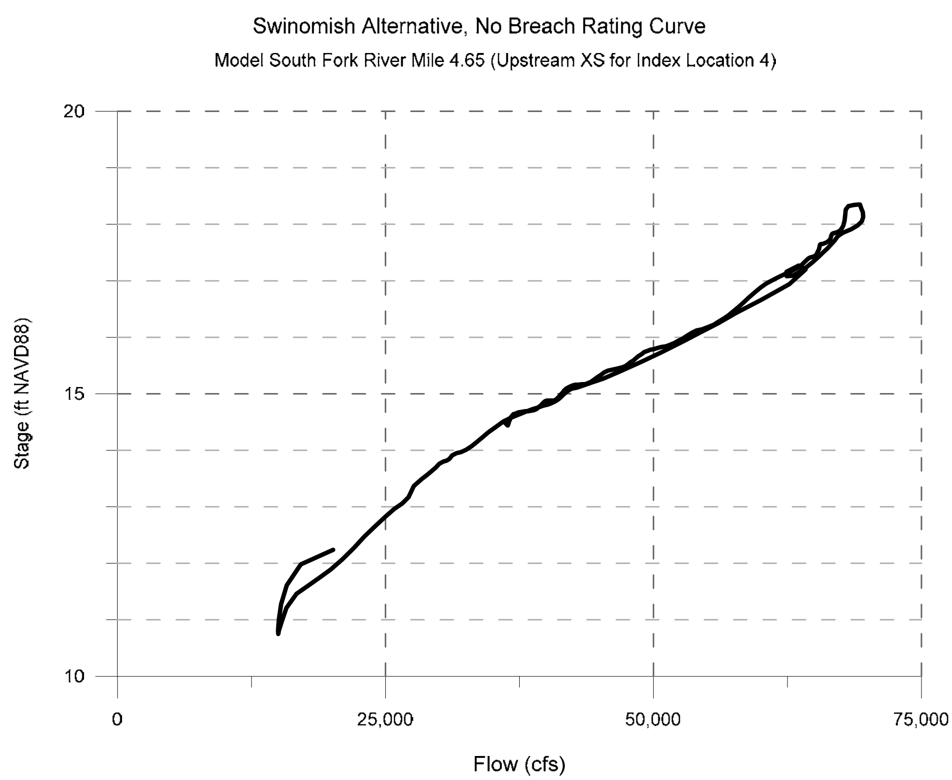
Note: Rating curve reflects hydraulic conditions post activation of the "fuse plug"

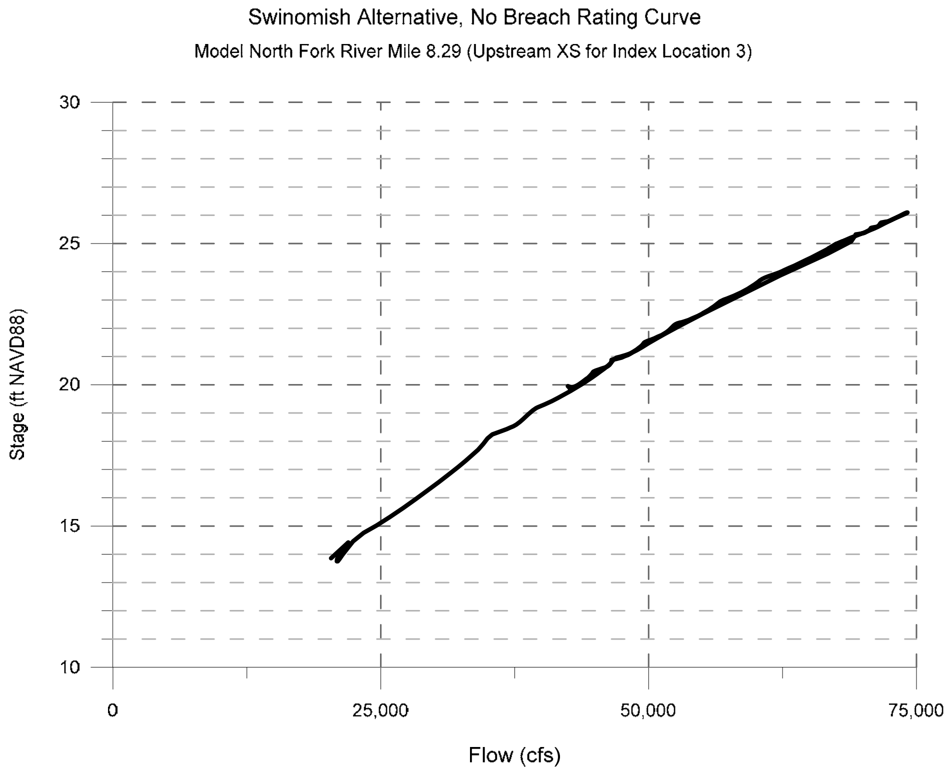
Swinomish Alternative, No Breach Rating Curve

Model River Mile 13.8 (Upstream XS for Index Locations 2, 2A, and 7)




Swinomish Alternative, No Breach Rating Curve
Model River Mile 12.4 (Upstream XS for Index Location 4A)
Stage (ft NAVD88)
40
35
30
25
20
15
0
50,000
100,000
150,000
Flow (cfs)


Swinomish Alternative, No Breach Rating Curve
Model South Fork River Mile 4.65 (Upstream XS for Index Location 4)
Stage (ft NAVD88)
20
15
10
0
25,000
50,000
75,000
Flow (cfs)


Swinomish Alternative, No Breach Rating Curve
Model North Fork River Mile 8.29 (Upstream XS for Index Location 3)
30
25
20
15
10
Stage (ft NAVD88)
0
25,000
50,000
75,000
Flow (cfs)

Skagit River Basin
Skagit River Flood Risk Management Feasibility Study

FINAL REPORT

HYDRAULIC TECHNICAL DOCUMENTATION

US Army Corps of Engineers

August 2013

HYDRAULICS TECHNICAL DOCUMENTATION
TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

APPENDICES

Skagit River Flood Risk Management Feasibility Study HYDRAULICS TECHNICAL DOCUMENTATION

1.0 Background

1.1 General

Authority for the Skagit River, Washington, flood risk management feasibility study is derived from Section 209 of the Flood Control Act of 1962 (Public Law 87-874). Section 209 authorized a comprehensive study of Puget Sound and Adjacent Waters, including tributaries such as the Skagit River, in the interest of flood risk management, navigation, and other water uses and related land resources. The current feasibility study was initiated in 1997 as an interim study under this statutory authority. Skagit County is the local sponsor of the feasibility study and is providing a combination of cash and in-kind services equaling 50 percent of the total study effort. The purpose of the study is to formulate and recommend a comprehensive flood hazard management plan for the Skagit River floodplain that will reduce flood risk at and downstream from Sedro-Woolley.

The authorization for the Skagit River Flood Risk Management Feasibility Study necessitated hydrologic and hydraulic analysis of the Skagit River basin. This allows for a basin-wide, systematic evaluation of the Skagit River. These analyses incorporate historic rainfall-runoff, reservoir operations, and flow along the major river systems to effectively evaluate the hydraulic performance of the flood management systems. The models can be used to assess the performance of the current systems or modified systems under a wide range of hydrologic conditions.

1.2 Purpose of Documentation

This report documents the work conducted for the Skagit River Flood Risk Management Feasibility Study to develop hydraulic computer models and to establish existing without-project hydraulic conditions. The main product components of this effort are:

- Description of the hydraulic analysis methodology
- Development of the hydraulic models (HEC-RAS and FLO-2D) for the Skagit River Basin
- Illustration of existing without-project conditions based on model results

Additional documentation of the hydraulic modeling conducted to provide input to economic flood damage analysis is provided in a separate Hydraulic Analysis Report (NHC 2013).

1.3 Study Area

The study area encompasses the Skagit River basin from Marblemount, Washington to Skagit Bay. It also includes the Baker River from the confluence with the Skagit to the Baker River at Concrete gage, the Sauk River from the confluence with the Skagit to the Sauk River at Sauk gage, and the Cascade River from the confluence of the Skagit to the Cascade River at Marblemount gage. The Skagit River basin has a drainage area of 3,115 square miles of which 2,737 square miles are above Concrete, Washington. The emphasis in this report is on hydraulic modeling for the lower Skagit River downstream from Sedro-Woolley. The damage reaches that are evaluated start at Sedro-Woolley and extend down to the mouth at Skagit Bay. The lower part of the study area of primary interest is illustrated in Figure 1.

1.4 Skagit River Basin

The Skagit River basin is located in the northwest corner of the State of Washington. The Skagit River basin extends about 110 miles in the north-south direction and about 90 miles in the east-west direction between the crest of the Cascade Range and Puget Sound. The northern end of the basin extends 28 miles into Canada.

The Skagit River originates in a network of narrow, precipitous mountain canyons in Canada and flows west and south into the United States where it continues l35 miles to Skagit Bay. Skagit River falls rapidly from its source at an elevation of about 8,000 ft to 1,600 ft at the United States-Canadian Border. Stream profiles on Figure 2 show that within the first 40-miles south of the International Border, the river falls a further 1,100 feet and that the remaining 500 feet of fall is distributed along the 95 miles of the lower river. The average bed slope from Concrete (at about RM 56) to the mouth is 0.045%

The Skagit Valley, the 100,000-acre valley area downstream from the town of Concrete, contains the largest residential and farming developments in the basin. The 32-mile long valley between Concrete (RM 56) and Sedro-Woolley (about RM 23) is from 1 to 3 miles wide, with mostly cattle and dairy pasture land and wooded areas. The valley walls in this section are steeply rising timbered hills.

Downstream from Sedro-Woolley, the valley descends to nearly sea level and widens to a flat, fertile floodplain and delta with an east-west width of about 11 miles and a north-south width of about 19 miles. The floodplain and delta joins the Samish River valley to the north, and extends west through Burlington and Mount Vernon to La Conner, and south to the Stillaguamish River. Between Sedro-Woolley and Mount Vernon, a large area of floodplain provides natural storage, primarily in the lower Nookachamps Creek Basin along the left overbank of the Skagit River. For very high river flows, a portion of the Skagit River in this reach can overflow the right bank and escape out of the system through Burlington to Padilla Bay and to Samish Bay. The Skagit River continues through a broad outwash plain in the lower reach nearest the river mouth and divides between two principal distributaries, the North Fork and the South Fork, which are approximately 7.3 and 8.1 miles long, respectively. About 60 percent of the discharge is carried by the North Fork and the remainder is carried by the South Fork during lower flows, but this split becomes closer to 50-50 with higher flows.

Figure 1. Lower Skagit Basin Hydraulic Model Features

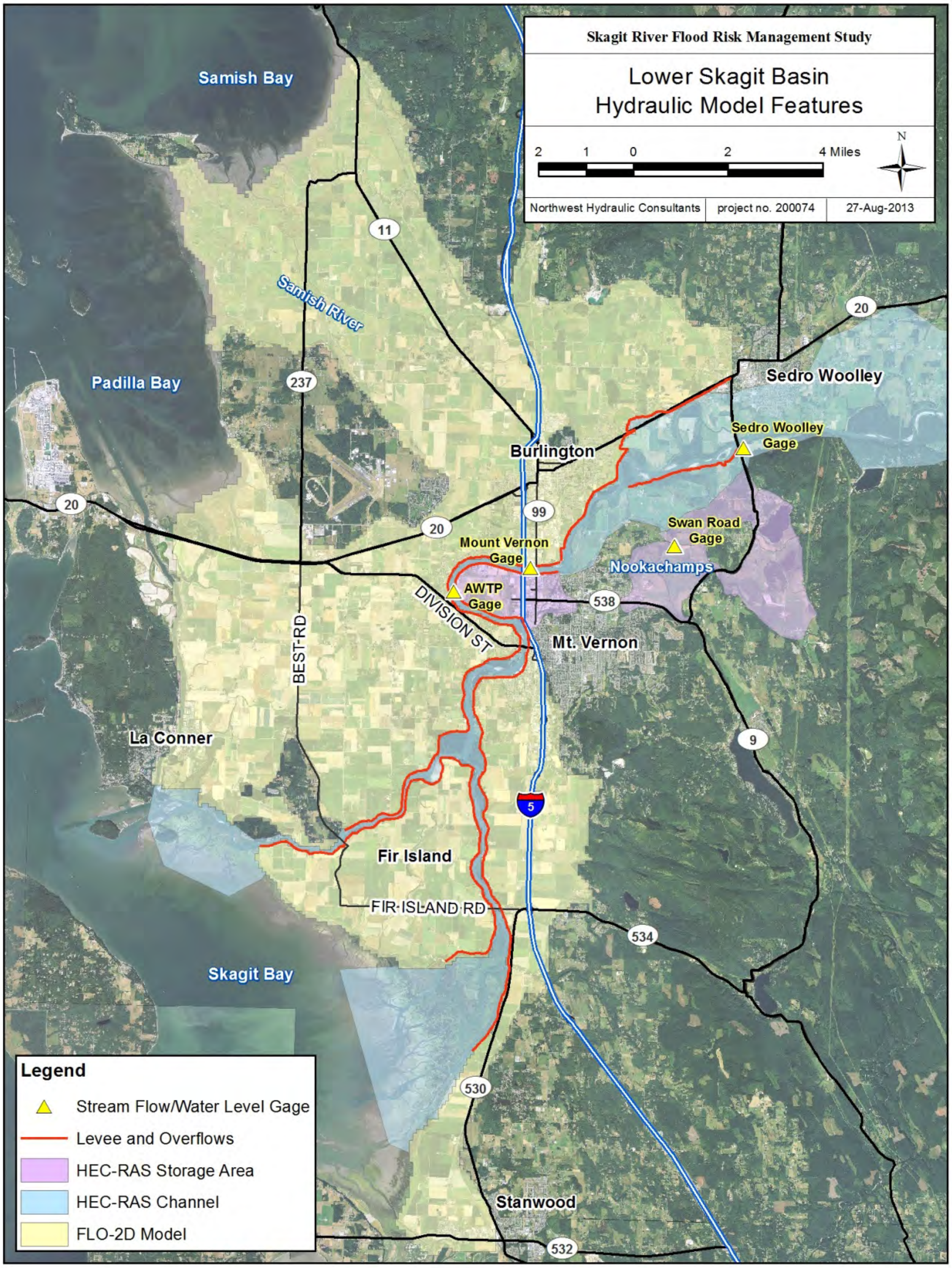
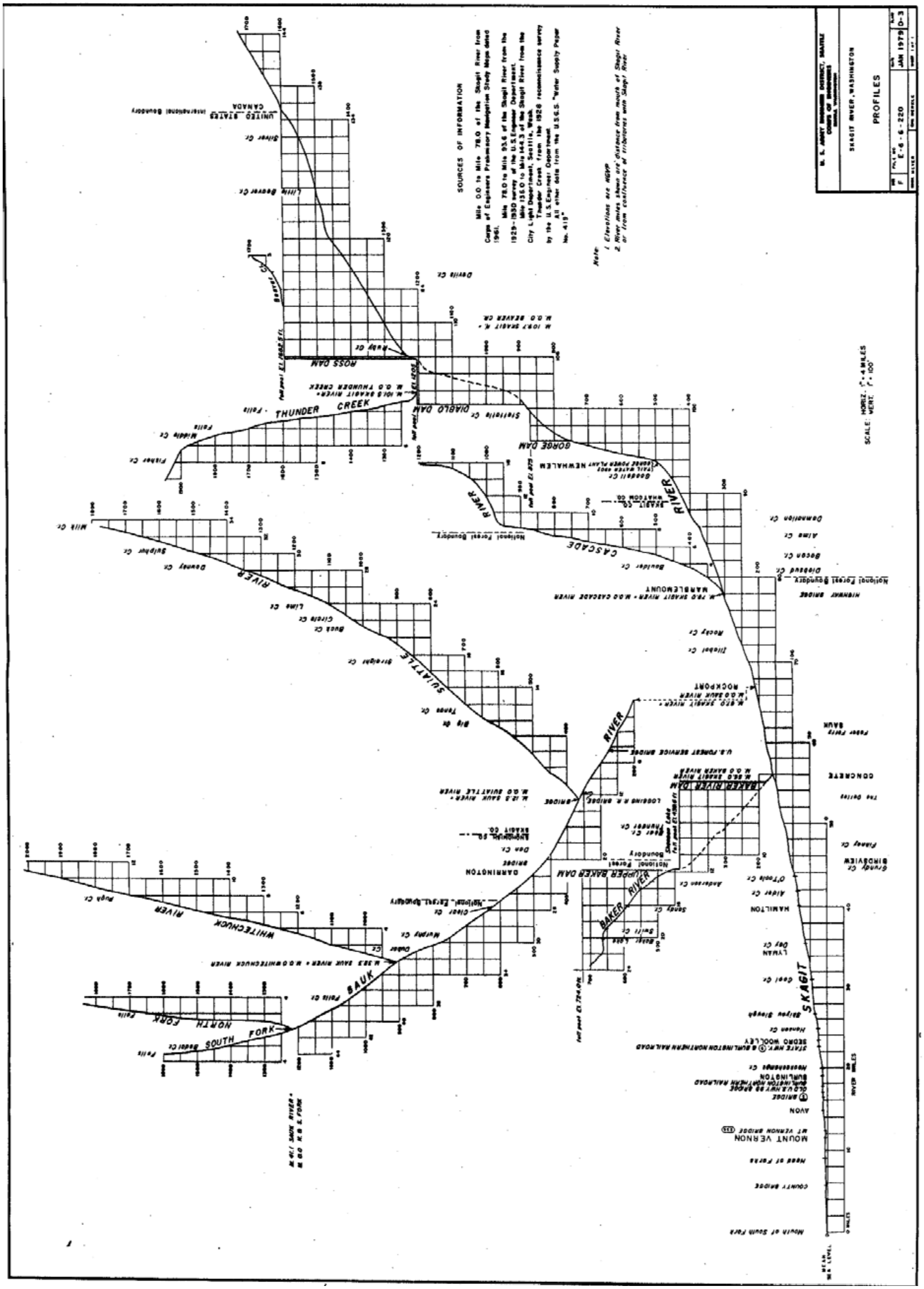


Figure 2. Skagit River and Major Tributary Stream Profiles



1.5 Study History

Hydraulic model development and hydraulic analyses for the Skagit River Flood Risk Management Feasibility Study were conducted by the Seattle District USACE in parallel with similar work by the District for preparation of an updated Flood Insurance Study (FIS) for Skagit County. Draft Hydraulic Technical Documentation for the Skagit River Flood Risk Management Feasibility Study was produced by the Seattle District in August 2004 following technical review by the Hydrologic Engineering Center (USACE 2004). Hydraulic analyses for the study were subsequently revised and updated by the District, however the Hydraulic Technical Documentation was not updated at that time. Additional hydraulic model development was also undertaken by the District for the FIS, focusing primarily on revisions to the FLO-2D model of the Skagit River floodplain. However, relevant aspects of those modifications were not carried over to the Flood Risk Management Feasibility Study.

Revisions to the hydraulic models used for the Flood Risk Management Study and preparation of a draft April 2011 update to the Hydraulic Technical Documentation were carried out by Northwest Hydraulic Consultants Inc. (NHC) under contract to the local sponsor, Skagit County (contract C20080424, Task Assignment 4, authorized 15 October 2009). Significant revisions to the circa-2004 models, made by NHC in consultation with the Seattle District, included:

- Conversion of all hydraulic models to the NAVD88 vertical datum.
- Geo-referencing of the portion of the HEC-RAS model downstream from Sedro-Woolley.
- Changes to the HEC-RAS model configuration to better represent storage in the lower Nookachamps Creek area.
- Recalibration of the HEC-RAS model for the lower basin below Sedro-Woolley and model validation against the floods of 1995 and 2006.
- Incorporation of updated levee profile and levee failure data.
- Adoption of the FLO-2D model from the 2008 draft FIS and modification of the FIS FLO-2D model to improve overall computational efficiency for the Flood Risk Management Study.
- Creation of an updated topographic basemap of the lower Skagit floodplain and incorporation of updated topographic data into the FLO-2D model.
- Updates to hydraulically significant floodplain features not incorporated into the FIS FLO-2D model.

Further revisions to the 2011 hydraulic models and preparation of the current 2013 Hydraulic Technical Documentation were carried out by NHC under contract to the Seattle District USACE (contract W912DW-11-D-1006, Task Order No.3). The principal changes to the 2011 models were as follows:

- Corrections and refinements to the HEC-RAS model representations of the Division Street, BNSF, Highway 9, and Great Northern Railroad bridges.

- Changes to the HEC-RAS model debris loading assumptions for the BNSF and Great Northern Railroad bridges.
- Modifications to the HEC-RAS model right bank lateral structure controlling spill over natural high ground upstream from the present Dike District 12 levees in the vicinity of Sterling.
- Changes (reductions) in the HEC-RAS model lateral structure weir coefficients for modeling of spill over natural high ground in the reach from Sedro-Woolley to Bellingham.
- Adjustments to HEC-RAS model overbank roughness to ensure consistent values in forested riparian areas.
- Minor changes to lateral structures to allow the model to run in HEC-RAS Version 4.1.0.
- Changes to HEC-RAS model hydrologic inputs to use regulated weighted hydrographs at Concrete which account for seasonal variation in flood control storage (see the 2013 Hydrology Technical Documentation [USACE 2013]).
- Inclusion of the Mount Vernon “flood wall” in the existing condition HEC-RAS model. At the time of writing (August 2013), the flood wall (which is a combination of concrete wall with stop logs and earthen levee) was under construction and that portion upstream from Division Street had largely been completed. The schedule for construction of the portion downstream from Division Street is not known. The entire length of the flood wall was included in the existing condition model.
- Elimination of floodplain infiltration in the FLO-2D model, consistent with observed winter conditions in the lower Skagit River floodplain.
- Corrections to the FLO-2D coding of levees in several locations to eliminate loss of water through the sea dykes.

Corrections and refinements to the HEC-RAS model representation of the BNSF bridge and its associated debris loading assumptions were developed through detailed review and investigation of the bridge hydraulic performance. The Task Report on BNSF bridge hydraulics is provided in Appendix A.

Additionally, sensitivity analyses were performed to determine whether further refinements were needed to the HEC-RAS model representation of the lower Nookachamps Creek storage area. The model represents this area as four linked storage cells. Subdivision into additional storage cells to improve modeling of the interaction between the storage area and the mainstem Skagit River was found to have very little impact on flows and water levels at key locations of interest.

1.6 Datum

The vertical datum used for hydraulic modeling in this study, for both the FLO-2D and HEC-RAS models and their output, is NAVD88. The horizontal datum is the Washington State

Plane Coordinate System North Zone, 1983/91 North American Datum. All elevations in this document are reported in feet to the NAVD88 datum unless specifically stated otherwise.

1.7 River Stationing

River stationing for the HEC-RAS models used in this study is understood to have originated from the hydraulic model created for a 1984 Flood Insurance Study. It should be noted that the model stationing reported as River Miles (RM), is inconsistent with current measured river lengths. The distance between RM 10.1, just upstream of the North and South Fork split, to RM 22.27, on the downstream side of the Highway 9 Bridge at Sedro-Woolley, is 12.17 miles based on the RM difference. However, the channel distance within the HEC-RAS model between the same two cross sections is 13.25 miles in the 2004 model and 13.42 miles in the updated, geo-referenced 2011 and current (2013) models. The difference in reach distance between the same locations in the 2004 and 2011/2013 HEC-RAS models is relatively small - around 1,000 feet (1.4%) - and easily explained by slight variations in the channel centerline selected for measurement between the two models. In contrast, the River Mile distance per the model stationing is over a mile less than calculated channel distance in both HEC-RAS models. There are no known major channel shifts, avulsions or meander cutoffs that can explain this discrepancy.

For consistency with previous work, the distributary point of the North and South Forks is set at RM 9.48 and with the exception of water surface profile plots, river miles (RM) in this report refer to the stationing as used in 2004.

Water surface profile plots of the system downstream from Sedro-Woolley provided in the report show actual distances as determined from the current (2013) geo-referenced HEC-RAS model. Thus, as an example, the model cross-section with the name RM 22.27, on the downstream side of the Highway 9 bridge, is the same cross-section in the current work as in prior work. In the profile plots, this cross-section is positioned according to measured channel distances at river mile 23.575.

2.0 Hydraulic Analysis Methodology

2.1 Model Extent

Hydraulic models developed for this study cover the Skagit River and its floodplain from Marblemount (RM 78.87) to Skagit Bay and also incorporate short reaches of major tributaries to the Skagit as noted in Section 1.3. The focus of hydraulic model development and application is on the lower part of the river and its floodplain downstream from Sedro-Woolley. The damage reaches that are to be evaluated start at Sedro-Woolley (RM 23.2) and extend down to the mouth of the Skagit River at Skagit Bay. This section describes the hydraulic analysis methodology, including the development of the HEC-RAS and FLO-2D hydraulic models, the modeling approach, and the levee failure methodology. The HEC-RAS and FLO-2D models will be used to identify existing without-project conditions and analyze the effects of various flood management measures and alternatives.

2.2 Study Approach

For this study, two numerical hydraulic models, HEC-RAS Version 4.1.0 and FLO-2D Version 2009, are utilized to represent hydraulic conditions. The steps taken to develop these models will be explained. In addition, information about the strengths, applicability, and limitations of each of these analytical tools will be presented.

The level of detail for a study of this type is always limited by the availability of geometric and topographic data, and is further constrained by limited or incomplete historical hydrologic data. Another limitation is the accuracy and applicability of the computer models used. While the models are continually being improved to better represent the river systems, no model is a perfect representation of actual riverine conditions. However, the models developed for this study are of appropriate detail to provide results for a systematic flood damage analysis of the lower Skagit River basin.

2.3 Floods Studied

For the hydraulic analysis, nine hypothetical floods with 2-, 5-, 10-, 25-, 50-, 75-, 100-, 250-, and 500-year return frequency are explicitly modeled. These flood hydrographs use the “average” case for reservoir regulation and are weighted to account for seasonal variation in reservoir flood control storage. For information on how the hydrographs are developed for input into the models, see the Hydrology Technical Documentation (USACE 2013).

2.4 Description of Hydraulic Models

Computer-based hydraulic models, such as HEC-RAS and FLO-2D, turn theoretical and empirical equations into useful analytical tools for simulating current, baseline conditions and analyzing alternative flood risk reduction scenarios. The two models are used jointly to simulate the channel and overbank hydraulics in the Skagit River system. In-channel flows and some overbank areas are simulated using HEC-RAS while the FLO-2D model is used to simulate flows in the remaining overbank areas. The HEC-RAS and FLO-2D models are interfaced through the Data Storage System (DSS) developed by the Corps of Engineers, Hydrologic Engineering Center (HEC). A map showing where the Skagit River is modeled with HEC-RAS and FLO-2D as well as locations of levees can be seen in Figure 1.

This dual model approach was selected to allow for efficient modeling of flood management measures and alternatives within HEC-RAS, while retaining the ability to model complex two-dimensional floodplain flows within FLO-2D.

2.4.1 HEC-RAS Model Development

The computer model HEC-RAS Version 4.1.0, developed by the Corps of Engineers Hydrologic Engineering Center, is used for this study. HEC-RAS is designed to simulate unsteady flow through a network of open channels, weirs, bypasses, and storage areas. For more information about the capabilities of this model, refer to the User's Manual (USACE 2010).

Two HEC-RAS models are used in the study. An upper basin model is used to route flows from the upper Skagit, Baker and Sauk Rivers, along with local tributaries, to the Skagit River near Concrete gage (RM 54.12). A lower basin HEC-RAS model is used to route flows from the Skagit River near Concrete gage down to Skagit Bay.

HEC-RAS is used to route both in-channel and floodplain flows in the upper basin model and in the lower basin model above RM 22.3 (the State Route 9 bridge). Downstream from RM 22.3, where the Skagit River enters the broad flat alluvial fan and delta, use of HEC-RAS is limited to the riverine channels and to modeling of flood storage in the lower portions of the Nookachamps Creek basin and the Riverbend area of Mount Vernon. Elsewhere, floodplain flows are modeled using FLO-2D (see Figure 1 and Section 2.4.2).

a. Purpose of Model

The purpose for using HEC-RAS in the Feasibility Study is to provide a means for understanding and representing the channel hydraulics in the Skagit River system. The upper basin model is used strictly for hydrologic routing of dam outflows, Sauk River flows and local tributary inflows to the Skagit River near Concrete gage as there are no damage reaches in the area. The lower basin model is used to determine river stage, velocity, and depth, as well as levee overtopping and levee breach flows onto the floodplain. The focus of the lower basin model is on flood behavior from Sedro-Woolley downstream.

b. Data Sources, Procedures and Process

Cross Sections

Original cross section data was developed in 1975 for the Flood Insurance Study (FIS) for Skagit County (FEMA, 1984). This data was collected by Seattle District USACE Survey Branch. Floodplain geometry for the study was obtained via aerial photogrammetry, while channel cross sections were field surveyed. All of the 52 cross sections from Concrete to Sedro-Woolley (RM 55.35 to RM 22.4) from the FIS are used for this study. In addition, 57 cross sections for the Skagit River from Marblemount to Concrete, 10 cross sections for the Cascade River, 13 cross sections on the Sauk River, and 4 cross sections on the Baker River are used from the FIS.

All of the cross sections from Sedro-Woolley to Skagit Bay were resurveyed in 1999 by Skagit County. Some of these surveys only included the underwater portions of the cross section, so some parts of the 1975 cross sections are used in this reach to provide full in-channel and overbank details. From RM 10.6 on the mainstem to XS 829 on the North Fork and XS 852.4 on the South Fork, cross sections are based on surveys completed by Northwest Hydraulic Consultants in 2010 (NHC 2011).

In the reach from the former Great Northern Railroad Bridge crossing of the Skagit River just below Sedro-Woolley (RM 22.4) to Skagit Bay, an analysis of 25 cross sections was completed by WEST Consultants, Inc. to determine the level of channel aggradation from 1975 to 1999 (WEST, 2001). Their findings showed that the majority of the stations have aggraded, and only a few have degraded. These results can be seen in Table 1. The hydraulic analyses presented in this report do not consider potential continued future aggradation and resultant increases in water surface elevation. Such changes should however be considered in future analysis of flood management alternatives.

Table 1. Skagit River Cross-Section Comparison (1975-1999)

Reach	River Station (miles)	1975 Thalweg (navd-ft)	1999 Thalweg (navd-ft)	Change in Thalweg (ft)	Average Change in Bed (ft)
Skagit R.	10.1	-13.3	-2.7	10.6	3.7
Skagit R.	10.6	-7.9	-3.6	4.3	0.9
Skagit R.	11.2	-10.2	-7.8	2.4	0.6
Skagit R.	11.7	-6.4	-1.2	5.2	1.8
Skagit R.	12.4	-4.5	-6.0	-1.5	1.5*
Skagit R.	12.9	-5.1	-1.2	3.9	1.0
Skagit R.	13.1	-18.9	-17.0	1.9	1.6
Skagit R.	13.8	1.3	1.1	-0.2	1.3*
Skagit R.	14.0	-5.6	-6.9	-1.3	2.2*
Skagit R.	15.0	0.4	-1.8	-2.2	0.1*
Skagit R.	15.1	-1.2	2.1	3.3	2.3
Skagit R.	15.9	-3.9	-2.3	1.6	2.6
Skagit R.	16.2	6.2	8.2	2.0	0.2
Skagit R.	16.6	5.0	7.4	2.4	2.4
Skagit R.	16.8	5.1	7.2	2.1	2.2
Skagit R.	17.0	8.7	7.7	-1.0	-1.5
Skagit R.**	17.5	-12.1	-10.4	1.7	-6.0
Skagit R.	17.9	2.3	6.5	4.2	2.0
Skagit R.	18.5	6.7	9.9	3.2	1.2
Skagit R.	19.4	5.2	8.0	2.8	2.4
Skagit R.	20.0	2.0	1.3	-0.7	2.7*
Skagit R.**	20.9	2.9	7.1	4.2	4.0
Skagit R.**	21.6	12.9	11.8	-1.1	1.9
Skagit R.**	21.9	11.4	9.8	-1.6	2.4
Skagit R.**	22.4	18.7	12.7	-6.0	-2.8
Average* **				**2.2**	**1.5**

Table 1. (continued)

Reach	River Station (miles)	1975 Thalweg (feet)	1999 Thalweg (feet)	Change in Thalweg (feet)	Average Change in Bed (feet)
SF Skagit R.	5.80	-7.9	-7.6	0.3	1.8
SF Skagit R.	6.30	-1.4	-1.8	-0.4	0.9*
SF Skagit R.**	6.95	-4.4	0.0	4.4	0.1
SF Skagit R.	7.80	1.2	0.7	-0.5	0.5*
SF Skagit R.	8.75	-9.2	-7.3	1.9	1.4
SF Skagit R.	9.25	-12.9	-15.3	-2.4	0.4*
Average* **				**-0.2**	**1.0**
NF Skagit R.	4.50	-7.9	-6.3	1.6	2.3
NF Skagit R.	4.75	-13.8	-9.6	4.2	2.8
NF Skagit R.	5.50	-5.4	-2.4	3.0	2.6
NF Skagit R.	6.20	-13.7	-3.3	10.4	1.1
NF Skagit R.	6.60	-4.9	-1.9	3.0	1.9
NF Skagit R.	7.20	-9.2	-8.7	0.5	0.8
NF Skagit R.**	7.33	-13.6	-9.7	3.9	2.9
NF Skagit R.	7.90	-7.8	-5.4	2.4	1.3
NF Skagit R.	8.10	-9.8	-7.3	2.5	1.1
NF Skagit R.	8.29	-8.8	-12.5	-3.7	-0.7
NF Skagit R.	8.85	-8.0	-5.8	2.2	2.3
Average* **				**2.6**	**1.6**

* Average section change and thalweg change are different (suggests lateral migration).
** Cross-sections are questionable, they do not appear to be surveyed at the same locations.
*** Does not include cross sections that are questionable.

Overbank and channel distances between cross sections upstream from Sedro-Woolley were assigned by scaling the linear channel and overbank distances between sections on a topographic map. From Sedro-Woolley downstream, the HEC-RAS model was geo-referenced using available GIS data with all measurements being developed within the GIS environment.

Overbank resistance factors are estimated based on engineering judgment from field assessment of the river and from interpretation of aerial photographs. In-channel resistance factors are based on model calibration for observed floods (see Section 3.0). Channel resistance factors of 0.030 to 0.035 are typical, while overbank resistance factors of 0.05 to 0.12 are assigned based on judgment, dependant primarily on land use, land cover, topography, and historic and expected depth of flooding.

<u>Storage Areas</u>
Storage Areas are used to simulate areas with significant potential for storage of flood waters with minimal flood conveyance. Storage areas are used to define portions of the lower Nookachamps Creek basin, North Mount Vernon and Riverbend. Storage areas are connected to the main river channel and other storage areas within HEC-RAS using lateral

structures. Embankment elevations and Stage-Volume tables were developed for each storage area by delineating each storage area boundary in GIS and calculating the volume using the ground surface topographic grid. The topographic grid used for this work is further described in Section 2.4.2

Bridges

The bridges in the lower Skagit River system modeled in this study are listed in Table 2. Information regarding bridge geometry, size, and other parameters included in the HEC-RAS model are obtained from bridge as-built drawings, field investigations, and photographs.

Table 2. Modeled Bridges on the Lower Skagit River

Bridge Name	Skagit River Mile
Great Northern RR	22.4
State Route 9	22.3
Burlington Northern RR	17.54
Riverside Drive	17.06
I-5	16.8
Division St.	12.94
South Fork	5.7 on SF
North Fork	5.75 on NF

Supplemental bridge data was field surveyed in 1998 by the Seattle District USACE Survey Section for the State Route 9 (SR-9) crossing at Sedro-Woolley, while bridge data (station, elevation, and distance to adjacent cross sections) for the former Great Northern Railroad bridge just upstream of the SR-9 crossing was estimated from field measurements, photographs, USGS topographic maps, and profile point data. Bridge data from HEC-RAS models developed by Pacific International Engineering were used where it was apparent that the information was more detailed. The Riverside Drive bridge was replaced in 2004 and the new bridge geometry incorporated into the current HEC-RAS model.

Further corrections and refinement to bridge geometry data were made for the 2013 modeling for the Division Street, BNSF, Highway 9, and former Great Northern Railroad bridges. These changes were based on various information sources including: limited field survey (for a portion of the BNSF bridge only), bridge as-built drawings (Division Street), photographs, and Lidar data.

Bridge Debris

The former Great Northern Railroad bridge at Sedro-Woolley and BNSF bridge between Mount Vernon and Burlington exhibit chronic debris entrapment behavior of large enough magnitude to affect flood hydraulics. A two-class bridge debris loading scenario was developed for the 2011 hydraulic modeling, with class defined by flood magnitude. Following detailed review and investigation of the BNSF bridge hydraulic performance (see Appendix A) the two-class of debris loads used in 2011 was dropped, and a single debris load was adopted for all floods, with a 6,000 sq. ft. debris blockage being applied to the BNSF bridge and 4,000 sq. ft. to the Great Northern bridge[1]. The debris dimensions were based on a review of flood photographs, discussions with county and dike district personnel involved in debris management who have experience of flood and debris conditions over many decades, and consideration of scour potential at the bridges which will offset the loss of conveyance area due to debris blockage. The BNSF bridge debris loading condition in particular is based on conditions observed during the November 29, 1995 flood. The County operates an annual debris removal program to ensure that debris is not lodged on any bridges prior to flood season. BNSF contractors routinely operate during floods using boats to dislodge debris from the railway bridge. Nevertheless it is assumed, based on past experience, that in-flood debris removal efforts, particularly for large floods, are only partially effective. Approximate debris loading dimensions assumed for the Great Northern and BNSF Railroad bridges are listed in Table 3. Other bridges were assumed to be free from debris.

Table 3. Debris Blockage Dimensions for the BNSF and Great Northern Railroad Bridges

Bridge	Total Width of Blockage (ft)	Depth of Blockage (ft)	Model Center Station of Blockage (ft)
BNSF	280	21.5	500
Great Northern	300	13.5	2900

<u>Levees</u>

The extent of levees in the Skagit River system is shown in Figure 1. Levee crest elevations were obtained from a variety of sources. Primary sources included: a recent survey by Woolpert, Inc (2010). for the Corps of Engineers; a 2004 survey by Skagit County; and a 2009 survey supplied by the City of Burlington. Elevations for Highway 20 in the Sterling area, where there are no formal levees but where flows can overtop, were extracted from 2009 aerial photogrammetry flown for the city of Burlington. For purposes of hydraulic model calibration, left bank overtopping elevations into downtown Mount Vernon were extracted from 2009 aerial photogrammetry flown for the city of Mount Vernon. For existing condition modeling (as opposed to model calibration), it was assumed that the left bank Mount Vernon “flood wall” was in place. The “flood wall”, comprising a combination of concrete wall and levee, extends 8,600 ft. from approximately RM 13.0 (upstream from Division Street) to RM 11.8, wrapping around the downstream end of the Mount Vernon Wastewater Treatment Plant. The flood wall geometry (alignment and crest elevations) was taken from plans by Pacific International Engineering dated 01/30/2009. At the time of writing (March 2013), that portion of the flood wall upstream from Division Street had largely been completed. The schedule for construction of the portion downstream from Division Street is not known.

[1] Note that Appendix A recommends that a 3,000 sq. ft. debris blockage be assumed at the BNSF bridge. Following review by the Seattle District, a 6,000 sq. ft. blockage was adopted for hydraulic modeling purposes for the Skagit River Flood Risk Management Feasibility Study.

Information about the integrity of the levees in the Skagit River system was obtained from geotechnical engineers from the Seattle District USACE, as discussed further in Section 2.4.1d. Flow exiting the channel in the HEC-RAS model, either due to levee overtopping or levee breaches, is assumed to freely leave the channel system with no backwater effects. These flows are recorded during the HEC-RAS model simulations within the DSS and subsequently used as inputs to the FLO-2D floodplain model described in Section 2.4.2.

<u>Diversion/Impoundment Structures</u>
No diversions or impoundment structures are modeled from Marblemount to the Mouth. The upper basin dams are upstream of the HEC-RAS model and their effects on the regulation of flood hydrographs are accounted for in the hydrologic analysis described in the Hydrology Technical Documentation (USACE 2013).

c. Boundary Conditions

The four primary types of boundary conditions in HEC-RAS are interior, internal, upstream, and downstream. Interior boundary conditions define reach connections and ensure continuity of flow. Internal boundary conditions are coded in HEC-RAS to represent levee overtopping and failures, storage area interactions, spillways or weir overflow/diversion structures, and bridge or culvert hydraulics.

Upstream boundary conditions are required for all reaches that are not connected to another reach at their upstream end. An upstream boundary condition is a flow hydrograph of discharge vs. time for a particular flood event.

For the upper basin model, upstream hydrographs are developed for the Skagit River at Marblemount, Cascade River at Marblemount, Sauk River at Sauk, and Baker River at Concrete (for methodology, refer to the Hydrology Technical Documentation [USACE 2013]). The flow at the Skagit River near Concrete gage resulting from the routing of these inflows, in addition to local tributary inflows, then forms the upstream boundary condition for the lower basin model.

Downstream boundary conditions are required at the downstream end of all river systems not connected to another reach or river. The downstream boundary condition for the upper basin model is the USGS rating curve for the Skagit River near Concrete gage (USGS gage 12194000). For the lower basin model, the downstream boundary condition for both the North and South Forks of the Skagit River is a tidal stage hydrograph, which has a primary peak at the Mean Higher High Water (8.39 feet NAVD88), a secondary peak at the Mean High Water (7.49 feet NAVD88), and a low at the Mean Low Water. The length of the flood hydrograph is substantially longer than the tidal cycle and during floods the extent of tidal influence is limited to only the lower few miles of each fork. Therefore the magnitude and timing of the highs and lows in the tidal hydrograph does not affect river hydraulics in any substantive way. Various sensitivity runs were performed confirming this.

Local tributary inflows are distributed evenly from Marblemount to Concrete for the upper basin model and from Concrete to Sedro-Woolley for the lower basin model. Nookachamps Creek is entered into the system as a lateral inflow to the Nookachamps storage areas (see Hydrology Technical Documentation [USACE 2013]) for a description of the derivation of these flows).

d. Uncertainty Analysis

Risk-based analysis requires estimation of the uncertainty in hydraulic model outputs, specifically in stage for a given flow, and the probability of levee failure.

<u>Channel Roughness</u>

Stage uncertainty due to uncertainty in channel roughness was determined by varying Manning's "n" values by +/- 30% from the calibrated model values with fixed debris loads of 6,000 sq. ft. and 4,000 sq. ft. on the BNSF and Great Northern railway bridges respectively. Additional discussion and analysis of stage uncertainty due to uncertainty in channel roughness is provided in the Hydraulic Analysis Report (NHC 2013).

<u>Bridge Debris</u>

Stage uncertainty due to bridge debris loading was determined by varying the debris blockage on the BNSF bridge from zero to 10,000 sq. ft. as representing reasonable upper and lower bounds on blockage. Uncertainty in debris load on the Great Northern bridge was not modeled. The low chord of the Great Northern bridge is above the estimated 500-year water level and there is significant conveyance capacity on the right bank floodplain such that the impact of debris blockage uncertainty is small and quite localized. Additional discussion and analysis of stage uncertainty due to uncertainty in bridge debris loading is provided in the Hydraulic Analysis Report (NHC 2013).

<u>Overall Stage Uncertainty</u>

Overall stage uncertainty under existing conditions for inclusion in the HEC-FDA model was calculated by taking the larger of the channel roughness and bridge debris loading uncertainties at each index point location (see the Hydraulic Analysis Report [NHC 2013] for additional discussion).

<u>Levee Breach Methodology</u>

A levee breach methodology was devised to determine when simulated flows would cause levees to fail and flow to enter a floodplain. To determine when and at what recurrence interval a levee would fail, a Probable Failure Point/Probable Non-Failure Point (PFP/PNP) analysis of the levee system was conducted by Seattle District geotechnical engineers. The PFP is defined as the in-channel water surface elevation (WSEL) at which there would be an 85% probability of levee failure. The PNP is defined at the in-channel WSEL at which there is a 15% probability of levee failure. A Likely Failure Point (LFP) is also defined at which there is a 50% probability of levee failure. For the present study, the LFP is taken to be midway between the PFP and the PNP.

PFP/PNP elevations were determined by the Seattle District at nine locations along the lower Skagit River. Analyses at eight of those locations were based on borings (2 borings per location) and geotechnical investigation undertaken by Shannon & Wilson Inc. (2011) under contract to the Seattle District. The locations for the borings were selected in consultation with local diking districts as being those most prone to failure. The ninth location was a known low point in the Dike District 12 levee system on the right bank of the Skagit River in Burlington. Geotechnical investigation by Golder Associates (2009) showed overtopping as being the most likely probable failure mode at this location, implying PFP/PNP elevations at the levee crest elevation, however PFP/PNP elevations were subsequently set below the levee crest elevation following review by the Seattle District prior to the 2013 hydraulic modeling.

Based on review of the Shannon & Wilson data and historic geotechnical data from previous investigations, the Seattle District assumed that each set of borings (eight sets of two) was representative of levee conditions over a specified reach of the river (Fischer 2010). To determine the PFPs and PNPs at any location within a specific reach, it was initially assumed that the distance from levee crest to PFP or PNP was the same as at the representative boring location for that reach. Further review and refinement of the PFPs and PNPs was conducted by the Seattle District prior to the 2013 hydraulic modeling, resulting in adjustments to the PFPs and PNPs for the mainstem right bank Dike District 12 levees (i.e. upstream from and including the model lateral structure at RM 14.59). The PFPs, LFPs and PNPs estimated for each lateral structure (i.e. levee segment) within the HEC-RAS model are listed in Table 4. Note that the data in Table 4 for the left bank mainstem Skagit River lateral structures at RM 13.049 and 12.39 are for points upstream and downstream from the Mount Vernon flood wall. For hydraulic modeling purposes, it is assumed that there is a very low risk of failure of the flood wall itself.

The HEC-RAS model makes its determination of when a levee fails using the water surface elevation at the user-specified failure station along the lateral structure. Levee failure occurs in HEC-RAS when the water surface elevation reaches a user-specified failure elevation for the given lateral structure, as discussed in more detail in the Hydraulic Analysis Report (NHC 2013). Levee failure is simulated by HEC-RAS as a levee breach. Flow through a levee breach is then routed into floodplain storage areas in HEC-RAS or saved to a DSS for input to the FLO-2D model.

The detailed embankment failure methods in HEC-RAS can simulate an enlarging breach corresponding to either a piping or overtopping failure. For simplicity, the Skagit River model uses overtopping failure algorithms to model breach enlargement for all levee failures. The breach starts when the failure elevation is exceeded, and is assumed to enlarge at a linear rate. Flow through an overtopping breach is given by a weir equation. Levee breach widths were determined through consultation with the USACE Seattle District. A maximum breach width of 300 feet was assumed for floods with a return period less than 100-years, and 400 feet for floods with a return period of 100-years and greater. All breaches are modeled to reach their maximum width within 3 hours of breach initiation. The levees are assumed to fail down to the existing floodplain ground level at the landward toe of the levee.

The development of specific levee failure scenarios and determination of floodplain inundation due to levee failure is discussed in the Hydraulic Analysis Report (NHC 2013).

Table 4. Levee Failure Points and Lateral Structures in HEC-RAS Model

MAINSTEM OF SKAGIT RIVER: LEFT BANK						MAINSTEM OF SKAGIT RIVER: RIGHT BANK					
Lateral Structure Station	Boring	Overtop Elevation	PFP (85%) Elevation	LFP (50%) Elevation	PNP (15%) Elevation	Lateral Structure Station	Boring	Overtop Elevation	PFP (85%) Elevation	LFP (50%) Elevation	PNP (15%) Elevation
						22.269	None	48.95	48.95	48.95	48.95
						21.999	None	43.63	43.63	43.63	43.63
						21.59*	None	48.66	47.66	46.66	45.66
						20.89	DD12	47.39	46.89	46.14	45.39
						19.99	DD12	47.31	46.81	46.06	45.31
						19.47	DD12	46.91	46.41	45.66	44.91
						18.56	DD12	43.91	43.41	42.66	41.91
						17.89*	None	45.46	44.96	44.21	43.46
17.529	DD17-1L	46.52	45.82	43.77	41.72	17.519	DD12	42.87	42.37	41.52	40.67
17.045	DD17-1L	44.80	44.10	42.05	40.00	17.049	DD17-1L	44.39	43.89	43.04	42.19
16.779*	DD17-1L	44.71	44.01	41.96	39.91	16.777	DD17-1L	44.19	43.69	42.69	41.69
16.599	DD17-2L	44.07	43.77	42.82	41.87	16.58	DD17-2L	44.01	43.51	42.51	41.51
16.29	DD17-2L	43.44	43.14	42.19	41.24	16.28	DD17-2L	43.39	42.89	41.89	40.89
15.899	DD17-2L	42.99	42.69	41.74	40.79	15.88	DD17-2L	41.66	41.16	40.16	39.16
15.08	DD17-2L	40.26	39.96	39.01	38.06	15.09	DD17-2L	40.63	40.13	39.13	38.13
14.58	DD17-2L	40.67	40.37	39.42	38.47	14.59	DD17-2L	40.46	39.96	38.96	37.96
13.98	DD17-3L	39.90	39.30	37.70	36.10	13.99	DD1-1R	40.85	40.45	39.25	38.05
13.78	DD17-3L	38.80	38.20	36.60	35.00	13.79*	DD1-1R	37.35	36.95	35.75	34.55
						13.09	DD1-1R	37.49	37.09	35.89	34.69
13.049	DD17-3L	37.44	36.84	35.24	33.64						
						12.9	DD1-1R	35.17	34.77	33.57	32.37
12.39*	DD3-1L	32.99	32.89	32.24	31.59	12.38	DD1-1R	35.67	35.27	34.07	32.87
11.69	DD3-1L	32.47	32.37	31.72	31.07	11.68	DD1-1R	34.25	33.85	32.65	31.45
11.18	DD3-1L	32.16	32.06	31.41	30.76	11.19	DD1-2R	34.26	31.26	30.26	29.26
10.599	DD3-1L	31.24	31.14	30.49	29.84	10.598	DD1-2R	31.78	28.78	27.78	26.78
10.099	DD3-1L	30.07	29.97	29.32	28.67	10.098	DD1-2R	29.57	26.57	25.57	24.57

* Existing condition index point located at upstream end of lateral structure.

Table 4. (continued)

NORTH FORK OF SKAGIT RIVER: LEFT BANK						NORTH FORK OF SKAGIT RIVER: RIGHT BANK					
Lateral Structure Station	Boring	Overtop Elevation	PFP (85%) Elevation	LFP (50%) Elevation	PNP (15%) Elevation	Lateral Structure Station	Boring	Overtop Elevation	PFP (85%) Elevation	LFP (50%) Elevation	PNP (15%) Elevation
939	DD22-2L	28.73	27.83	26.33	24.83	938	DD1-2R	27.75	24.75	23.75	22.75
884	DD22-2L	27.44	26.54	25.04	23.54	883	DD1-2R	28.05	25.05	24.05	23.05
828*	DD22-2L	26.91	25.99	24.49	22.99	827	DD1-2R	27.43	24.43	23.43	22.43
809	DD22-2L	26.57	25.67	24.17	22.67	808	DD1-2R	27.95	24.95	23.95	22.95
789	DD22-2L	26.89	25.99	24.49	22.99	788	DD1-2R	26.03	23.03	22.03	21.03
732	DD22-2L	25.58	24.68	23.18	21.68	731	DD1-2R	25.34	22.34	21.34	20.34
719	DD22-2L	24.53	23.63	22.13	20.63	718	DD1-2R	24.93	21.93	20.93	19.93
659	DD22-2L	23.30	22.40	20.90	19.40	658	DD1-2R	23.99	20.99	19.99	18.99
619	DD22-2L	22.13	21.23	19.73	18.23	618	DD1-2R	24.07	21.07	20.07	19.07
569	DD22-2L	21.83	20.93	19.43	17.93						
549	DD22-2L	20.71	19.81	18.31	16.81						
474	DD22-2L	16.96	16.06	14.56	13.06						
449	DD22-2L	16.57	15.67	14.17	12.67						
SOUTH FORK OF SKAGIT RIVER: LEFT BANK						SOUTH FORK OF SKAGIT RIVER: RIGHT BANK					
939	DD3-1L	29.66	29.56	28.91	28.26	938	DD22-1R	28.52	28.42	27.77	27.12
874	DD3-1L	26.57	26.47	25.82	25.17	873	DD22-1R	26.34	26.24	25.59	24.94
779	DD3-1L	24.80	24.70	24.05	23.40	778	DD22-1R	25.80	25.70	25.05	24.40
705	DD3-1L	23.68	23.58	22.93	22.28						
						694	DD22-1R	24.68	24.58	23.93	23.28
627	DD3-1L	21.66	21.56	20.91	20.26	628	DD22-1R	22.49	22.39	21.74	21.09
578	DD3-1L	21.27	21.17	20.52	19.87	577	DD22-1R	20.68	20.58	19.93	19.28
524	DD3-1L	18.97	18.87	18.22	17.57	523	DD22-1R	18.41	18.31	17.66	17.01
464*	DD3-1L	16.74	15.80	15.15	14.50						
339	DD3-1L	17.37	17.27	16.62	15.97						
249	DD3-1L	16.19	16.09	15.44	14.79						

* Existing condition index point located at upstream end of lateral structure.

e. Basic Assumptions and Limitations

It is important to note some of the basic capabilities, assumptions, and limitations inherent with the HEC-RAS models. HEC-RAS is used to simulate one-dimensional, unsteady flow. It is a fixed bed analysis and does not explicitly account for sediment movement, scour, or deposition. However, as described in Appendix A, the effect of scour at the BNSF bridge was approximately accounted for by modeling a somewhat smaller debris blockage than might be experienced in practice under the assumption that the reduction in conveyance area due to debris blockage would be at least partially offset by scour. The models assume no exchange with groundwater. The model is intended to adequately reproduce levee breaches and simulate channel hydraulics.

Floodfighting activities are simulated in hydraulic model calibration but not when applying the models to determine water levels and to characterize flood conditions for the hypothetical design flood events. Floodfighting activities included in the model calibration consist of construction of the temporary sand bag wall or flood barrier in Mount Vernon and sandbagging of the railroad track in the Sterling area (about RM 21.9).

2.4.2 FLO-2D Model Development

FLO-2D, developed by FLO-2D Software, Inc., is used to model overbank flood routing for this study in all areas downstream from SR-9 (RM 22.3) except the lower Nookachamps Creek basin, the Riverbend area, and North Mount Vernon. These three areas are represented as storage areas within the HEC-RAS model (see Section 2.4.1). Out-of-bank flows due to spill from the channel or levee breaches are generated in HEC-RAS and passed to the corresponding grid elements in FLO-2D to simulate floodplain flows. FLO-2D Version 2009.06 is being used to conduct this effort. More information about FLO-2D can be found in the FLO-2D reference manual (FLO-2D Software Inc. 2009).

a. Purpose of Model

FLO-2D is used in this study to model overbank flows in areas where the complexity of the floodplain is such that accurate results cannot be obtained using a one-dimensional approach such as HEC-RAS. FLO-2D has the capability of modeling both one-dimensional channel flow and two-dimensional overbank flow but for this study is run in overbank (i.e. floodplain) areas only. The FLO-2D model begins at the Sedro-Woolley bridges and extends to tidewater, exclusive of the main channel, Riverbend, North Mount Vernon and Nookachamps/Harts Slough areas, which are modeled within HEC-RAS (see Figure 1).

b. Procedures and Process

The FLO-2D model is based on the FEMA Flood Insurance Study (FIS) model developed in 2008. Extensive updates and modifications were made to the FIS model for this study and the model was updated to run under FLO-2D Version 2009.06.

In the FIS, FLO-2D was used to simulate flows for the entire river channel and floodplain system. In the present study, the main river channel, Riverbend, North Mount Vernon and Nookachamps/Harts Slough areas are modeled within HEC-RAS. Therefore the 1-D FLO-2D channel input files from the FIS model were removed and the grid cells for these areas were turned off. This results in the floodplain being broken into three distinct parts. The first covers the right bank of the Skagit River, starting at RM 22.3 and extending to the mouth of the North Fork Skagit River. This portion of the floodplain is modeled with 15,498 grid cells encompassing 56,930 acres. The second covers the left bank of the Skagit River, starting at RM 12.96 (downtown Mount Vernon) and extending past the mouth of the South Fork Skagit River south to Stanwood in Snohomish County. This portion of the model contains 2,981 grid cells covering 10,950 acres. The third covers Fir Island and is modeled with 2,118 grid cells covering 7,780 acres. This change, to provide for modeling of the channel system within HEC-RAS, was made to take advantage of the superior in-channel modeling capabilities of HEC-RAS, and to allow for efficient analysis of flood management measures and alternatives.

All grid cell elevations in the 2008 FIS model were updated using the best available topographic data. A 400-by-400 foot grid is utilized which provides the necessary detail on the floodplain without burdening the model computationally with an excessive number of grid cells. A composite elevation raster grid was created by combining seven recent topographic datasets. Each dataset was given a priority based on quality and age; where datasets overlapped the higher priority one was used. The final product is a 6-foot raster elevation grid. The approximately 4,400 elevation values in each 400-by-400 foot FLO-2D grid cell were then averaged to determine the grid cell elevation. Details on data sources, quality assurance checks and methods are given in Collins (2010).

Model parameters related to the effects of buildings on conveyance blockage and losses of flood storage were also updated. A GIS structures polygon coverage was created based on 1999 Corps of Engineers mapping. Structure polygons extracted from 2004 and 2009 Burlington and 2009 Mount Vernon aerial mapping were added. Finally the coverage was manually edited using the 2009 National Agriculture Imagery Program (NAIP) orthophoto. Effort focused on the Burlington - Mount Vernon area where there has been extensive development in the last 20 years and where structures are expected to be hydraulically significant. New small structures or those in rural areas where their effects on flow will be insignificant were not digitized. The cities of La Conner and Stanwood were not covered by the 1999 Corps mapping, although they both are within the FLO-2D model domain and subject to Skagit River floods. From a hydraulic point of view, they are both located behind sea dikes at the end of flood flow paths and will be subject to generally ponded conditions. Flood levels are governed by the sea dike elevations around them, and the extent of structures will not affect flood levels measurably. For these areas, FLO-2D model structure blockage parameters were estimated visually and applied.

Elevated roads, railroads, sea dikes and other features that behave as levees are coded separately in the FLO-2D model. Major features that clearly impact flood flows were checked and updated. Important features found to be missing from the FIS model and added were the Samish River levees, Fisher Slough levees and various levees around La Conner. In addition, the Interstate 5 roadway elevations were updated across the entire floodplain and the I-5 bridge over Gages Slough in Burlington was added.

Post-processing of the FLO-2D output in conjunction with basin topographic data is performed to generate and define inundated areas.

c. Boundary Conditions

The types of boundary conditions in the FLO-2D computer model include inflow and outflow boundary nodes, tailwater conditions, and inflow hydrographs. Inflow boundary nodes are identified in the input file, with associated inflow hydrographs representing levee overflows and breaches being calculated by the HEC-RAS model.

In addition to the flows representing overtopping and breaches from the HEC-RAS model, an inflow hydrograph is provided for the Samish River which is tributary to the right bank Skagit River floodplain north of Burlington. For information on derivation of Samish River inflows, refer to the Hydrology Technical Documentation. Other floodplain tributary inputs are too small to affect hydraulic results.

Sea dikes determine the downstream boundaries for the FLO-2D model. Outflow is allowed to occur over the sea dikes into the Swinomish Channel, Skagit Bay, Padilla Bay, and Samish Bay. In a small number of locations where outflow is not controlled by sea dikes (e.g. the outlet of the Samish River), the FLO-2D downstream boundary is an approximation of normal depth (tidal influence is not modeled).

d. Basic Assumptions and Limitations

Several basic assumptions and limitations must be considered with the FLO-2D model. Two-dimensional flow simulation in FLO-2D is limited to the eight directions of the compass (north, northeast, northwest, east, southeast, south, southwest, and west).

The simulations performed represent a fixed bed analysis, so erosion and sedimentation in the floodplain are not modeled. Culverts under roads or bay front outlet structures are not modeled. The reason that culverts are not modeled for overland flow in the existing condition model is that the capacities of the culverts are small compared with the overbank discharge. The FLO-2D models do not contain any sea dike failure scenarios and do not account for pump stations or any other flood fighting techniques to reduce the flood damage.

3.0 Model Calibration

3.1 Sources of Data

Information on flows and high-water marks has been collected for the November 2006, October 2003 and November 1995 flood events at a number of locations. Information on local tributary flows entering the Skagit below some of the major gages is fairly limited, however. The precipitation also varies from the upper basin to the lower basin and this information is not very detailed around the smaller basins, which limits the ability to use rainfall-runoff models to accurately estimate these flows.

3.2 HEC-RAS Calibration and Validation

The primary goal of the HEC-RAS model calibration was to accurately simulate stages downstream of Sedro-Woolley for a given discharge. Thus, the USGS gaged flows for the Skagit River near Concrete (USGS gage 12194000) were set as the upstream boundary condition, and local tributary inflows were adjusted as necessary to fit the observed discharge at the Mount Vernon USGS gage (USGS 12200500). The model's roughness values were then calibrated to the 2003 flood and validated for the 1995 and 2006 floods. The reason that the effort is focused on these three floods is because they best represent the current channel characteristics. Two older large floods with some available data were not used. The 1990 event had a levee failure during the event that would affect the calibration. The 1975 flood would be significantly affected by the channel changes that are shown in Table 1 in addition to very different levee heights than present and unknown flood fighting efforts.

The 2003, 2006 and 1995 events demonstrate the variability in flow between the Concrete and Mount Vernon gages that make simply routing flows from Concrete with assumed local inflows problematic. The October 2003 event was preceded by a fairly dry summer. This set up a condition where the overbank was dry preceding the first storm and allowed for greater losses in the overbank, due to factors such as infiltration to the groundwater, than a more typical condition such as the November 1995 flood. The 1995 flood had more typical antecedent soil conditions preceding the flood event, which allowed more water to make it downstream to Mount Vernon. The November 2006 event had dry antecedent conditions similar to the 2003 event. During the November 2006 event, the USGS Nookachamps Creek gage did not even peak during the flood but continued to rise for several weeks afterwards. We would therefore expect equal or greater losses in this event than 2003, but this is not the case in the published data as seen in Table 5. A more detailed discussion of the observed data for the 2006 event is included in the validation section of this chapter.

All model results presented in this chapter are for the most current (2013) HEC-RAS model unless stated otherwise. Model results from the 2004 HEC-RAS model are provided for comparative purposes in Tables 7 and 8 below.

Table 5. Reported USGS Gaged Peak Discharge (cfs)

Flood Event	Peak Flow at Concrete	Peak Flow at Mt. Vernon	Reduction in Peak Flow
November 1995	160,000	141,000	19,000
October 2003	166,000	135,000	31,000
November 2006	145,000	138,000	7,000

The calibrated HEC-RAS roughness ranges are listed by reach in Table 6.

Table 6. HEC-RAS Roughness Ranges (Manning's n values)

River Reach	Main Channel	Floodplain
Cascade River	0.04	0.12
Sauk River	0.025-0.038	0.04
Baker River	0.04	0.06-0.07
Skagit River from Marblemount to Concrete	0.038-0.040	0.15
Skagit River from Concrete to Sedro-Woolley	0.035	0.05-0.10
Skagit River from Sedro-Woolley to Forks	0.030-0.038	0.04-0.12
North Fork Skagit River	0.030-0.032	0.04-0.10
South Fork Skagit River	0.030-0.032	0.12

3.2.1 Calibration: October 2003 Event

The 2013 HEC-RAS model calibration simulates within 0.5 feet most of the high water marks for the 2003 event from Sedro-Woolley downstream. Table 7 and Figure 3 show the high water marks for the event and the model's simulated maximum water surface profile. Also shown in Table 7 are results from the 2004 HEC-RAS model.

The high water marks upstream of Sedro-Woolley are typically more than a foot above the model simulated water surface for the October 2003 event. This section of model uses cross sections dating to 1975, so the general aggradational trend of the lower Skagit River is believed to be at least partly responsible for this difference. Given that there are no damage reaches being evaluated between Concrete and Sedro-Woolley, roughness values were set to typical values for the observed floodplain land cover and channel in order to accurately route and attenuate flows downstream, rather than using very high roughness values which would be required to more accurately match high water marks in this reach, as in the 2004 hydraulic model.

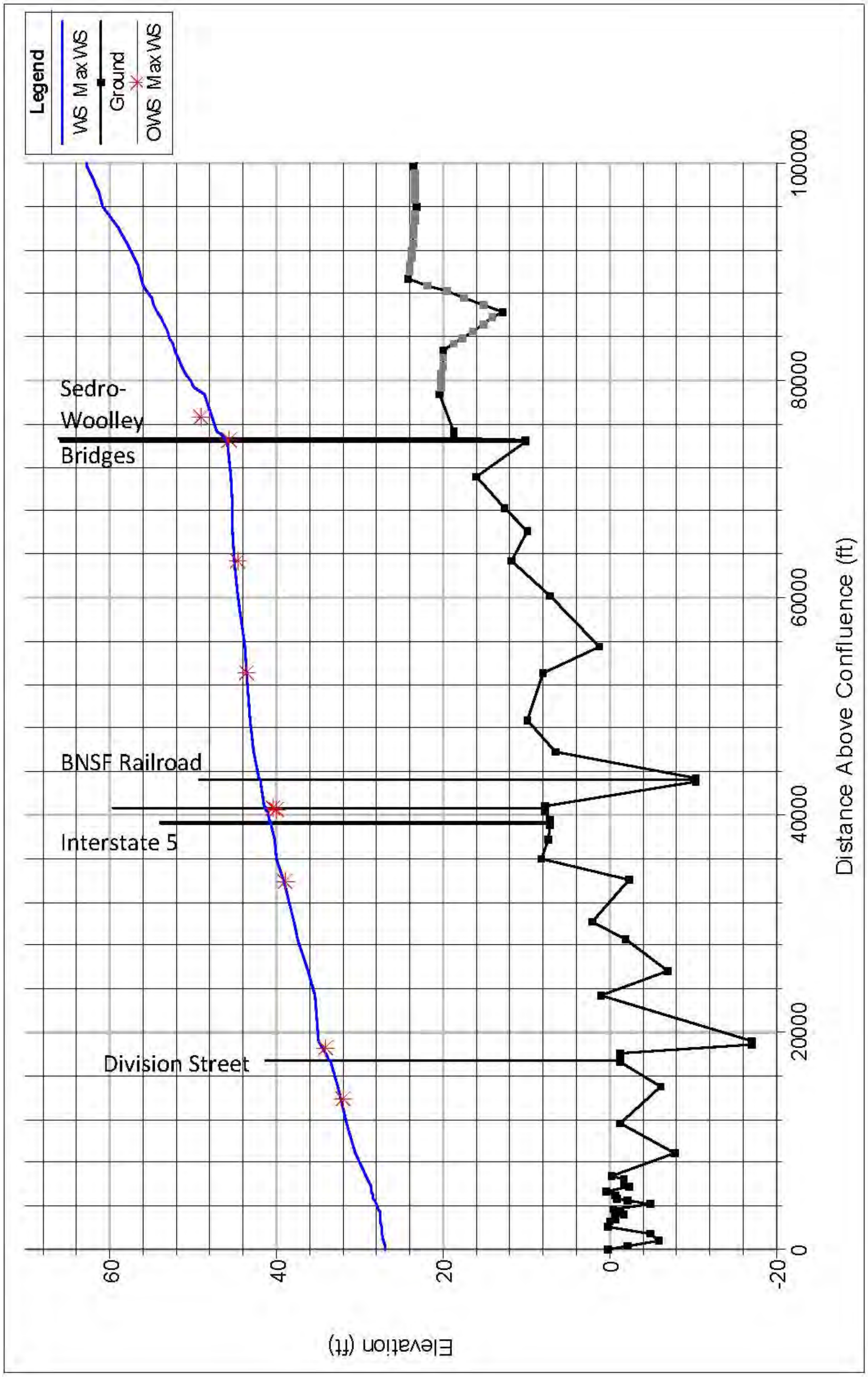
The lowest high water marks on the North and South Forks are around 3 feet higher than simulated, whereas high water marks upstream match well. The water surface profiles indicate a very steep drop where the river escapes the confinement of the levees and enters Skagit Bay. The location of cross sections on the forks is somewhat uncertain, so even small errors in river station can lead to large differences in simulation values given the steepness of the water surface locally. These two high water marks are below the location of any index points for damage reaches therefore the error does not affect the risk-based analysis.

The USGS reported discharge for the Skagit River near Concrete was used as the upstream boundary condition for the calibration. Local inflow between Concrete and Mount Vernon was estimated by regression between local inflows and observed flows on the North Fork Stillaguamish River, and then adjusted so that simulated flows at Mount Vernon matched observed flows (Figure 4). Due to a gage failure during the flood, no volume or timing comparisons are possible.

Table 7. October 21, 2003 Flood: Simulated vs. Observed High Water Marks

Source	River	Location	High Water Mark	2013 Model Results		2004 Model Results	
				Simulated Water Surface Elevation	Diff. from HWM	Simulated Water Surface Elevation	Diff. from HWM
		(River Mile)	(feet NAVD88)	(feet NAVD88)	(feet)	(feet NAVD88)	(feet)
USGS Gage	Mainstem Skagit	78.7	323.0	322.7	-0.3	322.7	-0.3
Skagit County	Mainstem Skagit	59.65	199.3	200.9	1.6	200.9	1.6
USGS Gage	Mainstem Skagit	54.1	176.0	172.3	-3.7	175.8	-0.1
Skagit County	Mainstem Skagit	49.75	153.8	151.6	-2.1	154.0	0.2
Skagit County	Mainstem Skagit	40.18	104.4	103.3	-1.1	104.7	0.3
Skagit County	Mainstem Skagit	29.90	67.3	65.8	-1.5	67.4	0.1
Skagit County	Mainstem Skagit	22.78	48.9	47.6	-1.3	48.9	-0.1
USGS Gage	Mainstem Skagit	22.3	45.6	46.0	0.4	46.1	0.5
Skagit County	Mainstem Skagit	21.6	44.5	45.0	0.5	43.8	-0.7
Skagit County	Mainstem Skagit	19.48	43.5	43.5	0.0	43.7	0.2
Skagit County	Mainstem Skagit	17.07	40.4	41.3	0.9	41.6	1.2
USGS Gage	Mainstem Skagit	17.04	40.0	41.2	1.2	41.6	1.6
Skagit County	Mainstem Skagit	15.89	39.0	39.0	0.0	39.7	0.7
Skagit County	Mainstem Skagit	13.03	34.0	34.5	0.5	34.9	0.9
Skagit County	Mainstem Skagit	12.18	32.0	32.2	0.2	33.2	1.3
Skagit County	North Fork Skagit	8.09	25.0	24.6	-0.4	26.0	1.0
Skagit County	North Fork Skagit	4.42	15.5	12.7	-2.8	15.4	-0.1
Skagit County	South Fork Skagit	5.8	19.7	19.7	0.1	-	-
Skagit County	South Fork Skagit	3.52	14.1	11.1	-3.0	14.0	-0.1
USGS Gage	Sauk River	5.40	288.7	289.3	0.6	289.3	0.6

Figure 3. HEC-RAS Simulated Water Surface Profile and Observed High Water Marks for October 2003 Event



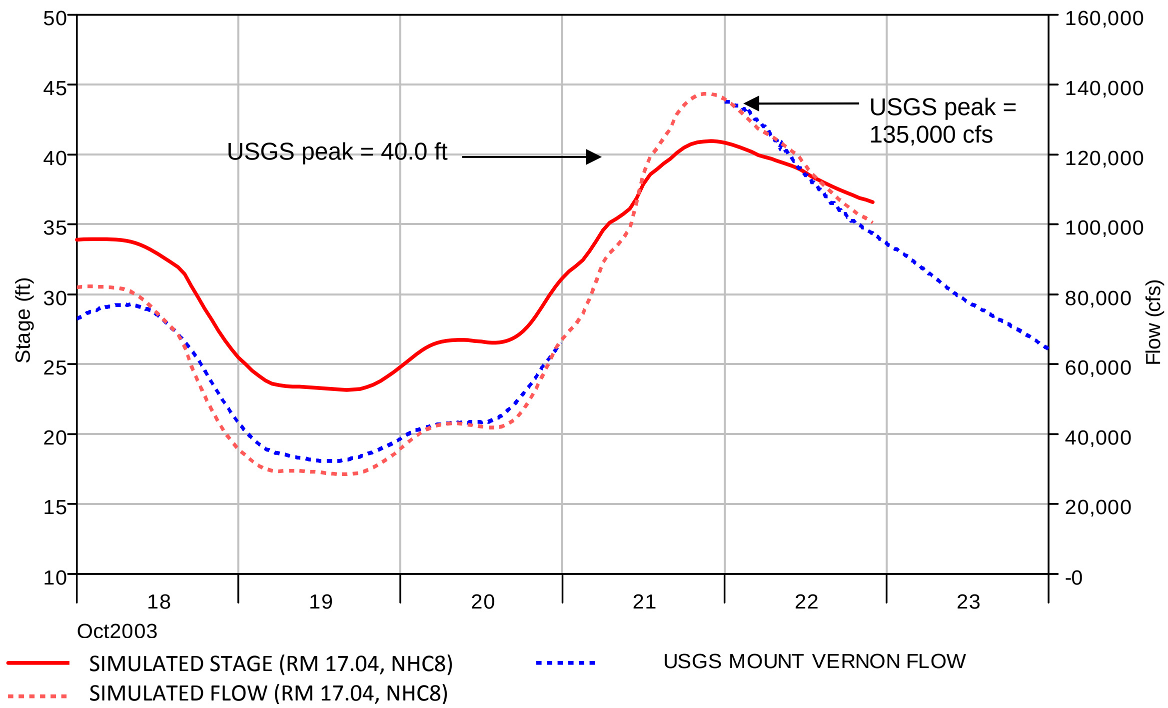


Figure 4. Comparison of USGS Gage Record to HEC-RAS Simulated Stage and Discharge Hydrographs at Mount Vernon for October 2003 Event

3.2.2 Validation: November 1995 Event

The November 1995 event was simulated using the calibrated HEC-RAS model to validate the model's calibration. Observed and simulated high water marks are shown in Table 8 and Figure 5. Also shown in Table 8 are results from the 2004 HEC-RAS model.

For the lower basin validation, the 1995 event was simulated using the USGS gaged flow at Concrete as the upstream boundary condition, and historic tides as the downstream boundary condition. Local inflow was determined as for the October 2003 calibration event and adjusted to match simulated and observed Mount Vernon flows reasonably well (Figure 6). Although a significant debris jam formed on the BNSF bridge during this event, it is believed that the debris accumulated on the receding limb of the flood hydrograph since no debris is evident in photographs taken close to the peak. Accordingly, the model validation assumes no bridge debris.

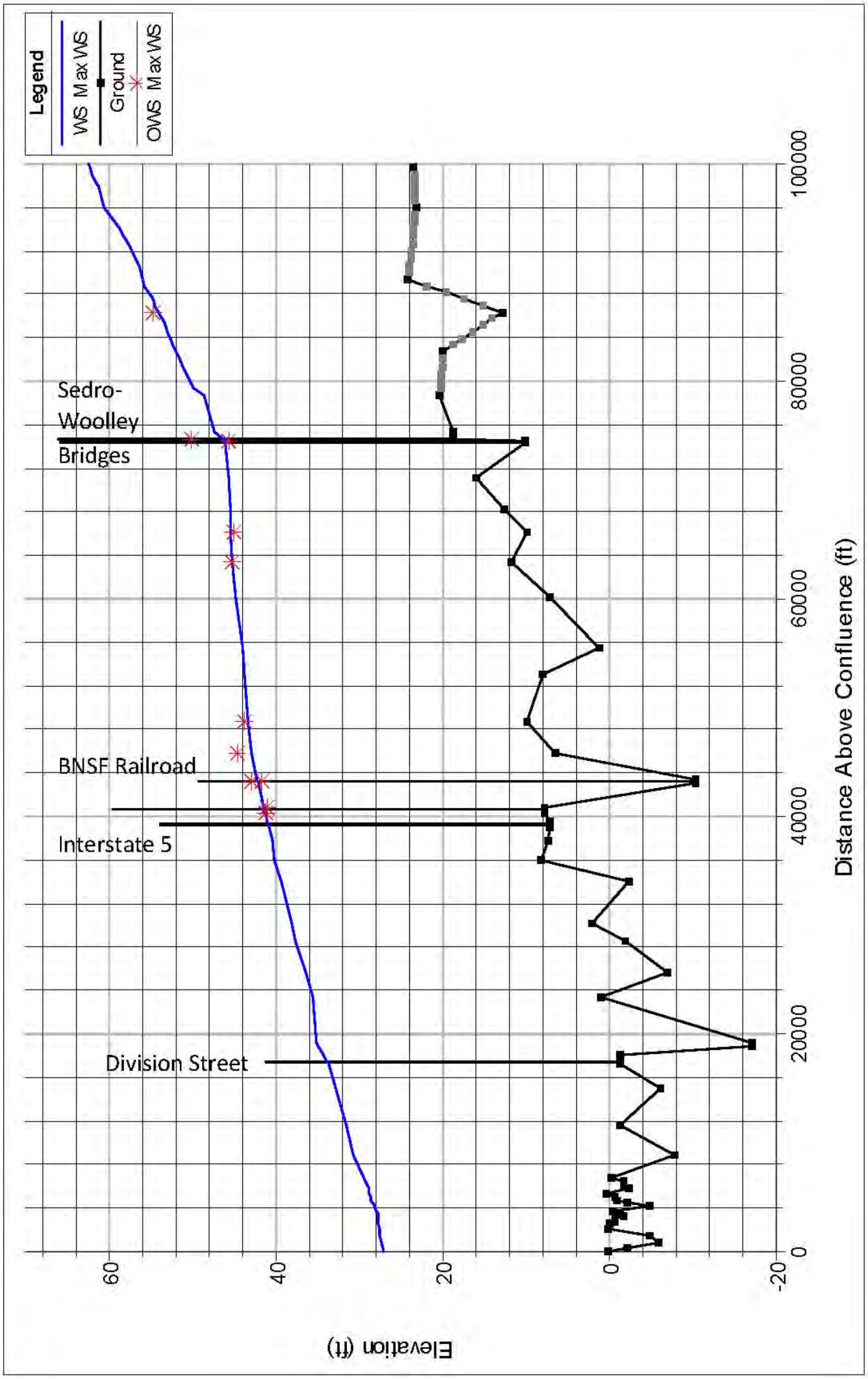
The model results (Figure 5) closely approximate the observed high water marks downstream of the Sedro-Woolley bridges (RM 22.3), as all but two of the marks are within 0.5-ft. Upstream of Sedro-Woolley, the USACE high water marks appear to be inconsistent and would require significant variation in roughness values to produce a good fit. Also, upstream of the Sedro-Woolley bridges, cross sections date back to 1975. Part of the calibration's inconsistency across events for this reach is likely due in part to the age of the data and significant channel changes that have occurred since 1975. Additional discussion of the 1995 high water marks and simulation results for the 1995 event is provided in Appendix A.

Table 8. November 29, 1995 Flood: Simulated vs. Observed High Water Marks

Source	Location	High Water Mark	2013 Model Results		2004 Model Results	
			Simulated Water Surface Elevation	Diff. from HWM	Simulated Water Surface Elevation	Diff. from HWM
	(River Mile)	(feet NAVD88)	(feet NAVD88)	(feet)	(feet NAVD88)	(feet)
USGS Gage	78.7	322.6	322.5	-0.1	322.5	-0.1
USACE	54.12	175.4	174.7	-0.6	174.7	-0.6
USGS Gage	54.10	175.3	171.8	-3.5	174.6	-0.8
USACE	52.90	166.6	165.8	-0.8	168.6	2.1
USACE	46.97	142.5	136.2	-6.3	138.6	-3.9
USACE	40.03	107.1	102.5	-4.5	103.5	-3.6
USACE	32.93	75.7	74.4	-1.2	75.3	-0.4
USACE	30.30	65.1	65.7	0.6	67.1	2.1
USACE	24.70	54.6	54.1	-0.5	56.4	1.8
USACE	22.40	50.0	46.2	-3.8	47.2	-2.8
Leonard Halverson	22.30	45.7	46.2	0.5	46.3	0.6
Leonard Halverson	21.93	45.1	45.3	0.2	45.4	0.3
Leonard Halverson	21.60	45.2	45.2	0.0	45.3	0.2
Leonard Halverson	18.57	43.8	43.3	-0.5	45.3	1.5
Leonard Halverson	17.90	44.6	42.9	-1.7	44.3	-0.3
Photograph (Chuck Bennett, DD#12)	17.54	41.7	42.1	0.4	n/a	n/a
Leonard Halverson	17.53	43.0	42.1	-0.9	43.1	0.1
Leonard Halverson	17.08	41.0	41.5	0.5	41.5	0.5
USGS Gage	17.04	41.1	41.3	0.2	41.4	0.3

Simulated flood volumes are less than 1% different than observed, and the time of peak matches observed data to within one hour. The 1995 flood event simulation confirms that the model is accurately simulating water surface elevations in the reach of primary interest for this study from Sedro-Woolley downstream.

Figure 5. HEC-RAS Simulated Water Surface Profile and Observed High Water Marks for November 1995 Event



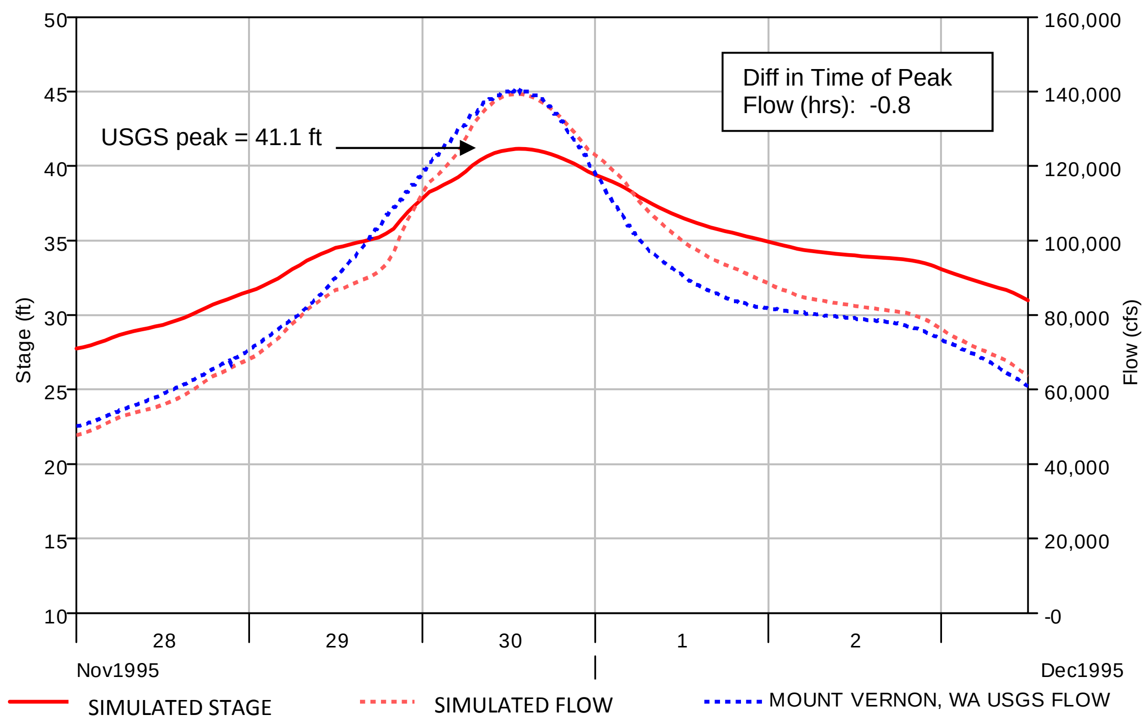


Figure 6. Comparison of USGS Gage Record to HEC-RAS Simulated Stage and Discharge Hydrographs at Mount Vernon for November 1995 Event

3.2.3 Validation: November 2006 Event

The November 2006 flood was also simulated to test the hydraulic model performance. For the downstream boundary condition, observed Seattle tides were obtained and corrected to Skagit Bay values.

Initial HEC-RAS simulations followed the same procedure as for the 2003 and 1995 flood model runs, with local inflows between Concrete and Mount Vernon being scaled so that the Mount Vernon gage simulated flows matched the published USGS peak flow. However, initial simulations results were poor, with significant over simulation of observed high water marks when local inflows were scaled so that the Mount Vernon gage simulated flow matched the published peak flow of 138,000 cfs. The USGS measured a discharge of 125,000 cfs on the rising limb of the November 2006 event, but rated the measurement “poor”. Nevertheless, as a result of this measurement, a new rating curve was developed using this measurement to define the high end of the rating curve. The revised rating was used to produce the currently published peak flow at the Mount Vernon gage of 138,000 cfs. Using the previous rating table, the peak flow would have only been around 110,000 cfs.

Considering published stage and discharge data for the Mount Vernon gage, if the published peak flow estimate for the 2006 event is correct, then the river bed must have scoured more than two feet during this flood, increasing the capacity of the river to convey more water at lower stage. The 2006 high water marks (Table 10) run 1 to 2 feet lower than coincident 2003 high water marks from upstream of Sedro-Woolley through downtown Mount Vernon, even though the published USGS peak at Mount Vernon for 2006 (138,000 cfs) is 3,000 cfs higher than the 2003 peak (135,000 cfs). This implies that the entire river channel for at least 12 miles scoured similar amounts. As upstream scouring should have supplied additional sediment to downstream reaches, it is difficult to conceive of this entire reach undergoing this level of scour in one flood. In addition, stage-discharge measurements since 2006 have consistently plotted above previous data, indicating the river bed has aggraded compared to pre-2006 conditions. The above indicate that the published peak discharge for the 2006 event at Mount Vernon may be too high.

The HEC-RAS model better simulates the majority of high water marks when lower flows at Mount Vernon are used. A lower 2006 peak flow at Mount Vernon is also more consistent with the estimated reduction in peak flows between Concrete and Mount Vernon reported for the 1995 and 2003 floods (Table 5). Simulation results presented here for the 2006 flood assume a peak flow at Mount Vernon of 123,000 cfs. Total simulated volume from November 6-9 is 9.4% lower than USGS values using this peak flow.

At the Mount Vernon gage, the HEC-RAS model simulates a stage that is consistently higher than the USGS published data (Figure 7). The simulated discharge hydrograph closely approximates the USGS published discharges below 120,000 cfs (Figure 7). However, near the 2006 event peak, above 120,000 cfs, the simulated and observed hydrographs quickly diverge, with the model predicting a peak discharge of 123,000 cfs compared with the published USGS peak of 138,000 cfs. One possibility is that the bed locally scoured (as the USGS observations suggest), which would lower the simulated stages at the gage without a significant impact on discharges.

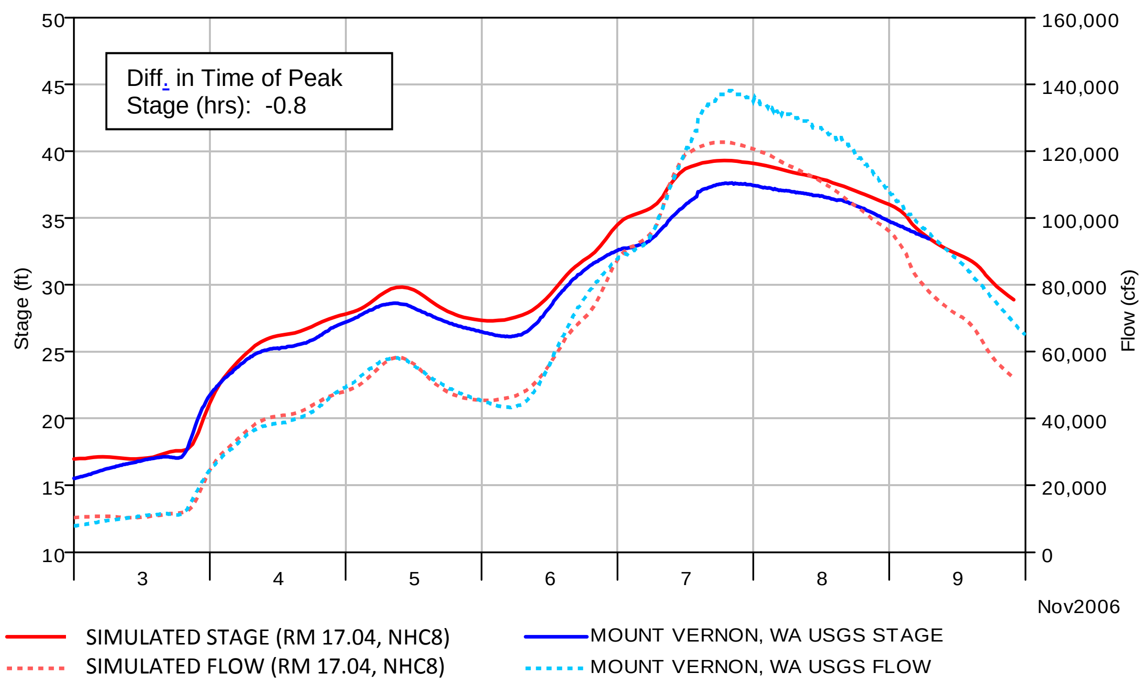


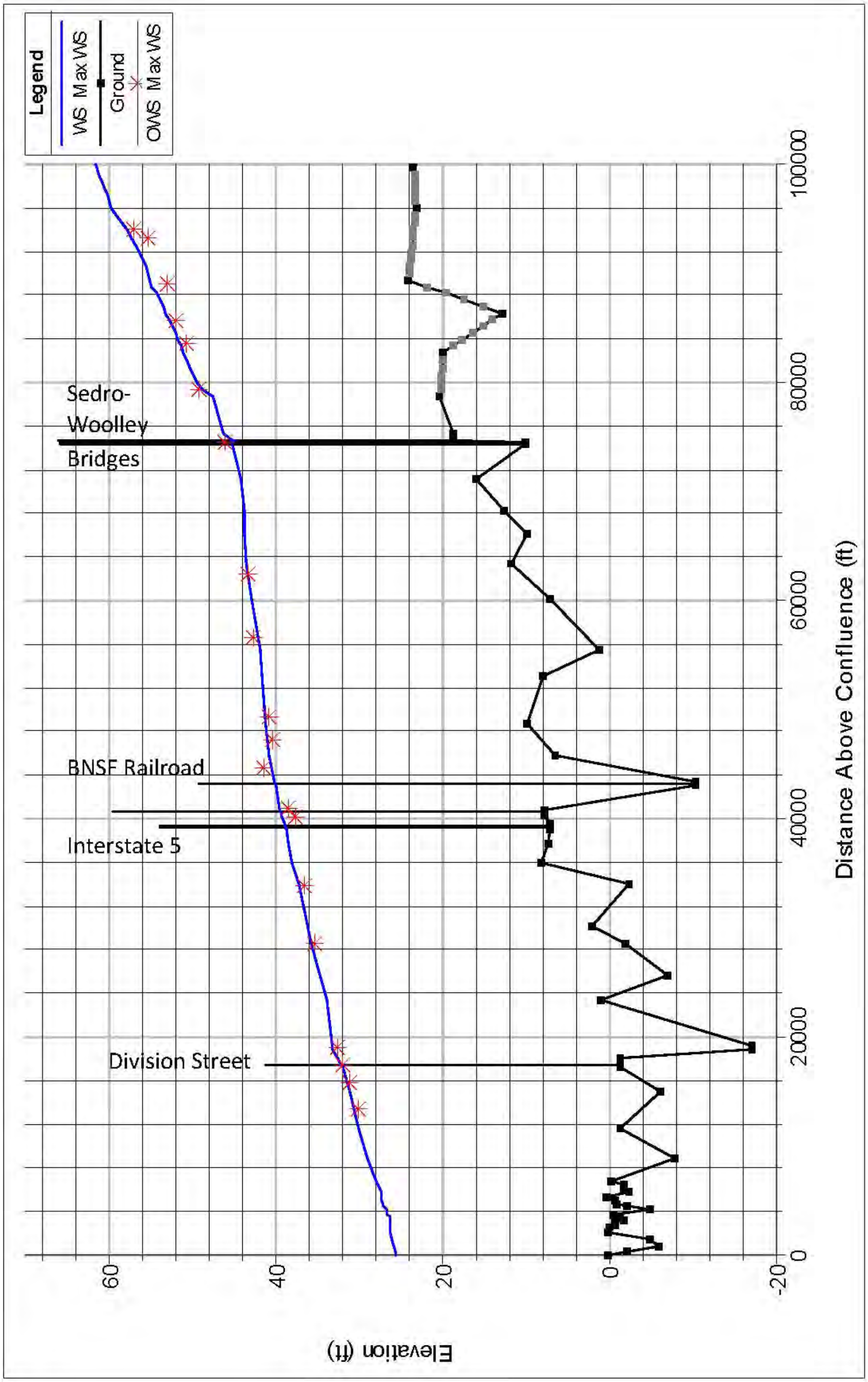
Figure 7. Comparison of USGS Gage Record to HEC-RAS Simulated Stage and Discharge Hydrographs at Mount Vernon for November 2006 Event

In general, the HEC-RAS simulation produced similar water surface elevations to the observed high water marks for the 2006 event, as shown in Table 9 and on the water surface profile of Figure 8. The primary exceptions to this are for the reach extending from the USGS Mount Vernon gage to upstream of the BNSF bridge, and in the Sedro-Woolley reach upstream from SR-9. It is noted that the 2006 high water mark data upstream from the Sedro-Woolley bridges are not true high water marks but represent observed water levels near to the crest of the flood. Based on information on the time of the flood peak and the time at which water levels were marked, it is believed that these data represent actual high water marks within a few tenths of a foot.

Table 9. November 7, 2006 Flood: Simulated vs. Observed High Water Marks

Source	River	Location	High Water Mark	2013 Model Results	
				Simulated Water Surface Elevation	Difference from HWM
		(River Mile)	(feet NAVD88)	(feet NAVD88)	(feet)
USGS Gage	Mainstem Skagit	54.1	173.6	170.7	-2.9
Skagit County	Mainstem Skagit	26.08	57.0	57.9	0.9
Skagit County	Mainstem Skagit	25.95	55.4	57.3	1.9
Skagit County	Mainstem Skagit	25.13	52.9	55.0	2.0
Skagit County	Mainstem Skagit	24.59	51.9	52.7	0.8
Skagit County	Mainstem Skagit	24.23	50.6	51.5	0.9
Skagit County	Mainstem Skagit	23.38	49.2	48.9	-0.3
USGS Gage	Mainstem Skagit	22.30	46.0	44.8	-1.2
Skagit County	Mainstem Skagit	22.29	43.6	45.2	1.6
Skagit County	Mainstem Skagit	21.40	43.2	43.2	0.0
Skagit County	Mainstem Skagit	20.9	42.6	42.8	0.1
Skagit County	Mainstem Skagit	18.77	40.9	41.2	0.4
Skagit County	Mainstem Skagit	18.31	40.5	40.9	0.5
Skagit County	Mainstem Skagit	17.79	41.5	40.4	-1.1
Skagit County	Mainstem Skagit	17.12	38.4	39.5	1.1
USGS Gage	Mainstem Skagit	17.04	37.6	39.2	1.6
Skagit County	Mainstem Skagit	15.85	36.5	37.2	0.7
Skagit County	Mainstem Skagit	14.80	35.4	35.7	0.3
Skagit County	Mainstem Skagit	14.59	35.4	35.7	0.3
Skagit County	Mainstem Skagit	13.05	32.5	33.2	0.7
Skagit County	Mainstem Skagit	12.96	32.0	32.2	0.2
Skagit County	Mainstem Skagit	12.65	31.1	31.5	0.3
Skagit County	Mainstem Skagit	12.09	30.0	30.6	0.7
Skagit County	South Fork Skagit	4.59	16.5	16.7	0.2

Figure 8. HEC-RAS Simulated Water Surface Profile and Observed High Water Marks for November 2006 Event



In addition to matching high water marks well with an assumed peak flow of 123,000 cfs, the model also provides good simulations of both the observed stage hydrographs for the USGS Nookachamps Creek near Clear Lake gage (USGS gage 12200100) and at the Anacortes Water Treatment Plant (AWTP) in Riverbend. Figure 9 compares the observed stage for the Nookachamps Creek near Clear Lake (Swan Road) gage to the simulated stage in the Skagit River main channel. The model representation of storage in the lower Nookachamps Creek basin appears to be good, with Nookachamps water levels lagging the main channel during the rising limb, before roughly equilibrating near the event's peak. The HEC-RAS model appears similarly well calibrated at the AWTP, downstream from the three-bridge corridor, as seen by the stage comparison in Figure 10. The USGS stage-only gage at Sedro-Woolley (USGS gage 12199000) is compared to the simulated hydrograph in Figure 11.

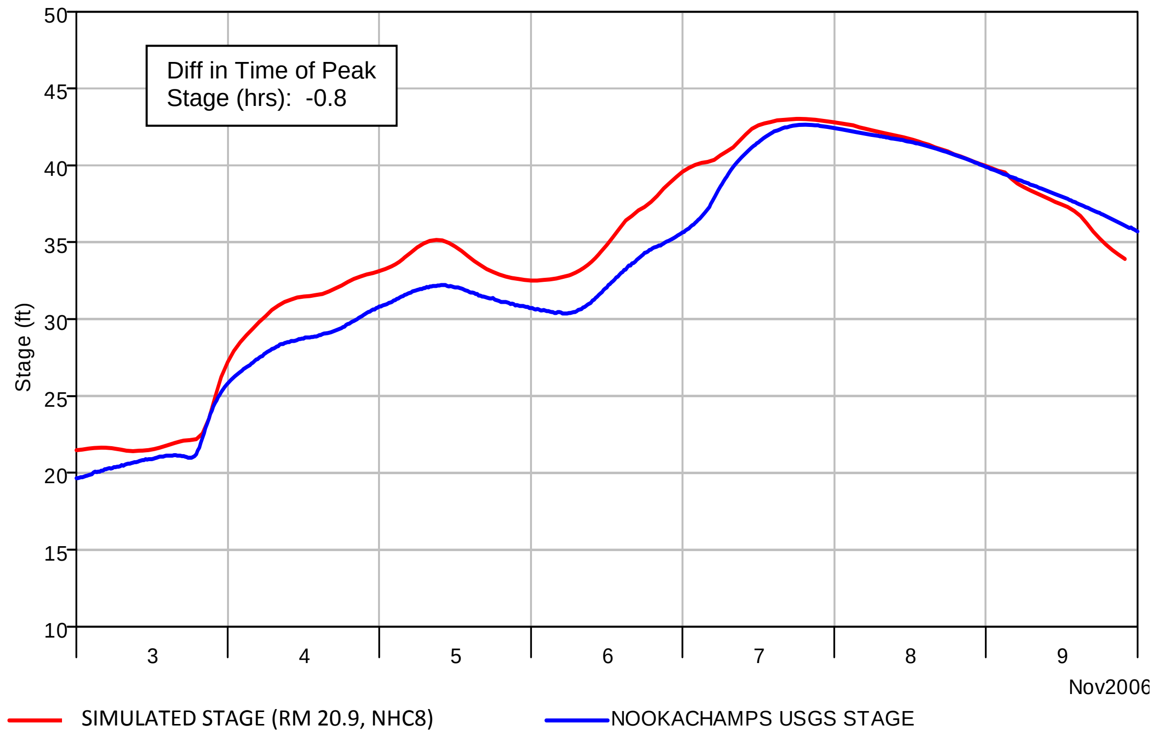


Figure 9. Comparison of USGS Gage Record to HEC-RAS Simulated Stage Hydrograph at the Nookachamps Creek Swan Road Crossing for November 2006 Event

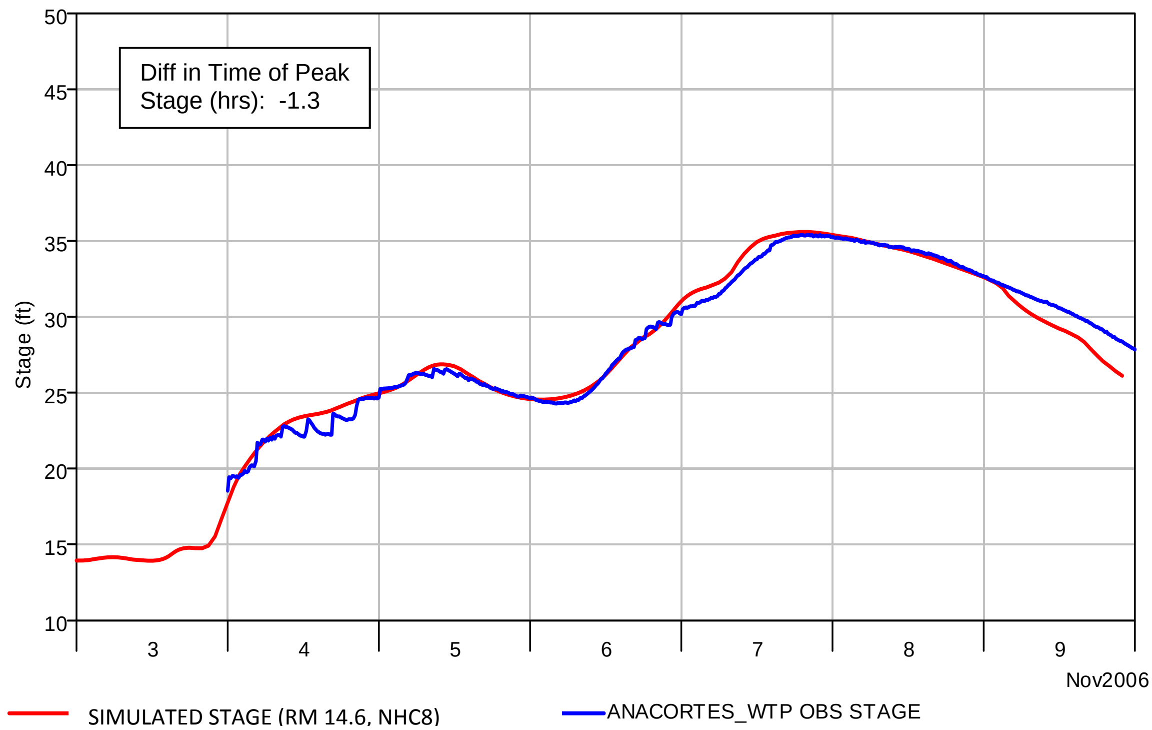


Figure 10. Comparison of Gaged Stage Hydrograph at Anacortes Water Treatment Plant to HEC-RAS Simulated Stage Hydrograph for November 2006 Event

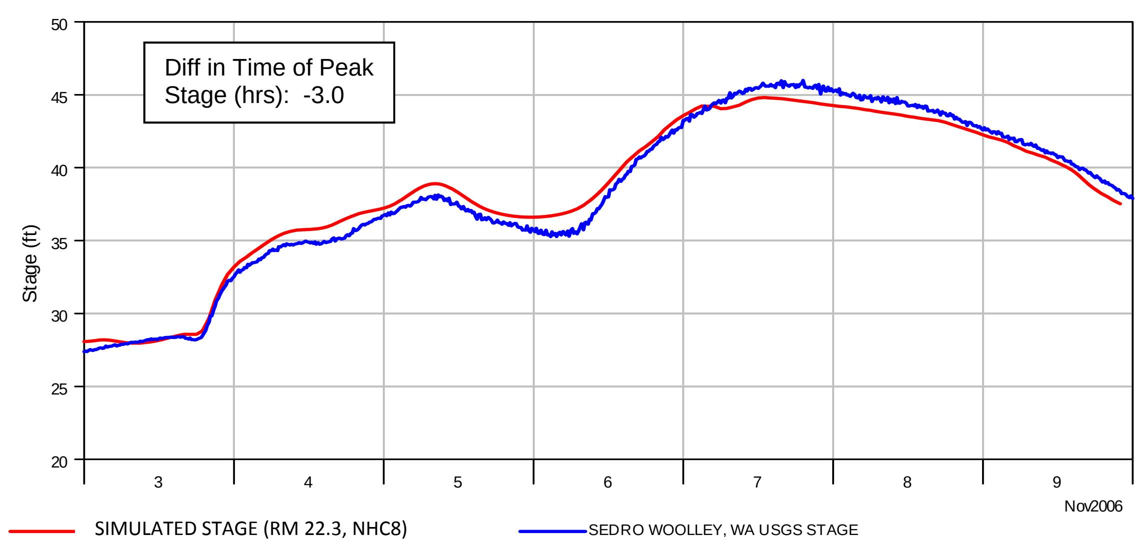


Figure 11. Comparison of USGS Gage Record to HEC-RAS Simulated Stage Hydrograph at Sedro-Woolley for November 2006 Event

3.3 FLO-2D Calibration

No data are available on floodplain flows or floodplain high water marks suitable for calibration or verification of the FLO-2D model. Therefore, Cowan's (1956) method is used to determine the floodplain roughness values. These are compared to previous studies giving typical roughness values found for certain ranges of depths of flows on specific types of floodplain surfaces to ensure they are appropriate. The derivations of these roughness values are listed in Table 10.

Table 10. FLO-2D Floodplain Roughness Values

Roughness Using Cowan (1956)									**Total**	**Other Literature**
Land Type	**Material Type**	**n_0**	**Degree of Irregularity**	**n_1**	**Effect of Obstructions**	**n_2**	**Vegetation**	**n_3**		**Ranges**
Agriculture	Earth	0.02	Moderate	0.01	Appreciable	0.025	Low	0.01	0.065	0.04-0.08[1]
Forested	Earth	0.02	Moderate	0.01	Appreciable	0.030	High	0.04	0.10	0.07-0.15[1]
Grass	Earth	0.02	Minor	0.005	Severe	0.06	Very High	0.065	0.15	0.15-0.24[2]
Developed	Pavement -Lawn	0-0.02	Smooth	0	Negligible-Appreciable	0 - 0.03	Low	0.01	0.01-0.06	0.011[2]-?

[1]From USACE (1993) EM 1110-2-1416
[2]From Engman (1986)

4.0 HEC-RAS/FLO-2D Model Results and Output

HEC-RAS and FLO-2D were jointly used to model the hydraulic conditions in the Skagit River Basin. Examples of the results of HEC-RAS simulations for various scenarios are provided in this section. These scenarios were developed to provide an understanding of the factors important in defining stage-uncertainty curves and of the bounds on flood behavior in order to guide the development of the modeling methodology used to support the economic analysis. Due to the extreme complexity of flooding patterns and consequent impacts on the economic analysis, discussion of the use of the hydraulic models for risk-based analysis is deferred to the Hydraulic Analysis Report (NHC 2013). More complete details of hydraulic modeling results, including delineation of floodplain inundation under various levee failure scenarios, are also provided in the Hydraulic Analysis Report.

4.1 No Breach Scenario

Simulations were performed using HEC-RAS to develop water surface profiles for a “no breach” scenario in which levees (and the natural river bank) are allowed to overtop but no levee failures or breaches occur. The “no breach” scenarios give an indication of the capacity of the system in the absence of levee failures and provide a bounding condition of overtopping volumes and inundation extents in the existing conditions floodplain. Note that unlike the model calibration runs, these simulations assume no flood fighting activities but they do assume that the Mount Vernon floodwall is in place.

Simulations were performed for the average channel roughness and the bridge debris loads indicated in Table 3 (6,000 sq. ft. on the BNSF bridge and 4,000 sq. ft. on the Great Northern Bridge). Water surface profiles for the nine hypothetical floods are provided in Appendix B along with the existing condition levee probable failure (PFP) and probable non-failure (PNP) elevations. Discharges at selected locations in the system are provided in Table 11. Also shown in Table 11 is estimated spill from the right bank of the Skagit upstream from the BNSF bridge. The reduction in peak flow from Sedro-Woolley to Mount Vernon (Riverside Bridge) is dependent on peak flow attenuation due to storage in the lower Nookachamps Creek basin, spill due to overtopping of Highway 20 in the vicinity of Sterling, and spill due to overtopping of the right bank Dike District 12 levees upstream from the BNSF bridge. Note in Appendix B that water surface profiles downstream from the BNSF bridge are very similar for 50-year events and larger due to upstream spill.

For large events (flows at the BNSF bridge greater than about 160,000 cfs), spill from the right bank upstream from the BNSF bridge is heavily dependent on assumed bridge debris loading conditions as discussed further in Section 4.2.

4.2 Bridge Debris Loading Scenarios

For flows greater than about 160,000 cfs, the assumed bridge debris loading at the BNSF bridge has a significant effect on system hydraulics. The hydraulic performance of the BNSF bridge under varying debris loads was investigated in detail as discussed in Appendix A. For large events, increased debris loading increases water levels upstream from the BNSF bridge for several miles. The impact is two-fold: increased water levels force more water into storage in Nookachamps, further attenuating peak flows; and increased water levels result in larger spill from the system through overtopping of Highway 20 and/or the right bank levees. Increased debris loads on the BNSF bridge therefore decrease downstream flows (and flood risk) and vice versa.

Simulations were performed for all nine hypothetical flood events for the fixed debris loads of Table 3 and for the scenario with no debris. Levees were assumed to overtop with no failures. Peak discharges at selected locations are shown in Table 11. Water surface profile plots with and without debris for the 25-, 50- and 100-year events are shown in Appendix C.

4.3 Infinite Levee Scenario

Simulations were also performed using HEC-RAS for "infinite levee" scenarios in which both levees and natural river banks are assumed to be of sufficient height to prevent all spill from the river. Levees are again assumed not to fail. Simulations were again performed for average roughness and the fixed debris loads from Table 3.

Discharges at selected locations in the system under the infinite levee scenarios are listed in Table 11. The infinite levee scenarios provide estimates of the channel capacity required under existing conditions in the absence of levee failures if all spill is prevented.

Table 11. HEC-RAS Hydraulic Model Results

	RM	2 -yr	5 -yr	10 -yr	25- yr	50- yr	75 -yr	100- yr	250- yr	500- yr
No Breach Scenario - Discharge (cfs) with Bridge Debris										
Sedro-Woolley	23.2	80,400	105,000	133,000	170,000	197,000	220,000	236,000	290,000	337,000
Sterling spill	21.59-22.269	0	0	0	900	12,200	23,600	30,900	52,300	72,500
Levee overtopping u/s BNSF bridge	17.89-20.89	0	0	0	100	7,700	23,900	35,800	69,800	97,500
Mount Vernon, Riverside Dr. bridge	17.04	77,000	91,300	117,000	149,000	163,000	166,000	167,000	168,000	169,000
South Fork	465	34,900	42,400	55,800	72,200	78,300	78,900	79,100	79,500	79,700
North Fork	829	41,700	48,900	61,200	76,600	82,500	83,200	83,400	83,800	84,100
No Breach Scenario – Discharge (cfs) without Bridge Debris										
Sedro-Woolley	23.2	80,500	105,000	134,000	171,000	197,000	219,000	235,000	290,000	337,000
Sterling spill	21.59-22.269	0	0	0	600	8,900	19,100	26,000	46,900	66,400
Levee overtopping u/s BNSF bridge	17.89-20.89	0	0	0	0	3,900	11,200	18,700	43,900	68,000
Mount Vernon, Riverside Dr. bridge	17.04	76,900	91,700	118,000	149,000	170,000	179,000	183,000	194,000	198,000
South Fork	465	34,900	42,600	56,200	72,300	79,900	80,800	81,100	81,400	81,500
North Fork	829	41,700	49,100	61,600	76,800	84,400	85,900	86,300	86,900	87,100
Infinite Levee Scenario - Discharge (cfs) with Bridge Debris										
Sedro-Woolley	23.2	80,300	105,000	133,000	170,000	197,000	220,000	236,000	289,000	325,000
Mount Vernon, Riverside Dr. bridge	17.04	76,900	92,900	119,000	150,000	168,000	192,000	206,000	245,000	283,000
South Fork	465	34,800	43,200	56,700	72,300	82,200	95,100	103,000	124,000	144,000
North Fork	829	41,600	49,500	62,100	77,100	85,400	96,800	103,000	121,000	138,000

5.0 References

Chow, V. T., 1959. Open-Channel Hydraulics. McGraw-Hill Book Co., New York, NY.

Collins, V.D., 2010. Memorandum to Lorna Ellestad, Skagit County, RE: Basemap Development, May 25, 2010. Northwest Hydraulic Consultants, Seattle, WA.

Cowan, W. L. 1956. Estimating Hydraulic Roughness Coefficients. Agricultural Engineering, vol. 37, no. 7, pp. 473-5, July 1956. In: Chow (1959).

Engman, E. T. 1986. Roughness Coefficients for Routing Surface Runoff. J. Irrigat. Drain. Eng. 112(1):39-53. In: (SCS, 1986), In: (Haan et al., 1994).

Federal Emergency Management Agency (FEMA), 1984. Flood Insurance Study, Skagit County, Washington, Unincorporated Areas, Community Number 530151.

Fischer, D., 2010. Memorandum for CENWS-PM-CP-CJ, Attn: Daniel Johnson, RE: Skagit River Levee Failure Points. From Dennis Fischer, P.E., Chief, Soils Section, Geotechnical and Environmental Restoration Branch, November 29, 2010.

Flo-2D Software Inc., 2009. Flo-2D Reference Manual. Flo-2D Software Inc., Nutrioso, AZ.

Golder Associates, 2009. Geotechnical Investigation and Levee Analysis, City of Burlington and Dike District 12 Levee Certification Project, Burlington Washington. Prepared for Pacific International Engineering, Edmonds, Washington, November 2009.

NHC, 2011. Conceptual Design Report for Cottonwood Slough Restoration Project. Prepared for Skagit Conservation District, Mt. Vernon, WA, January 2011.

NHC, 2013, Skagit River Basin General Investigation. Flood Risk Reduction – Hydraulic Analysis. Final Study Report. Prepared for Seattle District, USACE, August 2013.

Shannon & Wilson, 2011. Skagit River Levee General Investigation (GI), Levee Risk and Reliability Analysis, Prepared for Seattle District USACE, January 31, 2011.

U.S. Army Corps of Engineers, 1993. Engineering Design – River Hydraulics, EM-1110-2-1416, Washington, D.C.

U.S. Army Corps of Engineers, 2004. Draft Report, Hydraulic Technical Documentation, Skagit River Basin, Skagit River Flood Damage Reduction Feasibility Study, May 2004.

U.S. Army Corps of Engineers, 2010. HEC-RAS River Analysis System User's Manual, Version 4.1. January 2010.

U.S. Army Corps of Engineers, 2013. Hydrology Technical Documentation, Skagit River Basin, Skagit River Flood Risk Management Feasibility Study, August 2013.

West Consultants, 2001. Skagit River Cross-Section Comparison and Analysis. Unpublished memorandum to Seattle District USACE.

Woolpert, 2010. Survey Report, Group 5, Seattle District USACE, August 24, 2010.

APPENDIX A

BNSF Bridge Hydraulic Modeling

1 BNSF Bridge Hydraulic Modeling

1.1 Introduction

The Burlington Northern Santa Fe Railroad (BNSF) bridge is located just east (upstream) of the Interstate 5 and Riverside Drive bridges in Mt. Vernon. The BNSF bridge is the most important hydraulic structure in this reach of the river. The bridge has a relatively low deck elevation and a history of entrapping and retaining debris during high flows. A debris jam estimated at about 450 to 500 ft wide by 10 to 20 ft thick at its maximum formed on the bridge in the November 1995 flood[1], providing the basis for debris loading assumptions in recent hydraulic modeling for the Skagit River General Investigation (Skagit River GI). Previous HEC-RAS hydraulic modeling for the Skagit River GI, reported in the April 2011 draft Hydraulic Technical Documentation, showed head loss through the bridge of the order of three to four feet for 25-year events and larger under the assumed 500 ft wide by 20 ft thick debris blockage. Modeled backwater effects from the BNSF bridge extend upstream to approximately the Highway 9 crossing of the Skagit River at Sedro-Woolley, inducing additional flooding of the left bank Nookachamps basin and resulting in potentially substantial spill from the right bank of the Skagit in the vicinity of Sterling (from approximately RM 21 to RM 22 or roughly 12 to 13 miles upstream from the junction of the North and South Forks). Right bank spill upstream from the BNSF bridge flows north and west across the floodplain and does not re-enter the mainstem Skagit River. Previous modeling showed that spill amounts upstream from the BNSF bridge are quite sensitive to the head loss through the bridge. Large spills upstream from the BNSF bridge have the effect of reducing flows and hence flood risk downstream from the bridge. The hydraulic performance of the bridge is therefore a potentially critical factor in analysis and design of flood management measures and alternatives throughout the lower Skagit River.

The head loss through the BSNF bridge in previous modeling (of the order of three to four feet for 25-year events and larger, as noted above) is significantly larger than observed during recent large floods (November 1990, November 1995, October 2003 and November 2006) raising concerns about the reliability of previous modeling. The purpose of the work described in this report section was: to reexamine the computational approach to modeling the hydraulic performance of the bridge; to perform sensitivity analyses to determine how various model parameters and assumptions influence the computed water surface profile through the bridge opening; and to recommend a computational approach and set of model parameters and assumptions for future modeling. The following factors were considered in the sensitivity analyses:

- discharge rate (from approximately 120,000 to 320,000 cfs)
- debris blockage (from zero to 20,000 square feet)
- HEC-RAS contraction and expansion coefficients
- HEC-RAS right bank station placement
- Steady state vs. unsteady flow modeling

[1] The peak discharge during the 1995 flood was 141,000 cfs (about a 25-year return period) at the USGS Skagit River near Mount Vernon gage, located 0.5 miles downstream from the BNSF bridge.

Consideration was also given to scour potential at the bridge. The scour assessment was performed after the sensitivity studies and the order of presentation in this report reflects the order in which the work proceeded.

Part way through this work, it was determined, from examination of photographs from the November 1995 flood (see Section 1.2), that the existing condition bridge geometry in the HEC-RAS model was incorrect, and had apparently been incorrect for many years. Before completing the sensitivity analyses, additional work was therefore conducted to survey the bridge, update the representation of the bridge geometry in the HEC-RAS model, and reassess the model calibration.

1.2 Bridge Survey and Reassessment of HEC-RAS Model Calibration

1.2.1 Bridge Survey

NHC staff completed a partial survey of the BNSF bridge on 6 November 2012. For access and safety reasons, the survey was restricted to the right bank piers and right bank low chord. Survey grade GPS was used to establish control points from which a survey level was used to determine elevations. A nearby WSDOT monument was surveyed before and after the bridge survey as a quality assurance check. The low chord elevation of the over-water spans of the bridge were subsequently estimated from spot elevations of the bridge deck taken from aerial mapping obtained from the City of Burlington, dated 2009.

Substantial differences exist between the surveyed and previously assumed bridge geometry, as shown in Figure A-1 and Figure A-2. The main differences are: 1) the bridge deck is approximately 6.4 feet thick, not 10 feet as previously assumed; 2) the bridge deck has a vertical curve, with the right bank deck about three feet lower than the deck in the main channel area; and 3) the low chord is significantly lower than previously assumed (about six feet lower in the right overbank area and 3 feet lower over the main channel). The lower low chord elevation is of particular importance since it results in the bridge going into pressure flow at a lower discharge than previously assumed. The low chord elevation varies from 43.01 ft NAVD88 in the right overbank area to 45.51 ft NAVD88 over the main channel and approximately 47.5 ft NAVD88 at the Whitmarsh Road underpass on the right bank. Bridge overtopping elevations vary from 49.37 ft NAVD88 in the right overbank to 51.87 ft NAVD88 over the main channel.

The results from the level survey were processed, the bridge geometry revised in the HEC-RAS model, and the model calibration re-assessed, prior to sensitivity analysis.

1.2.2 1995 High Water Data and Model Calibration

The error in the bridge geometry in the previous HEC-RAS model was identified from photographs of the November 1995 flood (Figure A-3 and Figure A-4). These photographs, taken close to the peak of the flood (reportedly at 12:30 pm on 30 November 1995), show the maximum water level close to the low chord of the bridge and inconsistent with the previous model's representation of the bridge.

After surveying the bridge low chord and piers, the photographs were used to estimate a November 1995 high water elevation on the upstream face of the bridge of 41.66 ft NAVD88. This high water mark

(HWM "A") is deemed more reliable than a previously reported high water mark of 42.97 ft downstream of the bridge (HWM "B"), and was given substantial weight when re-assessing the model calibration. In addition to the new high water mark, the same set of photographs shows that there was no significant debris accumulation *during the flood peak*. Photographs of the large debris jam which developed during this event were evidently taken on the receding limb of the flood, some time after the peak stage occurred. For this reason, the 4,000 square foot debris blockage previously assumed in model calibration to the 1995 flood data was removed.

The changes described above (changing the bridge geometry, removing the debris blockage, and adding a new high water mark) were significant enough to warrant a reassessment of the model calibration. Figure A-5 shows 1995 water surface profiles for the previous calibration and for the updated model configuration. This flood did not reach the low chord of the bridge, and changes to the bridge pier geometry were minor. Therefore, the approximately 0.75 foot reduction in backwater seen in Figure A-5 is due to removal of the debris blockage and changing the "low flow" solution method from momentum to energy (see definitions and discussion of computational approaches in Section 1.3 below). Downstream of the bridge, the difference is minimal. The impact on the model calibration of the change in bridge geometry is minimal because the new, lower deck elevation is still above the peak water level for the 1995 flood, and so free surface flow is maintained.

Overall, calibration to the 1995 high water data was somewhat improved by the above changes. All high water marks discussed here are shown on Figure A-5. The newly identified high water mark on the upstream face of the BNSF bridge (HWM "A"), determined from the photographs, matches the revised water surface profile reasonably well. The high water mark just below the bridge (HWM "B"), was discounted because it is inconsistent with the photographic evidence and would result in an implausibly large water surface slope between the BNSF bridge and the downstream high water mark (HWM "G") taken from the USGS stream gage just downstream from the Riverside Drive bridge. HWM "C" is not considered valid and was discounted as an outlier. Two of the remaining three high water marks between the BNSF bridge and Highway 9 (HWMs "D","E" and "F") are better replicated by the revised model. Upstream of Highway 9, the differences between the models dissipate.

The 2003 and 2006 floods were also re-run with the new bridge geometry and energy solution method. For both of these floods, simulated water surface elevations were reduced around 0.2 feet in the Nookachamps area, slightly improving the calibration in this area. As neither of these floods was modeled with a debris load and as neither flood reached the low chord of the bridge, these changes are attributed to a change in solution method. The minor change in the bridge geometry to reflect the pilings driven around the pier that failed in the 1995 flood is unlikely to affect the computed water surface profile.

1.3 Investigation of HEC-RAS Bridge Computational Methods

The original scope of work called for investigating all bridge modeling approaches available in HEC-RAS as part of the sensitivity testing. HEC-RAS allows the use of different computational methods for "low" and "high" flow at bridges. "Low" flow is defined as flow under the bridge with water surface elevations not reaching the bridge low chord. For most significant bridges, this would include all but the largest flood discharges. “High” flow is when water surface elevations result in pressure flow under the bridge and potentially additional weir flow over the deck. For low flows, HEC-RAS modeling options are:

- energy balance
- momentum balance
- Yarnell method
- WSPRO Method

For high flows (pressure flow under the bridge, and weir flow over the bridge), options are:

- energy balance
- pressure/weir method

In addition, HEC-RAS allows the option of converting all bridges in a model to lidded cross sections.

In the course of this work, problems were encountered in application of most of the modeling options. Converting bridges to lidded cross sections was determined not to be an option for this particular application because debris blockage is not accounted for in the conversion process.

For low flow methods, efforts to use the WSPRO method were unsuccessful; the model crashed when using this option. The momentum method gave numerous warnings regarding invalid solutions, although results were still reported. The bridge modeling situation was discussed with Dr. Gary Brunner at the USACE Hydrologic Engineering Center. His opinion was that the debris blockages being modeled exceeded the range for which the Yarnell and momentum methods were appropriate and recommended use of the energy method. He also noted that placing all debris in a single block such that it covered multiple adjacent piers, as in previous modeling, would result in incorrect results. Debris geometries were therefore modified to use multiple debris blockages sized to ensure no overlap with adjacent piers or debris. The energy method was tested over the full range of debris blockages (0 to 20,000 square feet) for low flow conditions and was found to give apparently reasonable results without error or caution notes. Therefore this method was used for all subsequent low flow sensitivity testing.

Under high flow conditions, the energy method resulted in numerous error and cautions. The pressure/weir flow method produced somewhat higher headwater results, but did not exhibit the same computational issues. High flow modeling methods were also discussed with Dr. Brunner and he concurred that the pressure/weir flow method should be used. All sensitivity testing discussed herein uses this method for high flows. It was determined that the most appropriate trigger elevation to use for pressure/weir flow calculations was the main bridge span low chord elevation of 45.5 ft NAVD88, as opposed to the highest low chord elevation which occurs on the span crossing Whitmarsh Road near the right bank (Station 1000, Figure A-2).

1.4 Sensitivity Analysis

Sensitivity analyses were conducted on a number of key model variables. For comparison, a “base” case was selected. This consisted of 10,000 square feet of debris blockage, the right bank station set at 727 feet (i.e. at the edge of the low flow channel – see Figure A-2), and contraction/expansion coefficients of 0.1 and 0.3 respectively. The amount of debris blockage, right bank station and contraction/expansion coefficients were then varied systematically to explore the sensitivity of the bridge to the various parameters. In all cases, only one variable was changed per run. The base case right bank station and contraction/expansion coefficients were as used in the model calibration.

The existing condition HEC-RAS model was modified for the sensitivity analysis in order to allow evaluation of BNSF bridge performance under extremely high flows. The right bank levees between Sedro Woolley and the bridge were removed to prevent overtopping flows from leaving the model domain upstream from the bridge. The left bank Nookachamps storage areas were also disconnected in order to improve model speed and stability at high flows. Levees downstream of the bridge were left at their current (existing condition) height. As a result of the miles of overtopping levee downstream from the bridge, tailwater elevations are very similar over a large range of high flows. Minor modification to the levee geometry at the bifurcation of the North and South Forks was also required in order to allow the model to run in HEC-RAS Version 4.1 (previous analyses used Version 4.0).

The 500-year Average Regulation Condition Flood from the March 2011 Hydrology Technical Documentation was run for each scenario in order to obtain results over a wide range of flow, including the transition from low flow to high flow hydraulics. Results are presented as ratings curves, selected water surface profiles, and in tabular form as described below. It should be noted that in the rating curve plots, a small hysteresis loop is evident in all runs. The water surface profile plots use nominal flow rates for each profile. Because the model was run in unsteady mode, flows between runs were never exactly the same for a given time step; therefore the water surface profile figures show results for flows that are within a few of percent of each other but not equal. This causes slight variations in results (including tail water elevations), but the dominant variation by far in each comparison is due to the variable being tested. Model results for each group of sensitivity runs are discussed in sections 1.4.1 through 1.4.4 below. Table A-1 gives detailed hydraulic output results at the bridge for all sensitivity runs over the full range of flows.

Interpretation of data in Table A-1 requires some care. In particular, it will be noted that the data show some significant variations in the elevation above which pressure/weir flow calculations are used. Several factors appear to affect the apparent switch to pressure flow as reported in Table A-1:

i) The model will default to energy calculations if a valid pressure/weir solution cannot be found – this appears to be occurring at the transition to pressure flow.

ii) the upstream water level reported in Table A-1 is from the cross-section immediately upstream from the bridge, whereas the trigger for switching to pressure flow is at a cross-section internal to the bridge.

iii) Model output is reported at an hourly time step so reporting of the change from energy to pressure flow may be up to one hour later than actual.

1.4.1 Model Sensitivity to Debris Blockage

The BNSF bridge has demonstrated a propensity for spurring the formation of debris jams during high flows. The debris jams do not occur during every large flood, however, and, as shown in the 1995 event, the bridge may remain clear of debris during the peak flow but trap debris later in the event. Because a debris jam could potentially influence discharge rates and water levels upstream and downstream of the bridge, a sensitivity analysis of various debris blockages was conducted. The debris jam sizes considered were: 0, 3000, 6000, 8000, 10000, 14000, and 20000 square feet. Debris was distributed over a number of piers, and the areas quoted above are in addition to the blockage due to the piers themselves. To the extent possible, debris was placed to avoid encroachment on that portion of the main channel between the left bank and Pier 1 (piers are numbered from left to right looking downstream) consistent with past observations. However, because of their size, this was not possible with the 14,000 and 20,000 square foot blockages. The placement of debris for the various size blockages is shown in Figure A-6 through Figure A-11.

Figure A-12 shows the tail water rating downstream from the bridge and rating curves immediately upstream of the bridge with the various blockage configurations and for flows ranging from 30,000 to roughly 320,000 cfs. The tail water rating follows the familiar convex curve. The break in slope and flattening of the tail water rating at a flow of about 175,000 cfs corresponds to the overtopping of the levee system downstream from the bridge. With no debris, the upstream rating closely follows the tail water rating up to a flow of about 220,000 cfs. Above that point, the transition to pressure flow results in an increase in head loss through the bridge opening and a divergence of the upstream and downstream ratings. As debris is added and as the degree of blockage increases, a more severe “step” forms in the upstream rating curves as the transition to pressure flow occurs at lower and lower flows. Debris blockages from 6,000 to 14,000 square feet trigger pressure flow conditions at discharges in the 150,000 to 170,000 cfs range. Figure A-13, Figure A-14 and Figure A-15 show water surface profiles with the various debris blockages at flows of approximately 150,000 cfs, 200,000 cfs and 250,000 cfs respectively.

1.4.2 Model Sensitivity to Contraction and Expansion Coefficients

In unsteady flow models, HEC-RAS develops families of rating curves for each bridge that represent the full range of flows and headwater stage under various tail water elevations. The curves are calculated based on the bridge modeling method chosen, each of which has key parameters that may be varied. For energy method calculations, the contraction and expansion coefficients are multiplied by the change in velocity head between sections to estimates losses. For bridges with large changes in velocity due to contracted openings, the model solution can be quite sensitive to these coefficients. To test the sensitivity to these parameters, simulations were conducted with three sets of coefficients for contraction and expansion: 0.1/0.3 (the base condition), 0.3/0.5, and 0.5/0.7. The rating curves resulting from these simulations are shown in Figure A-16 and water surface profiles at selected discharges are shown in Figure A-17.

As can been in the rating curves (Figure A-16), for flows up to about 140,000 cfs, altering the coefficients has an approximately linear impact on the upstream rating curves (i.e., the difference between 0.1/0.3

and 0.3/0.5 is about the same as the difference between 0.3/0.5 and 0.5/0.7), along with the expected result that higher coefficients lead to greater head loss and less efficient conveyance. Above about 170,000 cfs, the bridge transitions fully to pressure flow and a HEC-RAS computational approach which does not make use of contraction/ expansion coefficients, hence the solutions converge, as can be seen in both Figure A-16 and Figure A-17.

As noted previously, the set of contraction/expansion coefficients used in the model calibration was 0.1/0.3.

1.4.3 Model Sensitivity to Right Bank Station

Simulations were conducted to assess the impact of the placement of the right bank station immediately upstream and downstream of the BNSF bridge. The right bank in this area is a flat low lying field (see Figure A-2) which floods at a flow of roughly 50,000 cfs. The area typically has a healthy grass cover but may be covered by sand, which deposits preferentially in this area during floods. The bank station represents the transition between channel and overbank areas, and the choice is a somewhat subjective matter in this case. HEC-RAS uses the bank station as a change in roughness location, as well as a partitioning tool when dividing the cross-section into sections for computation. Two locations for the bank station were tested: the existing location near the edge of the low-flow channel, and at the right edge of the cross-section, which is approximately the edge of water during extremely high flows. The scenario with the bank station placed at the right edge of the cross-section has a channel n-value (0.034) extended across the floodplain to the bank station.

Figure A-18 shows the rating curves resulting from these two scenarios and Figure A-19 shows corresponding water surface profiles for flows of 150,000, 200,000 and 250,000 cfs. Results from the two scenarios are almost indistinguishable. This is in large part because under the assumed 10,000 square foot debris load, flow across much of the right bank area in question is blocked by debris. Greater differences would be expected under lower debris loads.

1.4.4 Steady State vs. Unsteady Flow Modeling

Simulations were performed in steady state mode for flows of 150,000, 200,000 and 250,000 cfs for each of the sensitivity scenarios described in Sections 1.4.1 through 1.4.3 above, and water surface profiles were then compared against the corresponding results from the unsteady flow runs. The results of the steady state simulations and the corresponding unsteady flow simulations were essentially identical. Because the results of the steady and corresponding unsteady flow simulations are so close, comparison plots are not included in the report.

1.5 Effects of Bed Scour on Bridge Hydraulics

All simulations described above assume a fixed channel bed, but a brief review of hydraulic outputs and available sediment data indicates that it is likely that significant scour takes place at the bridge under flood flow conditions. The failure of Pier 8 of the bridge due to scour in the 1995 flood provides additional evidence supporting this hypothesis.

Sediment sampling of the entire lower river system was undertaken as part of a geomorphology task for the Skagit GI study in 2002[2]. Multiple grab samples and full transect bed material samples were obtained at or around the bridge. The study results indicate that the bridge is located within the gravel-sand transition of the Skagit River. The report states that the mean bulk sample D_{50} was 5.4 mm upstream of the bridge and 0.6 mm downstream. Bed material samples at this location and further upstream indicate a finer gradation than the bulk surface samples; a D_{50} of less than 1 mm is reported. Recent work by the USGS[3] sampling at the next bridge downstream confirms the sand bed nature of the channel below the BNSF bridge.

Scour potential was investigated for a no-debris and 10,000 square foot debris (base case) scenario. Approach velocities to the bridge are in the range of 6 to 9 feet/second for flows from 150,000 cfs to 250,000 cfs under these scenarios (Figure A-20). This range of flows is considered to include the range of greatest interest for analysis of the various potential flood management alternatives. In the bridge opening, velocities increase slightly under the no-debris condition to values in the range of from 7 to 11 feet/second, while with debris, the velocity increases to a maximum of about 16 feet/second (Figure A-21). Estimates of potential scour due to general and contraction scour only were generated using the hydraulic design tools in HEC-RAS and some external references. Local abutment and pier scour were not evaluated.

Results using the contraction scour tool in HEC-RAS for the main channel only are presented in Table A-2 for a flow 150,000 cfs. A conservative D_{50} of 10mm was used (this is the single largest bulk sample value from the vicinity of the bridge) and scour was forced to be live bed. The estimated scoured area was calculated by multiplying the scour depth by the wetted perimeter of main channel (excluding piers) in the cross section.

The analysis has a few notes of interest:

i) Scour is predicted to occur even with no debris on the piers.

ii) Scour area is 70% of debris blockage area. (Note this is at a relatively low flood flow of 150,000 cfs; the bridge is not in pressure flow and velocities are at their minimum [Figure A-21]).

iii) No right overbank scour is calculated, but the pier failure on this overbank in 1995 is evidence that if flows are sufficient to strip away the vegetative cover, significant scour would also be expected here. (Note, however, that we have no information on the nature or condition of the pier foundation.)

---

[2] Cherry, S. and Jackson, G., 2002. Geomorphology and Sediment Transport Study of Skagit River Flood Hazard Mitigation Project -Skagit County, Washington: Phase 1 INTERIM REPORT. Prepared for U.S. Army Corps of Engineers, Seattle District by Pentec Environmental. December 20, 2002.

[3] Curran, C.A., Grossman, E.E., and Mastin, M.C., 2009, Measurements of suspended sediment and flow distribution with implications for habitat restoration in the Skagit River Delta, Washington: Seattle, Washington, 2009 Puget Sound Georgia Basin Ecosystem Conference, February 8-11, 2009. Obtained at http://puget.usgs.gov/posters/Curran+Grossman+Mastin.Meas-Susp-Sed_poster.pdf .

The scour calculations indicate that most of the waterway area reduction from the debris blockage is likely to be compensated by scour of the bed. Not accounted for in these calculations are areas that may be resistant to scour, either from natural bedrock outcroppings or riprap placed over time by the railroad. It is known that the piers in the main channel are protected by riprap (see the 1993 low water photograph in Figure A-22). Nevertheless, unless the entire channel is armored under the bridge it seems likely that extensive bed scour will occur under flood conditions.

As stated as the beginning of this section, all simulations performed in this work assumed a fixed channel bed. It should also be noted that no modeling was performed for the “with scour” condition.

1.6 Further Refinements to HEC-RAS Model Representation of BNSF Bridge

At the conclusion of the sensitivity runs, the following additional refinements were made to the model representation of the BNSF bridge:

i) Skew of approximately 10° was applied to the bridge and the cross-sections immediately upstream and downstream. The correction for skew results in a slight reduction in the effective channel width.

ii) The pier spacing was adjusted to more closely reflect actual spacing based on measurements from aerial photographs.

iii) The shapes of piers 4 through 12 (piers are numbered from left to right looking downstream) were modified (tapered) to more closely reflect the actual pier shapes. Piers 1 through 3 were already tapered in the model.

The final bridge geometry is shown in Figure A-23. By comparison with Figure A-2, it can be seen that the principal changes are in the spacing of the main channel piers 1 through 3, an increase in the effective pier widths as a result of the skew adjustment, and a slight reduction in channel width. The impact of these refinements on bridge hydraulics is illustrated in Figure A-24, Figure A-25 and Figure A-26 which show rating curves with the changes (“Skewed Bridge”) and for the original sensitivity runs (“Before Skew Adjustment”) for scenarios with no debris and with 3,000 and 6,000 sq. ft. of debris. All runs assumed 0.1/0.3 contraction/expansion coefficients and the right bank station at the edge of the low flow channel. The changes (primarily the skew adjustment) result in the bridge transitioning to pressure/weir flow at a somewhat lower discharge and a slightly higher stage for a given discharge. These changes do not affect the conclusions and recommendations presented in Section 1.7 based on the sensitivity runs and assessment of scour potential.

1.7 Conclusions and Recommendations

Debris accumulation at the BNSF bridge is highly variable both from flood to flood and within individual flood events. The largest documented blockage in the recent past formed during the flood of November 1995. This event had a peak flow of 141,000 cfs at Mt. Vernon for a return period of approximately 25-years.

Photographs taken during the 1995 flood indicate that the bridge was clear of debris at the time of the peak flow and that the debris jam (subsequently estimated as having maximum dimensions of approximately 450 to 500 ft wide by 10 to 20 ft deep) formed over a relatively short period of time on the receding limb of the flood hydrograph. We speculate that the jam initially formed as a raft of debris lodging on the bridge piers and then trapping other debris moving down river. There is nothing to indicate that the debris jam could not have formed earlier in the event and been in place at the time of the peak discharge. Selection of parameters to model the hydraulic performance of the BNSF bridge should therefore consider scenarios with and without debris blockage.

As shown in the 1995 flood, the BNSF bridge is capable of collecting and building impressive debris jams in a short amount of time. Long term trends will likely increase both the total volume and individual log sizes in the flood-borne debris load. This is due to projected increases in peak flows and hence channel migration associated with climate change, and as the numerous restoration projects on the Skagit River banks mature and begin to provide increasingly large conifers to the river. Debris accumulation on the bridge is a very real risk, however extrapolation of debris loads to extreme flood conditions is a speculative endeavor.

Balancing the impacts on bridge hydraulics of debris accumulation is the expectation that the river bed in the vicinity of the bridge is highly mobile under flood conditions and can be expected to adjust to debris blockage through scour. Analysis of scour potential (Section 1.5) for a scenario with a flow of 150,000 cfs and a debris blockage of 10,000 square feet, resulted in a scour area of approximately 7,000 square feet, or 70% of the debris blockage area. We would expect scour depth and area to increase as both blockage size and discharge increase.

Since the HEC-RAS model is not capable of simulating a mobile bed with the unsteady flow computations (HEC-RAS does have sediment transport modeling capability however this is for “quasi-unsteady” mode and is typically used for estimating long term trends), the effects of scour in scenarios with debris blockage can be most readily accounted for by reducing the assumed blockage area by the estimated scour area. Based on the analysis of scour potential, for example, the hydraulic performance with a 10,000 square foot blockage could be modeled using a net 3,000 square foot blockage, assuming 7,000 square feet of scour area.

Ratings upstream and downstream from the bridge with no blockage and with a 3,000 square foot blockage are shown in Figure A-25 for the simulations with and without skew adjustment. The impacts of a 3,000 square foot blockage on the upstream rating are insignificant until the flow reaches about 190,000 cfs without skew adjustment and roughly 175,000 cfs with skew adjustment, above which the ratings with and without debris start to diverge. Note that the flattening in the downstream rating at flows above 175,000 cfs is the result of overtopping of levees downstream from the bridge. The downstream levees were kept at their existing height for the purposes of this analysis.

Increasing the blockage area by 50% to 15,000 square feet and continuing to assume a scour area of 70% of the blockage, would result in a net blockage for modeling purposes (i.e. after accounting for scour) of 4,500 square feet. Interpolating from the suite of ratings in Figure A-12 shows that the impacts of this blockage are minor until the flow reaches about 180,000 cfs (roughly 170,000 cfs with skew adjustment), above which the ratings with and without debris again start to diverge.

If scour offsets the effects of debris blockage to the extent estimated here, then it appears that the hydraulic performance of the BNSF bridge would be relatively insensitive to debris load over a wide range of blockage sizes for flows up to at least 160,000 cfs. Previous hydraulic modeling without debris loads shows that flows much greater than this magnitude are unlikely at the BNSF bridge under existing conditions because of spill over the upstream Dike District 12 levees and from the right bank of the Skagit in the vicinity of Sterling (RM 21 to RM 22 or from 3.5 to 4.5 miles upstream from the BNSF bridge). Measures which would allow passage of flows on the order of 200,000 cfs and greater would include raising upstream and downstream levees and construction of a right bank levee at Sterling. Raising the downstream levees would change the downstream bridge rating and affect the bridge hydraulic performance as characterized in this report for flows greater than about 175,000 cfs.

Given the various uncertainties in the size of debris blockages, potential scour depths, and the nature of future flood management measures we recommend adoption of a fixed design debris blockage of 3,000 square feet for current feasibility studies. For the flow range of greatest interest, this produces upstream water levels only slightly higher than scenarios without debris. Recognizing the very limited scour analysis undertaken here and the current lack of detailed information on bed conditions at the bridge (e.g., while the bridge piers are known to have riprap protection, there is no detailed information on the size or extent of existing scour protection), this assumption should only be used for feasibility study purposes and should be revisited before more detailed design is undertaken.

With regard to other hydraulic model parameters, we recommend that the contraction/expansion coefficients remain set at 0.1/0.3 as in model calibration, and that the model's right bank location remain at the edge of the low flow channel also as in model calibration. In both cases, we see no strong justification for departing from the calibration values. Further, model results are insensitive to the right bank station location.

Finally, we recommend that future bridge modeling for this study use the energy approach for low flows and pressure/weir flow for high flows. These methods were found to be robust and to produce plausible results for the full range of flows and blockage conditions examined.

Table A-1: Selected Bridge Hydraulics Output Assuming No Scour

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
Base	20OCT2003 0600	37.17	36.80	Energy only	94615		94615	8.8
Base	20OCT2003 0700	37.31	36.92	Energy only	95515		95515	8.8
Base	20OCT2003 0800	37.44	37.05	Energy only	96434		96434	8.8
Base	20OCT2003 0900	37.58	37.19	Energy only	97391		97391	8.9
Base	20OCT2003 1000	37.75	37.36	Energy only	98689		98689	8.9
Base	20OCT2003 1100	37.98	37.58	Energy only	100433		100433	9.0
Base	20OCT2003 1200	38.24	37.83	Energy only	102278		102278	9.1
Base	20OCT2003 1300	38.62	38.20	Energy only	105269		105269	9.3
Base	20OCT2003 1400	39.00	38.57	Energy only	108169		108169	9.4
Base	20OCT2003 1500	39.45	39.01	Energy only	111632		111632	9.5
Base	20OCT2003 1600	39.92	39.47	Energy only	115271		115271	9.7
Base	20OCT2003 1700	40.33	39.87	Energy only	118052		118052	9.7
Base	20OCT2003 1800	40.58	40.12	Energy only	119749		119749	9.7
Base	20OCT2003 1900	40.87	40.40	Energy only	122022		122022	9.8
Base	20OCT2003 2000	41.22	40.73	Energy only	124829		124829	9.8
Base	20OCT2003 2100	41.87	41.36	Energy only	130956		130956	10.0
Base	20OCT2003 2200	42.64	42.11	Energy only	137473		137473	10.2
Base	20OCT2003 2300	43.42	42.87	Energy only	144060		144060	10.3
Base	20OCT2003 2400	44.20	43.62	Energy only	150994		150994	10.4
Base	21OCT2003 0100	44.98	44.38	Energy only	158324		158324	10.5
Base	21OCT2003 0200	45.88	45.29	Energy only	162146		162146	10.6
Base	21OCT2003 0300	46.94	46.37	Press/Weir	164504	56	164560	9.5
Base	21OCT2003 0400	48.08	47.52	Press/Weir	167572	78	167650	9.7
Base	21OCT2003 0500	49.28	48.75	Press/Weir	171049	106	171154	9.9
Base	21OCT2003 0600	50.35	49.75	Press/Weir	186599	334	186933	10.8
Base	21OCT2003 0700	51.07	50.34	Press/Weir	209355	974	210330	12.1
Base	21OCT2003 0800	51.77	50.96	Press/Weir	223182	1635	224817	12.9
Base	21OCT2003 0900	52.41	51.53	Press/Weir	235968	2700	238668	13.6
Base	21OCT2003 1000	53.00	52.05	Press/Weir	246403	5369	251772	14.2
Base	21OCT2003 1100	53.54	52.51	Press/Weir	255444	8336	263780	14.7
Base	21OCT2003 1200	54.01	52.92	Press/Weir	263436	11482	274918	15.2
Base	21OCT2003 1300	54.42	53.28	Press/Weir	270256	14565	284820	15.6
Base	21OCT2003 1400	54.76	53.57	Press/Weir	276064	17421	293485	15.9
Base	21OCT2003 1500	55.05	53.81	Press/Weir	280773	19993	300765	16.2
Base	21OCT2003 1600	55.29	54.02	Press/Weir	284280	21716	305996	16.4
Base	21OCT2003 1700	55.47	54.18	Press/Weir	286897	23171	310068	16.5
Base	21OCT2003 1800	55.60	54.30	Press/Weir	288634	24283	312917	16.6
Base	21OCT2003 1900	55.69	54.37	Press/Weir	289742	25008	314750	16.7
0debris	20OCT2003 0600	36.93	36.53	Energy only	95523		95523	5.1

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
0debris	20OCT2003 0700	37.04	36.65	Energy only	96295		96295	5.1
0debris	20OCT2003 0800	37.17	36.77	Energy only	97188		97188	5.1
0debris	20OCT2003 0900	37.30	36.89	Energy only	98135		98135	5.1
0debris	20OCT2003 1000	37.44	37.03	Energy only	99150		99150	5.1
0debris	20OCT2003 1100	37.59	37.18	Energy only	100377		100377	5.2
0debris	20OCT2003 1200	37.80	37.38	Energy only	102031		102031	5.2
0debris	20OCT2003 1300	38.11	37.68	Energy only	104554		104554	5.3
0debris	20OCT2003 1400	38.53	38.08	Energy only	108074		108074	5.3
0debris	20OCT2003 1500	39.00	38.54	Energy only	111744		111744	5.4
0debris	20OCT2003 1600	39.48	39.00	Energy only	115523		115523	5.5
0debris	20OCT2003 1700	39.97	39.49	Energy only	119485		119485	5.5
0debris	20OCT2003 1800	40.29	39.80	Energy only	121391		121391	5.5
0debris	20OCT2003 1900	40.56	40.07	Energy only	123525		123525	5.6
0debris	20OCT2003 2000	40.89	40.38	Energy only	126242		126242	5.6
0debris	20OCT2003 2100	41.32	40.79	Energy only	130203		130203	5.7
0debris	20OCT2003 2200	42.13	41.57	Energy only	137713		137713	5.8
0debris	20OCT2003 2300	42.91	42.33	Energy only	144586		144586	5.9
0debris	20OCT2003 2400	43.69	43.09	Energy only	151729		151729	6.0
0debris	21OCT2003 0100	44.48	43.85	Energy only	159297		159297	6.2
0debris	21OCT2003 0200	45.28	44.62	Energy only	167101		167101	6.4
0debris	21OCT2003 0300	46.12	45.44	Energy only	174426		174426	6.5
0debris	21OCT2003 0400	46.96	46.24	Energy only	183725		183725	6.8
0debris	21OCT2003 0500	47.62	46.81	Energy only	199120		199120	7.4
0debris	21OCT2003 0600	48.20	47.30	Energy only	213478		213478	7.9
0debris	21OCT2003 0700	48.79	47.82	Energy only	224243		224243	8.3
0debris	21OCT2003 0800	49.41	48.39	Energy only	233942		233942	8.6
0debris	21OCT2003 0900	49.99	48.89	Press/Weir	246015	33	246048	9.1
0debris	21OCT2003 1000	50.41	49.18	Press/Weir	262857	273	263130	9.7
0debris	21OCT2003 1100	50.70	49.33	Press/Weir	277898	647	278544	10.3
0debris	21OCT2003 1200	51.00	49.56	Press/Weir	286891	959	287850	10.6
0debris	21OCT2003 1300	51.27	49.76	Press/Weir	294974	1292	296266	10.9
0debris	21OCT2003 1400	51.50	49.93	Press/Weir	301910	1625	303534	11.1
0debris	21OCT2003 1500	51.69	50.08	Press/Weir	307563	1956	309519	11.3
0debris	21OCT2003 1600	51.84	50.19	Press/Weir	311726	2262	313987	11.5
0debris	21OCT2003 1700	51.95	50.28	Press/Weir	314723	2488	317211	11.6
0debris	21OCT2003 1800	52.03	50.34	Press/Weir	316630	2682	319312	11.7
0debris	21OCT2003 1900	52.06	50.36	Press/Weir	317515	2787	320302	11.7
3kdebris	20OCT2003 0600	36.99	36.60	Energy only	95238		95238	6.1
3kdebris	20OCT2003 0700	37.11	36.72	Energy only	96108		96108	6.1
3kdebris	20OCT2003 0800	37.24	36.84	Energy only	97017		97017	6.1

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
3kdebris	20OCT2003 0900	37.37	36.97	Energy only	97971		97971	6.1
3kdebris	20OCT2003 1000	37.51	37.11	Energy only	98987		98987	6.1
3kdebris	20OCT2003 1100	37.68	37.27	Energy only	100356		100356	6.2
3kdebris	20OCT2003 1200	37.93	37.51	Energy only	102472		102472	6.2
3kdebris	20OCT2003 1300	38.21	37.79	Energy only	104555		104555	6.2
3kdebris	20OCT2003 1400	38.65	38.21	Energy only	108189		108189	6.3
3kdebris	20OCT2003 1500	39.10	38.65	Energy only	111797		111797	6.4
3kdebris	20OCT2003 1600	39.58	39.11	Energy only	115549		115549	6.4
3kdebris	20OCT2003 1700	40.07	39.59	Energy only	119477		119477	6.5
3kdebris	20OCT2003 1800	40.36	39.87	Energy only	121101		121101	6.5
3kdebris	20OCT2003 1900	40.63	40.14	Energy only	123283		123283	6.5
3kdebris	20OCT2003 2000	40.96	40.45	Energy only	126031		126031	6.5
3kdebris	20OCT2003 2100	41.44	40.91	Energy only	130674		130674	6.6
3kdebris	20OCT2003 2200	42.23	41.68	Energy only	137760		137760	6.7
3kdebris	20OCT2003 2300	43.01	42.44	Energy only	144581		144581	6.8
3kdebris	20OCT2003 2400	43.79	43.19	Energy only	151675		151675	6.9
3kdebris	21OCT2003 0100	44.58	43.95	Energy only	159185		159185	7.1
3kdebris	21OCT2003 0200	45.37	44.72	Energy only	166943		166943	7.2
3kdebris	21OCT2003 0300	46.21	45.54	Energy only	174222		174222	7.4
3kdebris	21OCT2003 0400	47.06	46.35	Energy only	183275		183275	7.6
3kdebris	21OCT2003 0500	47.79	47.01	Energy only	195677		195677	8.1
3kdebris	21OCT2003 0600	48.62	47.82	Energy only	203269		203269	8.5
3kdebris	21OCT2003 0700	49.45	48.62	Press Only	212454		212454	8.8
3kdebris	21OCT2003 0800	50.16	49.24	Press/Weir	227332	227	227559	9.4
3kdebris	21OCT2003 0900	50.66	49.60	Press/Weir	246870	283	247153	10.2
3kdebris	21OCT2003 1000	51.04	49.85	Press/Weir	262949	778	263727	10.9
3kdebris	21OCT2003 1100	51.36	50.07	Press/Weir	275861	1379	277240	11.5
3kdebris	21OCT2003 1200	51.68	50.32	Press/Weir	284390	1894	286284	11.8
3kdebris	21OCT2003 1300	51.98	50.56	Press/Weir	292009	2481	294490	12.1
3kdebris	21OCT2003 1400	52.24	50.77	Press/Weir	298425	3226	301651	12.4
3kdebris	21OCT2003 1500	52.46	50.94	Press/Weir	303613	4040	307652	12.6
3kdebris	21OCT2003 1600	52.63	51.08	Press/Weir	307578	4776	312354	12.8
3kdebris	21OCT2003 1700	52.76	51.18	Press/Weir	310384	5398	315781	12.9
3kdebris	21OCT2003 1800	52.85	51.25	Press/Weir	312212	5872	318084	13.0
3kdebris	21OCT2003 1900	52.89	51.29	Press/Weir	313187	6102	319289	13.0
6kdebris	20OCT2003 0600	37.11	36.73	Energy only	94866		94866	7.5
6kdebris	20OCT2003 0700	37.24	36.85	Energy only	95765		95765	7.5
6kdebris	20OCT2003 0800	37.37	36.98	Energy only	96690		96690	7.5
6kdebris	20OCT2003 0900	37.50	37.11	Energy only	97657		97657	7.5
6kdebris	20OCT2003 1000	37.66	37.26	Energy only	98840		98840	7.6

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
6kdebris	20OCT2003 1100	37.84	37.44	Energy only	100219		100219	7.6
6kdebris	20OCT2003 1200	38.12	37.71	Energy only	102449		102449	7.7
6kdebris	20OCT2003 1300	38.47	38.05	Energy only	105348		105348	7.7
6kdebris	20OCT2003 1400	38.85	38.42	Energy only	108310		108310	7.7
6kdebris	20OCT2003 1500	39.30	38.85	Energy only	111863		111863	7.8
6kdebris	20OCT2003 1600	39.76	39.30	Energy only	115583		115583	7.8
6kdebris	20OCT2003 1700	40.21	39.74	Energy only	118992		118992	7.8
6kdebris	20OCT2003 1800	40.46	39.99	Energy only	120569		120569	7.8
6kdebris	20OCT2003 1900	40.74	40.26	Energy only	122819		122819	7.8
6kdebris	20OCT2003 2000	41.08	40.58	Energy only	125619		125619	7.8
6kdebris	20OCT2003 2100	41.64	41.12	Energy only	131145		131145	7.9
6kdebris	20OCT2003 2200	42.41	41.87	Energy only	137900		137900	8.0
6kdebris	20OCT2003 2300	43.19	42.62	Energy only	144648		144648	8.1
6kdebris	20OCT2003 2400	43.96	43.37	Energy only	151660		151660	8.1
6kdebris	21OCT2003 0100	44.74	44.12	Energy only	159088		159088	8.2
6kdebris	21OCT2003 0200	45.53	44.88	Energy only	166764		166764	8.4
6kdebris	21OCT2003 0300	46.45	45.81	Energy only	170959		170959	8.4
6kdebris	21OCT2003 0400	47.49	46.87	Energy only	174454		174454	8.5
6kdebris	21OCT2003 0500	48.57	47.94	Energy only	180390		180390	8.6
6kdebris	21OCT2003 0600	49.61	48.97	Press/Weir	189313	69	189382	9.0
6kdebris	21OCT2003 0700	50.30	49.50	Press/Weir	213994	285	214279	10.1
6kdebris	21OCT2003 0800	50.88	49.98	Press/Weir	230413	606	231020	10.9
6kdebris	21OCT2003 0900	51.41	50.41	Press/Weir	245194	892	246086	11.6
6kdebris	21OCT2003 1000	51.88	50.79	Press/Weir	257995	1681	259677	12.2
6kdebris	21OCT2003 1100	52.30	51.13	Press/Weir	269195	2716	271911	12.8
6kdebris	21OCT2003 1200	52.67	51.43	Press/Weir	278284	4275	282559	13.2
6kdebris	21OCT2003 1300	52.99	51.68	Press/Weir	285860	6085	291945	13.6
6kdebris	21OCT2003 1400	53.26	51.89	Press/Weir	292288	7687	299975	13.9
6kdebris	21OCT2003 1500	53.49	52.08	Press/Weir	297060	9050	306111	14.1
6kdebris	21OCT2003 1600	53.67	52.23	Press/Weir	300665	10185	310850	14.3
6kdebris	21OCT2003 1700	53.82	52.36	Press/Weir	303172	11157	314329	14.4
6kdebris	21OCT2003 1800	53.92	52.44	Press/Weir	304916	11820	316736	14.5
6kdebris	21OCT2003 1900	53.98	52.49	Press/Weir	305922	12182	318105	14.5
8kdebris	20OCT2003 0600	37.15	36.77	Energy only	94704		94704	8.3
8kdebris	20OCT2003 0700	37.29	36.90	Energy only	95605		95605	8.3
8kdebris	20OCT2003 0800	37.42	37.03	Energy only	96528		96528	8.3
8kdebris	20OCT2003 0900	37.55	37.16	Energy only	97488		97488	8.3
8kdebris	20OCT2003 1000	37.72	37.33	Energy only	98756		98756	8.4
8kdebris	20OCT2003 1100	37.94	37.54	Energy only	100490		100490	8.4
8kdebris	20OCT2003 1200	38.20	37.79	Energy only	102351		102351	8.5

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
8kdebris	20OCT2003 1300	38.57	38.15	Energy only	105315		105315	8.6
8kdebris	20OCT2003 1400	38.95	38.52	Energy only	108242		108242	8.7
8kdebris	20OCT2003 1500	39.40	38.96	Energy only	111733		111733	8.8
8kdebris	20OCT2003 1600	39.87	39.41	Energy only	115398		115398	8.9
8kdebris	20OCT2003 1700	40.29	39.83	Energy only	118382		118382	8.9
8kdebris	20OCT2003 1800	40.54	40.07	Energy only	120048		120048	8.9
8kdebris	20OCT2003 1900	40.83	40.35	Energy only	122331		122331	8.9
8kdebris	20OCT2003 2000	41.17	40.68	Energy only	125156		125156	8.9
8kdebris	20OCT2003 2100	41.78	41.27	Energy only	131136		131136	9.1
8kdebris	20OCT2003 2200	42.56	42.02	Energy only	137775		137775	9.1
8kdebris	20OCT2003 2300	43.32	42.76	Energy only	144506		144506	9.2
8kdebris	20OCT2003 2400	44.09	43.51	Energy only	151510		151510	9.2
8kdebris	21OCT2003 0100	44.87	44.26	Energy only	158922		158922	9.3
8kdebris	21OCT2003 0200	45.75	45.14	Energy only	163375		163375	9.3
8kdebris	21OCT2003 0300	46.78	46.19	Press Only	166051		166051	8.7
8kdebris	21OCT2003 0400	47.91	47.34	Press Only	168752		168752	8.8
8kdebris	21OCT2003 0500	49.13	48.59	Press/Weir	171383	13	171395	8.9
8kdebris	21OCT2003 0600	50.10	49.45	Press/Weir	192256	182	192438	10.0
8kdebris	21OCT2003 0700	50.74	49.96	Press/Weir	214311	594	214905	11.2
8kdebris	21OCT2003 0800	51.35	50.48	Press/Weir	228873	1041	229915	11.9
8kdebris	21OCT2003 0900	51.91	50.96	Press/Weir	242274	1643	243916	12.6
8kdebris	21OCT2003 1000	52.43	51.40	Press/Weir	253955	2919	256874	13.2
8kdebris	21OCT2003 1100	52.90	51.80	Press/Weir	263574	5062	268635	13.7
8kdebris	21OCT2003 1200	53.31	52.14	Press/Weir	272065	7447	279512	14.2
8kdebris	21OCT2003 1300	53.67	52.44	Press/Weir	279323	9754	289077	14.6
8kdebris	21OCT2003 1400	53.97	52.69	Press/Weir	285285	11992	297277	14.9
8kdebris	21OCT2003 1500	54.22	52.90	Press/Weir	289973	13820	303794	15.1
8kdebris	21OCT2003 1600	54.43	53.08	Press/Weir	293527	15266	308793	15.3
8kdebris	21OCT2003 1700	54.60	53.22	Press/Weir	296167	16393	312560	15.4
8kdebris	21OCT2003 1800	54.71	53.32	Press/Weir	297901	17265	315167	15.5
8kdebris	21OCT2003 1900	54.78	53.38	Press/Weir	298976	17766	316742	15.6
14kdebris	20OCT2003 0600	37.37	37.00	Energy only	94017		94017	11.2
14kdebris	20OCT2003 0700	37.51	37.14	Energy only	94924		94924	11.3
14kdebris	20OCT2003 0800	37.64	37.27	Energy only	95847		95847	11.4
14kdebris	20OCT2003 0900	37.82	37.44	Energy only	97147		97147	11.5
14kdebris	20OCT2003 1000	38.05	37.67	Energy only	98785		98785	11.6
14kdebris	20OCT2003 1100	38.27	37.88	Energy only	100258		100258	11.7
14kdebris	20OCT2003 1200	38.61	38.21	Energy only	102823		102823	11.8
14kdebris	20OCT2003 1300	38.94	38.53	Energy only	105034		105034	12.0
14kdebris	20OCT2003 1400	39.34	38.92	Energy only	108077		108077	12.2

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
14kdebris	20OCT2003 1500	39.79	39.36	Energy only	111427		111427	12.4
14kdebris	20OCT2003 1600	40.26	39.82	Energy only	114965		114965	12.6
14kdebris	20OCT2003 1700	40.55	40.11	Energy only	116613		116613	12.7
14kdebris	20OCT2003 1800	40.81	40.36	Energy only	118500		118500	12.8
14kdebris	20OCT2003 1900	41.12	40.66	Energy only	120816		120816	12.9
14kdebris	20OCT2003 2000	41.48	41.01	Energy only	123698		123698	13.0
14kdebris	20OCT2003 2100	42.28	41.79	Energy only	130851		130851	13.4
14kdebris	20OCT2003 2200	43.05	42.54	Energy only	137071		137071	13.6
14kdebris	20OCT2003 2300	43.83	43.30	Energy only	143328		143328	13.8
14kdebris	20OCT2003 2400	44.61	44.06	Energy only	150108		150108	14.1
14kdebris	21OCT2003 0100	45.49	44.94	Energy only	154053		154053	14.2
14kdebris	21OCT2003 0200	46.49	45.96	Energy only	156746		156746	14.2
14kdebris	21OCT2003 0300	47.61	47.09	Press/Weir	158662	546	159208	11.8
14kdebris	21OCT2003 0400	48.80	48.31	Press/Weir	161188	649	161837	12.0
14kdebris	21OCT2003 0500	50.05	49.58	Press/Weir	164850	806	165656	12.2
14kdebris	21OCT2003 0600	51.24	50.73	Press/Weir	175307	1354	176661	13.0
14kdebris	21OCT2003 0700	52.17	51.57	Press/Weir	193762	3561	197323	14.4
14kdebris	21OCT2003 0800	53.02	52.35	Press/Weir	205893	7399	213291	15.3
14kdebris	21OCT2003 0900	53.82	53.09	Press/Weir	215243	11700	226943	16.0
14kdebris	21OCT2003 1000	54.56	53.77	Press/Weir	223269	16619	239888	16.6
14kdebris	21OCT2003 1100	55.22	54.37	Press/Weir	231388	21155	252544	17.2
14kdebris	21OCT2003 1200	55.81	54.90	Press/Weir	238060	26452	264512	17.7
14kdebris	21OCT2003 1300	56.31	55.35	Press/Weir	243874	31487	275361	18.1
14kdebris	21OCT2003 1400	56.73	55.73	Press/Weir	248762	36076	284838	18.5
14kdebris	21OCT2003 1500	57.00	55.91	Press/Weir	255906	42964	298870	19.0
14kdebris	21OCT2003 1600	57.32	56.26	Press/Weir	255137	42209	297346	19.0
14kdebris	21OCT2003 1700	57.60	56.52	Press/Weir	257179	44318	301498	19.1
14kdebris	21OCT2003 1800	57.82	56.72	Press/Weir	258960	46130	305090	19.2
14kdebris	21OCT2003 1900	57.97	56.86	Press/Weir	260084	47637	307721	19.3
20kdebris	20OCT2003 0600	42.28	42.09	Energy only	83147		83147	29.3
20kdebris	20OCT2003 0700	42.50	42.30	Energy only	83820		83820	29.3
20kdebris	20OCT2003 0800	42.73	42.53	Energy only	84529		84529	29.3
20kdebris	20OCT2003 0900	43.21	43.01	Energy only	86047		86047	29.3
20kdebris	20OCT2003 1000	43.67	43.47	Energy only	87522		87522	29.0
20kdebris	20OCT2003 1100	44.14	43.95	Energy only	89029		89029	29.5
20kdebris	20OCT2003 1200	44.64	44.44	Energy only	90584		90584	29.7
20kdebris	20OCT2003 1300	45.17	44.97	Energy only	91756		91756	29.7
20kdebris	20OCT2003 1400	45.75	45.56	Energy only	92093		92093	29.7
20kdebris	20OCT2003 1500	46.42	46.24	Energy only	92478		92478	29.8
20kdebris	20OCT2003 1600	47.14	46.97	Energy only	92891		92891	29.8

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
20kdebris	20OCT2003 1700	47.91	47.74	Energy only	93329		93329	29.8
20kdebris	20OCT2003 1800	48.72	48.56	Press/Weir	93137	660	93798	12.5
20kdebris	20OCT2003 1900	49.60	49.45	Press/Weir	93606	695	94301	12.5
20kdebris	20OCT2003 2000	50.54	50.38	Press/Weir	94814	763	95577	12.7
20kdebris	20OCT2003 2100	51.37	51.19	Press/Weir	105233	2087	107319	14.1
20kdebris	20OCT2003 2200	52.15	51.94	Press/Weir	112456	4857	117313	15.1
20kdebris	20OCT2003 2300	52.89	52.66	Press/Weir	118278	9034	127312	15.8
20kdebris	20OCT2003 2400	53.60	53.34	Press/Weir	123246	13785	137031	16.5
20kdebris	21OCT2003 0100	54.29	54.00	Press/Weir	127581	19065	146646	17.1
20kdebris	21OCT2003 0200	54.97	54.65	Press/Weir	131771	24777	156548	17.6
20kdebris	21OCT2003 0300	55.65	55.30	Press/Weir	135482	31551	167033	18.1
20kdebris	21OCT2003 0400	56.33	55.95	Press/Weir	139114	38344	177458	18.6
20kdebris	21OCT2003 0500	57.01	56.59	Press/Weir	142225	45600	187825	19.0
20kdebris	21OCT2003 0600	57.68	57.23	Press/Weir	144899	53244	198143	19.4
20kdebris	21OCT2003 0700	58.33	57.85	Press/Weir	147390	61304	208693	19.7
20kdebris	21OCT2003 0800	58.98	58.46	Press/Weir	149864	69283	219147	20.1
20kdebris	21OCT2003 0900	59.60	59.05	Press/Weir	152335	77264	229599	20.4
20kdebris	21OCT2003 1000	60.14	59.55	Press/Weir	154286	84513	238799	20.6
20kdebris	21OCT2003 1100	60.56	59.95	Press/Weir	168715	77636	246351	22.6
20kdebris	21OCT2003 1200	60.95	60.31	Press/Weir	170538	82654	253193	22.8
20kdebris	21OCT2003 1300	61.36	60.70	Press/Weir	172458	88103	260561	23.1
20kdebris	21OCT2003 1400	61.78	61.09	Press/Weir	174311	93716	268027	23.3
20kdebris	21OCT2003 1500	62.16	61.45	Press/Weir	176106	98787	274893	23.6
20kdebris	21OCT2003 1600	62.53	61.79	Press/Weir	177836	104140	281976	23.8
20kdebris	21OCT2003 1700	62.85	62.09	Press/Weir	179404	108966	288370	24.0
20kdebris	21OCT2003 1800	63.11	62.33	Press/Weir	180699	112904	293603	24.2
20kdebris	21OCT2003 1900	63.31	62.52	Press/Weir	181650	115989	297639	24.3
Cont 0.3 0.5	20OCT2003 0600	37.49	37.13	Energy only	93660		93660	8.6
Cont 0.3 0.5	20OCT2003 0700	37.63	37.27	Energy only	94565		94565	8.7
Cont 0.3 0.5	20OCT2003 0800	37.80	37.43	Energy only	95763		95763	8.7
Cont 0.3 0.5	20OCT2003 0900	38.02	37.64	Energy only	97350		97350	8.8
Cont 0.3 0.5	20OCT2003 1000	38.22	37.84	Energy only	98647		98647	8.8
Cont 0.3 0.5	20OCT2003 1100	38.48	38.10	Energy only	100626		100626	8.9
Cont 0.3 0.5	20OCT2003 1200	38.80	38.41	Energy only	102772		102772	9.0
Cont 0.3 0.5	20OCT2003 1300	39.11	38.71	Energy only	104986		104986	9.1
Cont 0.3 0.5	20OCT2003 1400	39.53	39.12	Energy only	108084		108084	9.3
Cont 0.3 0.5	20OCT2003 1500	39.98	39.55	Energy only	111374		111374	9.4
Cont 0.3 0.5	20OCT2003 1600	40.42	39.99	Energy only	114575		114575	9.4
Cont 0.3 0.5	20OCT2003 1700	40.66	40.23	Energy only	115948		115948	9.4
Cont 0.3 0.5	20OCT2003 1800	40.93	40.49	Energy only	117930		117930	9.5

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
Cont 0.3 0.5	20OCT2003 1900	41.24	40.79	Energy only	120294		120294	9.5
Cont 0.3 0.5	20OCT2003 2000	41.69	41.23	Energy only	124365		124365	9.7
Cont 0.3 0.5	20OCT2003 2100	42.47	41.98	Energy only	130820		130820	9.8
Cont 0.3 0.5	20OCT2003 2200	43.22	42.72	Energy only	136886		136886	9.9
Cont 0.3 0.5	20OCT2003 2300	43.98	43.46	Energy only	143087		143087	10.0
Cont 0.3 0.5	20OCT2003 2400	44.78	44.24	Energy only	148815		148815	10.1
Cont 0.3 0.5	21OCT2003 0100	45.69	45.16	Energy only	152233		152233	10.1
Cont 0.3 0.5	21OCT2003 0200	46.71	46.19	Energy only	155165		155165	10.2
Cont 0.3 0.5	21OCT2003 0300	47.82	47.32	Press/Weir	157847	25	157872	9.1
Cont 0.3 0.5	21OCT2003 0400	48.99	48.50	Press/Weir	161800	40	161840	9.3
Cont 0.3 0.5	21OCT2003 0500	50.04	49.50	Press/Weir	176265	161	176426	10.2
Cont 0.3 0.5	21OCT2003 0600	50.74	50.07	Press/Weir	199748	636	200384	11.5
Cont 0.3 0.5	21OCT2003 0700	51.38	50.62	Press/Weir	214722	1196	215918	12.4
Cont 0.3 0.5	21OCT2003 0800	52.01	51.18	Press/Weir	227029	1892	228920	13.1
Cont 0.3 0.5	21OCT2003 0900	52.60	51.70	Press/Weir	238881	3046	241927	13.8
Cont 0.3 0.5	21OCT2003 1000	53.13	52.17	Press/Weir	248022	6241	254263	14.3
Cont 0.3 0.5	21OCT2003 1100	53.63	52.60	Press/Weir	256381	9106	265487	14.8
Cont 0.3 0.5	21OCT2003 1200	54.08	52.99	Press/Weir	263841	12274	276115	15.2
Cont 0.3 0.5	21OCT2003 1300	54.46	53.32	Press/Weir	270525	15196	285721	15.6
Cont 0.3 0.5	21OCT2003 1400	54.80	53.60	Press/Weir	276064	18082	294146	15.9
Cont 0.3 0.5	21OCT2003 1500	55.08	53.85	Press/Weir	280270	20209	300479	16.2
Cont 0.3 0.5	21OCT2003 1600	55.31	54.05	Press/Weir	283602	22067	305669	16.3
Cont 0.3 0.5	21OCT2003 1700	55.50	54.21	Press/Weir	286287	23457	309744	16.5
Cont 0.3 0.5	21OCT2003 1800	55.63	54.33	Press/Weir	288154	24561	312714	16.6
Cont 0.3 0.5	21OCT2003 1900	55.71	54.40	Press/Weir	289337	25281	314618	16.7
Cont 0.5 0.7	20OCT2003 0600	37.81	37.47	Energy only	92961		92961	8.5
Cont 0.5 0.7	20OCT2003 0700	38.04	37.69	Energy only	94520		94520	8.6
Cont 0.5 0.7	20OCT2003 0800	38.24	37.88	Energy only	95725		95725	8.6
Cont 0.5 0.7	20OCT2003 0900	38.42	38.06	Energy only	96860		96860	8.7
Cont 0.5 0.7	20OCT2003 1000	38.69	38.33	Energy only	98788		98788	8.7
Cont 0.5 0.7	20OCT2003 1100	38.96	38.59	Energy only	100540		100540	8.8
Cont 0.5 0.7	20OCT2003 1200	39.24	38.87	Energy only	102400		102400	8.9
Cont 0.5 0.7	20OCT2003 1300	39.60	39.22	Energy only	104956		104956	9.0
Cont 0.5 0.7	20OCT2003 1400	40.02	39.63	Energy only	107929		107929	9.1
Cont 0.5 0.7	20OCT2003 1500	40.46	40.06	Energy only	111075		111075	9.2
Cont 0.5 0.7	20OCT2003 1600	40.73	40.32	Energy only	112529		112529	9.2
Cont 0.5 0.7	20OCT2003 1700	40.97	40.56	Energy only	114256		114256	9.2
Cont 0.5 0.7	20OCT2003 1800	41.25	40.84	Energy only	116373		116373	9.3
Cont 0.5 0.7	20OCT2003 1900	41.59	41.16	Energy only	118865		118865	9.3
Cont 0.5 0.7	20OCT2003 2000	42.28	41.83	Energy only	124730		124730	9.5

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
Cont 0.5 0.7	20OCT2003 2100	43.01	42.55	Energy only	130480		130480	9.6
Cont 0.5 0.7	20OCT2003 2200	43.75	43.27	Energy only	136225		136225	9.7
Cont 0.5 0.7	20OCT2003 2300	44.51	44.01	Energy only	142184		142184	9.8
Cont 0.5 0.7	20OCT2003 2400	45.34	44.84	Energy only	145983		145983	9.8
Cont 0.5 0.7	21OCT2003 0100	46.29	45.80	Energy only	148919		148919	9.9
Cont 0.5 0.7	21OCT2003 0200	47.32	46.85	Press only	151677		151677	8.7
Cont 0.5 0.7	21OCT2003 0300	48.45	48.00	Press only	154298		154298	8.9
Cont 0.5 0.7	21OCT2003 0400	49.65	49.21	Press/Weir	157178	24	157202	9.1
Cont 0.5 0.7	21OCT2003 0500	50.36	49.76	Press/Weir	187150	343	187494	10.8
Cont 0.5 0.7	21OCT2003 0600	50.95	50.25	Press/Weir	205068	799	205867	11.8
Cont 0.5 0.7	21OCT2003 0700	51.56	50.78	Press/Weir	217469	1331	218800	12.5
Cont 0.5 0.7	21OCT2003 0800	52.14	51.30	Press/Weir	229245	2049	231293	13.2
Cont 0.5 0.7	21OCT2003 0900	52.69	51.79	Press/Weir	239412	4027	243439	13.8
Cont 0.5 0.7	21OCT2003 1000	53.21	52.24	Press/Weir	248497	6715	255212	14.3
Cont 0.5 0.7	21OCT2003 1100	53.68	52.65	Press/Weir	256546	9705	266251	14.8
Cont 0.5 0.7	21OCT2003 1200	54.11	53.02	Press/Weir	263917	12716	276633	15.2
Cont 0.5 0.7	21OCT2003 1300	54.48	53.34	Press/Weir	270381	15638	286018	15.6
Cont 0.5 0.7	21OCT2003 1400	54.81	53.62	Press/Weir	275384	18273	293657	15.9
Cont 0.5 0.7	21OCT2003 1500	55.10	53.87	Press/Weir	279573	20392	299965	16.1
Cont 0.5 0.7	21OCT2003 1600	55.33	54.07	Press/Weir	282922	22252	305174	16.3
Cont 0.5 0.7	21OCT2003 1700	55.51	54.23	Press/Weir	285613	23673	309286	16.5
Cont 0.5 0.7	21OCT2003 1800	55.65	54.35	Press/Weir	287523	24800	312323	16.6
Cont 0.5 0.7	21OCT2003 1900	55.74	54.43	Press/Weir	288760	25552	314311	16.6
BankSta	20OCT2003 0600	37.17	36.86	Energy only		94562	94562	8.7
BankSta	20OCT2003 0700	37.30	36.99	Energy only		95465	95465	8.7
BankSta	20OCT2003 0800	37.43	37.12	Energy only		96384	96384	8.8
BankSta	20OCT2003 0900	37.56	37.25	Energy only		97331	97331	8.8
BankSta	20OCT2003 1000	37.74	37.43	Energy only		98703	98703	8.9
BankSta	20OCT2003 1100	37.98	37.66	Energy only		100537	100537	9.0
BankSta	20OCT2003 1200	38.23	37.91	Energy only		102390	102390	9.0
BankSta	20OCT2003 1300	38.61	38.28	Energy only		105386	105386	9.2
BankSta	20OCT2003 1400	38.99	38.65	Energy only		108334	108334	9.3
BankSta	20OCT2003 1500	39.43	39.09	Energy only		111809	111809	9.4
BankSta	20OCT2003 1600	39.90	39.55	Energy only		115465	115465	9.5
BankSta	20OCT2003 1700	40.28	39.93	Energy only		118084	118084	9.6
BankSta	20OCT2003 1800	40.53	40.17	Energy only		119836	119836	9.6
BankSta	20OCT2003 1900	40.82	40.45	Energy only		122146	122146	9.6
BankSta	20OCT2003 2000	41.16	40.79	Energy only		124993	124993	9.7
BankSta	20OCT2003 2100	41.80	41.41	Energy only		131203	131203	9.9
BankSta	20OCT2003 2200	42.57	42.16	Energy only		137752	137752	10.0

Plan	Profile	E.G. US.	W.S. US.	Br Sel Method	Q Bridge	Q Weir	Q Total	BR Open Vel
BankSta	20OCT2003 2300	43.33	42.91	Energy only		144413	144413	10.1
BankSta	20OCT2003 2400	44.10	43.67	Energy only		151399	151399	10.2
BankSta	21OCT2003 0100	44.87	44.42	Energy only		158825	158825	10.4
BankSta	21OCT2003 0200	45.82	45.38	Energy only		161431	161431	10.4
BankSta	21OCT2003 0300	46.89	46.47	Press/Weir	163901	21	163923	9.4
BankSta	21OCT2003 0400	48.04	47.64	Press/Weir	166906	36	166942	9.6
BankSta	21OCT2003 0500	49.26	48.88	Press/Weir	170278	55	170333	9.8
BankSta	21OCT2003 0600	50.26	49.82	Press/Weir	189020	282	189302	10.9
BankSta	21OCT2003 0700	50.98	50.46	Press/Weir	209060	767	209828	12.0
BankSta	21OCT2003 0800	51.69	51.11	Press/Weir	222491	1307	223798	12.8
BankSta	21OCT2003 0900	52.35	51.73	Press/Weir	234274	3075	237349	13.5
BankSta	21OCT2003 1000	52.96	52.29	Press/Weir	244388	5872	250260	14.1
BankSta	21OCT2003 1100	53.51	52.79	Press/Weir	253277	8926	262203	14.6
BankSta	21OCT2003 1200	54.00	53.24	Press/Weir	261018	12237	273255	15.0
BankSta	21OCT2003 1300	54.42	53.62	Press/Weir	267875	15386	283261	15.4
BankSta	21OCT2003 1400	54.79	53.97	Press/Weir	273207	18249	291456	15.7
BankSta	21OCT2003 1500	55.11	54.26	Press/Weir	277757	20633	298390	16.0
BankSta	21OCT2003 1600	55.37	54.49	Press/Weir	281417	22606	304023	16.2
BankSta	21OCT2003 1700	55.56	54.67	Press/Weir	284170	24248	308418	16.4
BankSta	21OCT2003 1800	55.71	54.80	Press/Weir	286169	25455	311624	16.5
BankSta	21OCT2003 1900	55.80	54.89	Press/Weir	287410	26305	313715	16.6

Table A-2: Main Channel Scour Depth and Area at 150,000 cfs

Condition	Depth of Scour (ft)	Estimated Scoured Area (sq. ft.)
No Debris	2.5	1,500
10,000 SF Debris	11.4	7,000

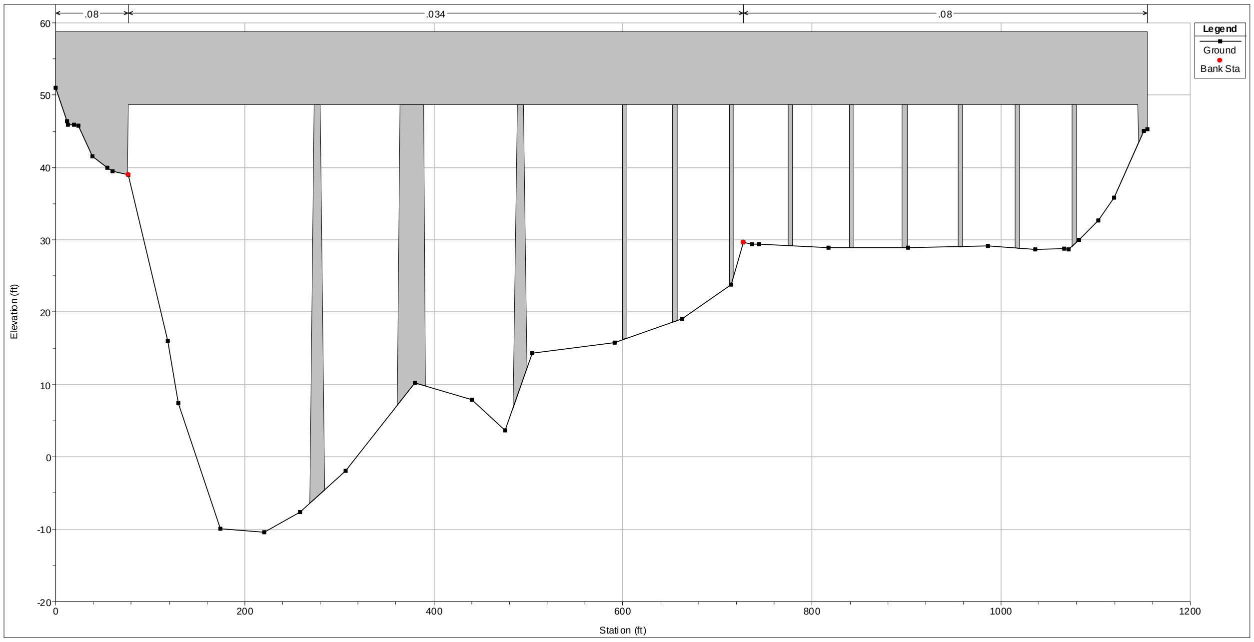


Figure A-1: Previously assumed BNSF bridge geometry.

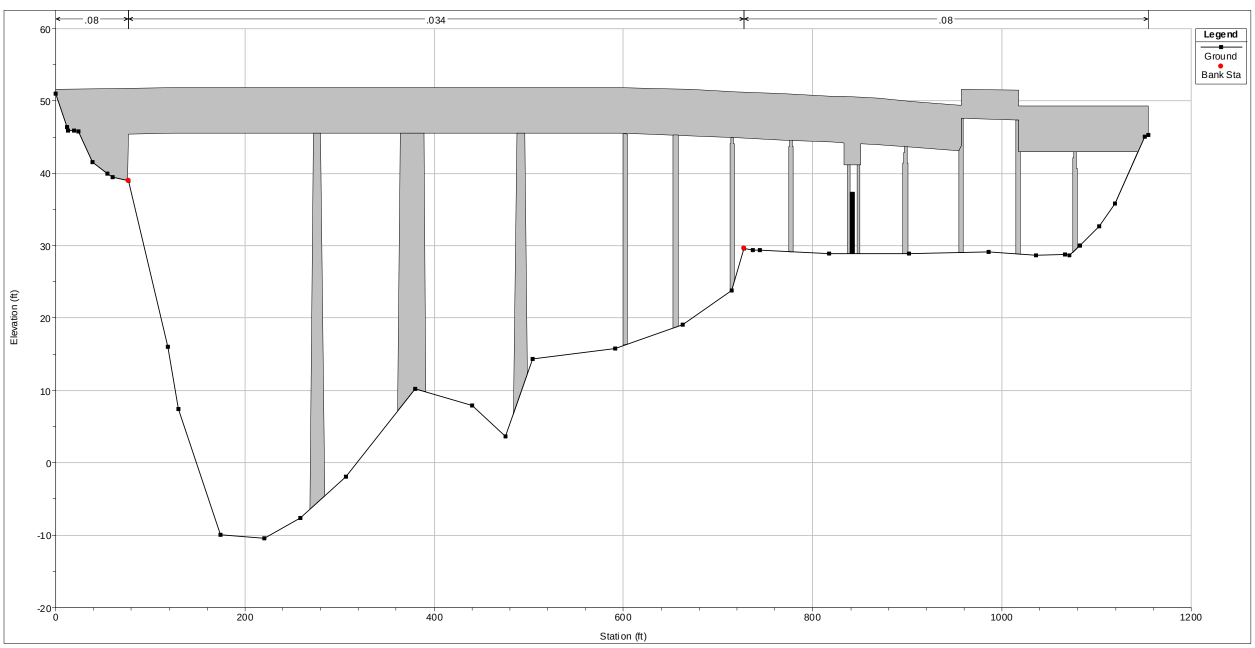


Figure A-2: BNSF bridge geometry from November 2012 survey.

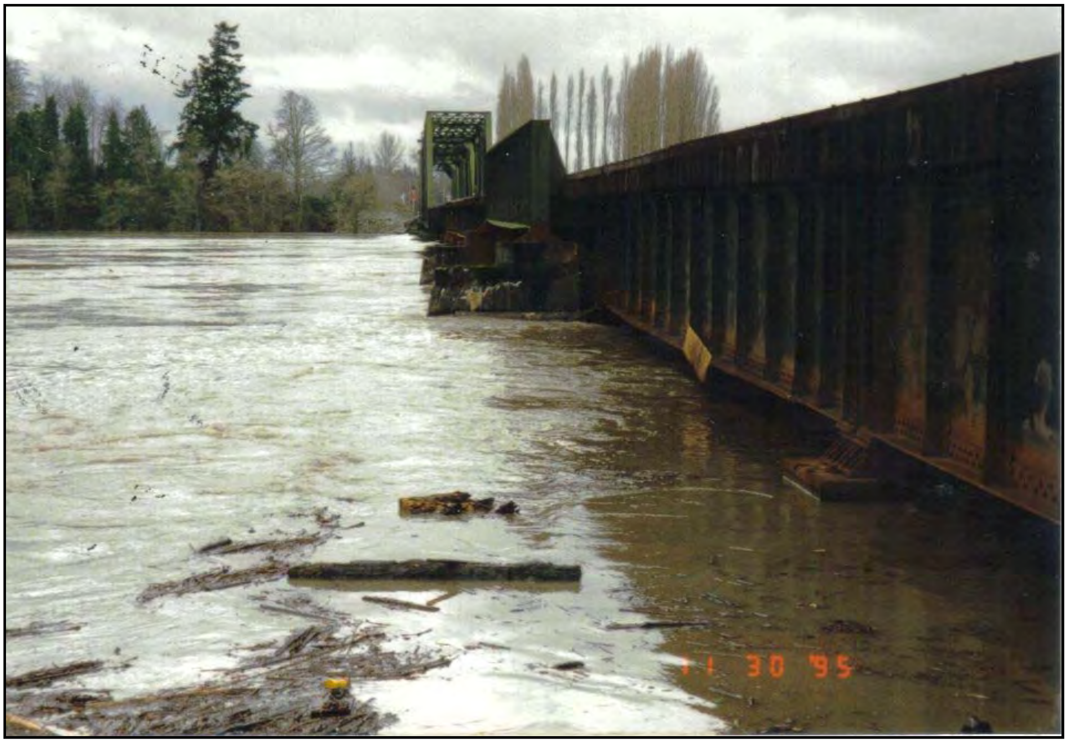


Figure A-3: BNSF Bridge during November 1995 flood (photo by Chuck Bennett, Dike District 12, courtesy of www.skagitriverhistory.com)

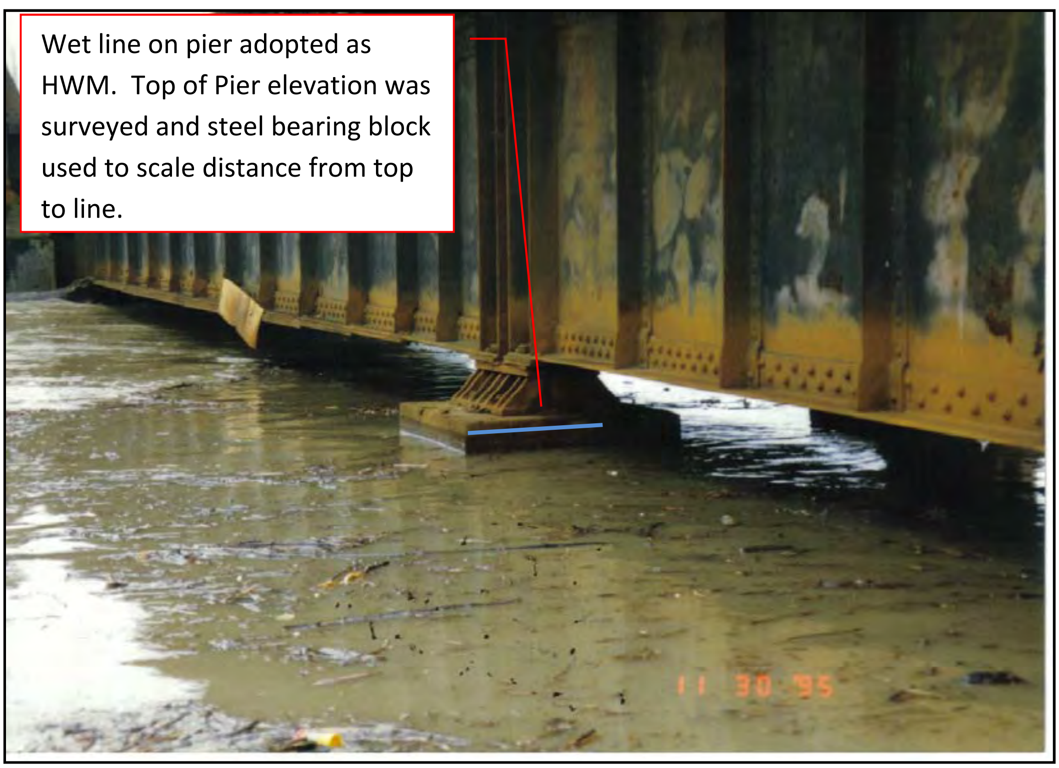


Figure A-4: Close up of BNSF Bridge Pier during November 1995 flood (photo by Chuck Bennett, Dike District 12, courtesy of www.skagitriverhistory.com)

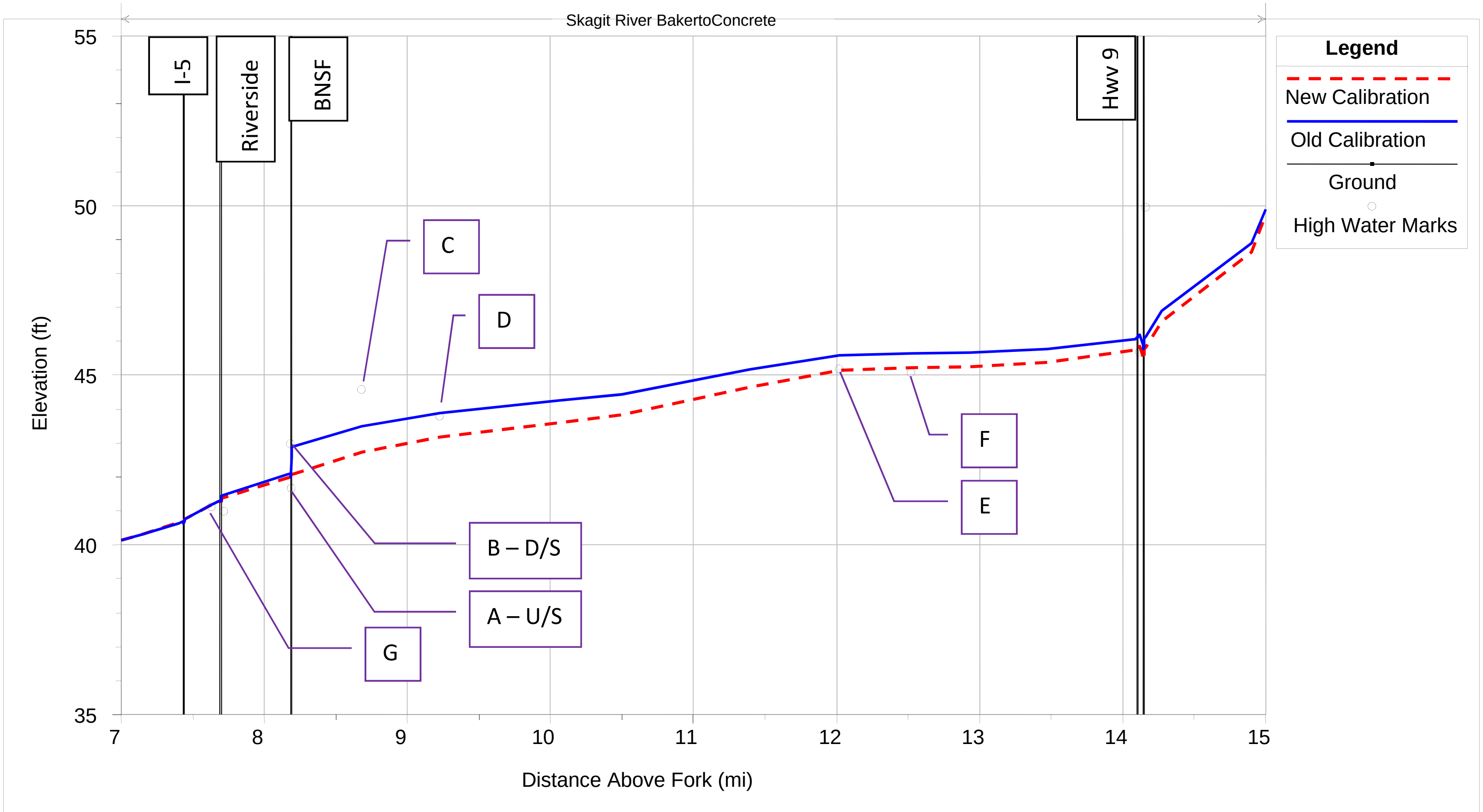


Figure A-5: 1995 Event Simulated Water Surface Profiles and High Water Marks.

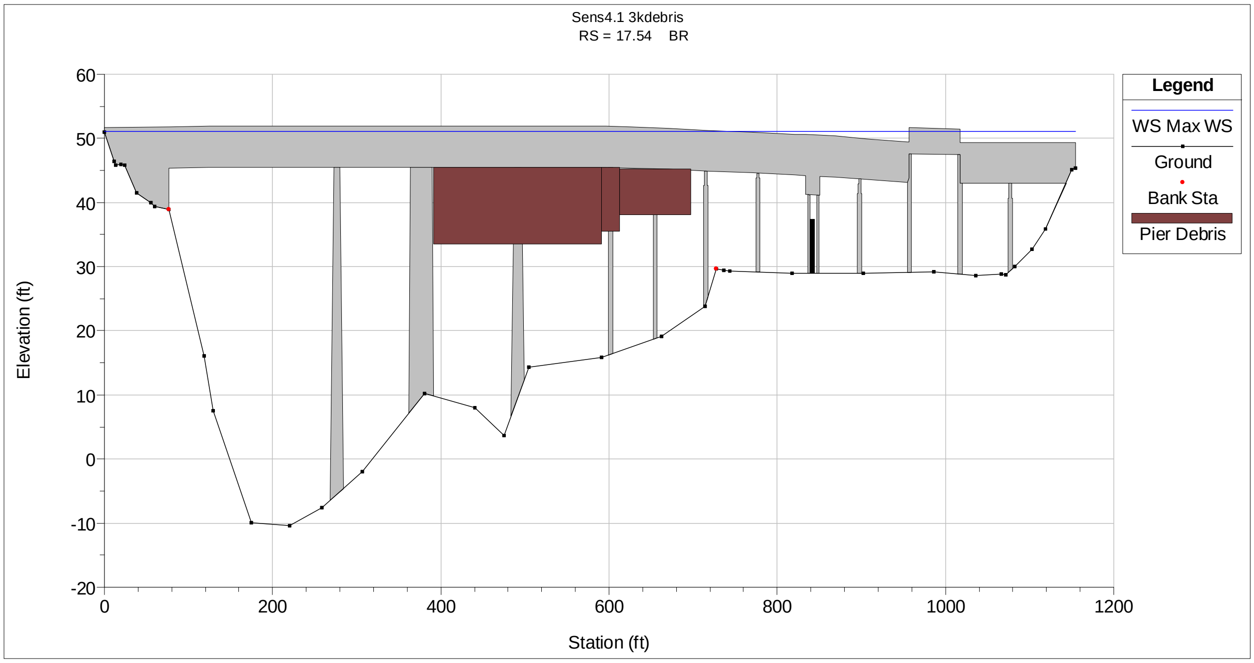


Figure A-6: BNSF Bridge – 3,000 sq. ft. of debris.

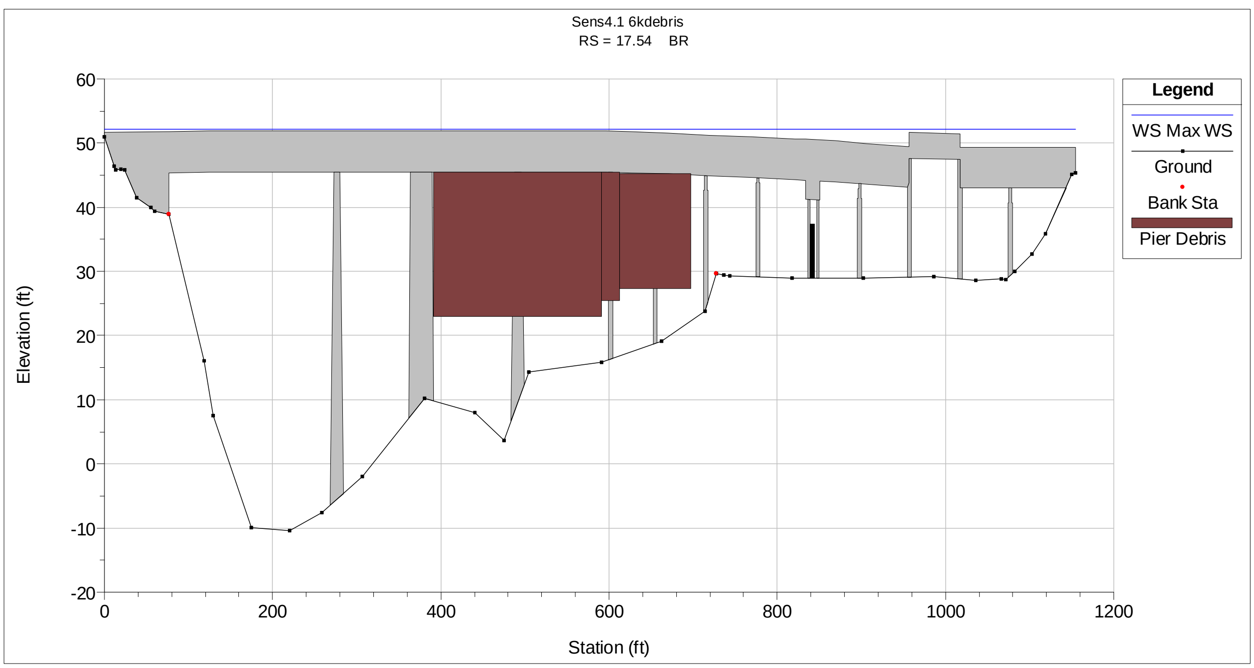


Figure A-7: BNSF Bridge – 6,000 sq. ft. of debris.

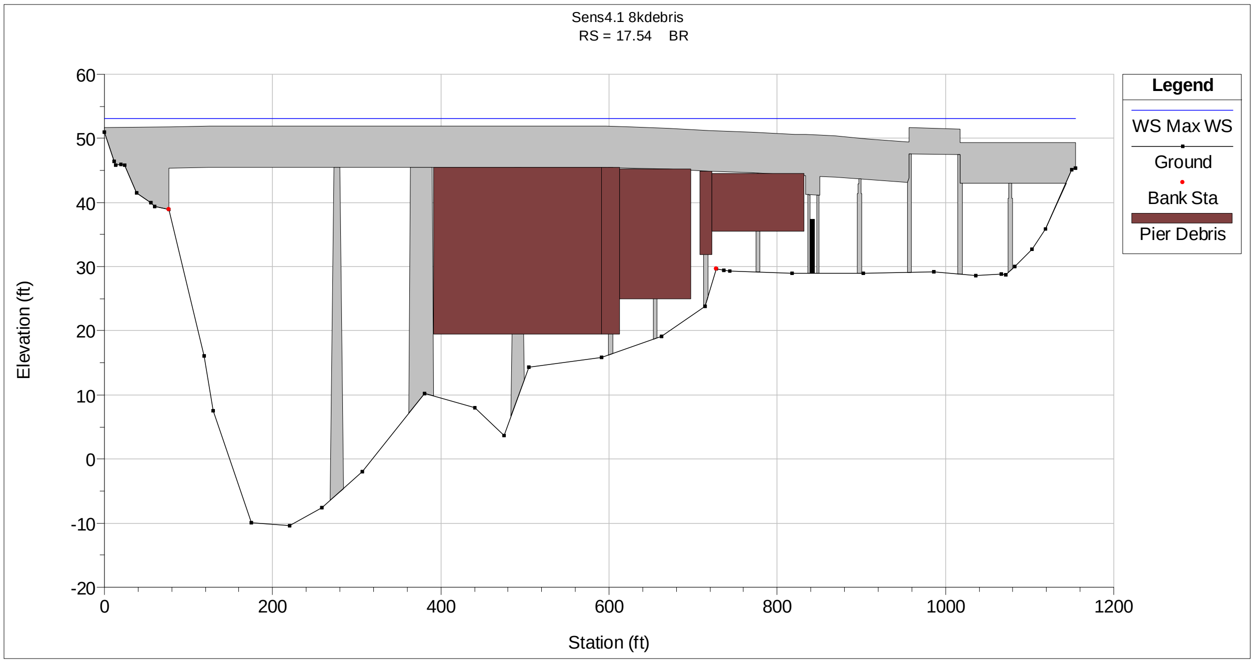


Figure A-8: BNSF Bridge – 8,000 sq. ft. of debris.

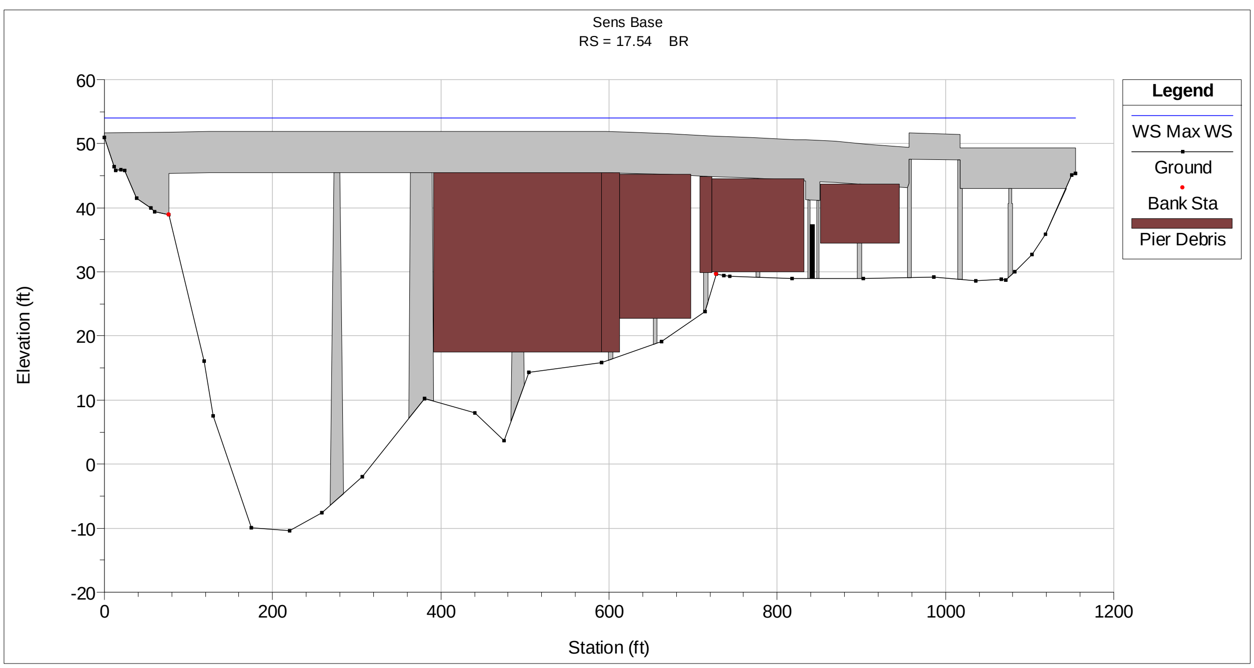


Figure A-9: BNSF Bridge – 10,000 sq. ft. of debris (base scenario)

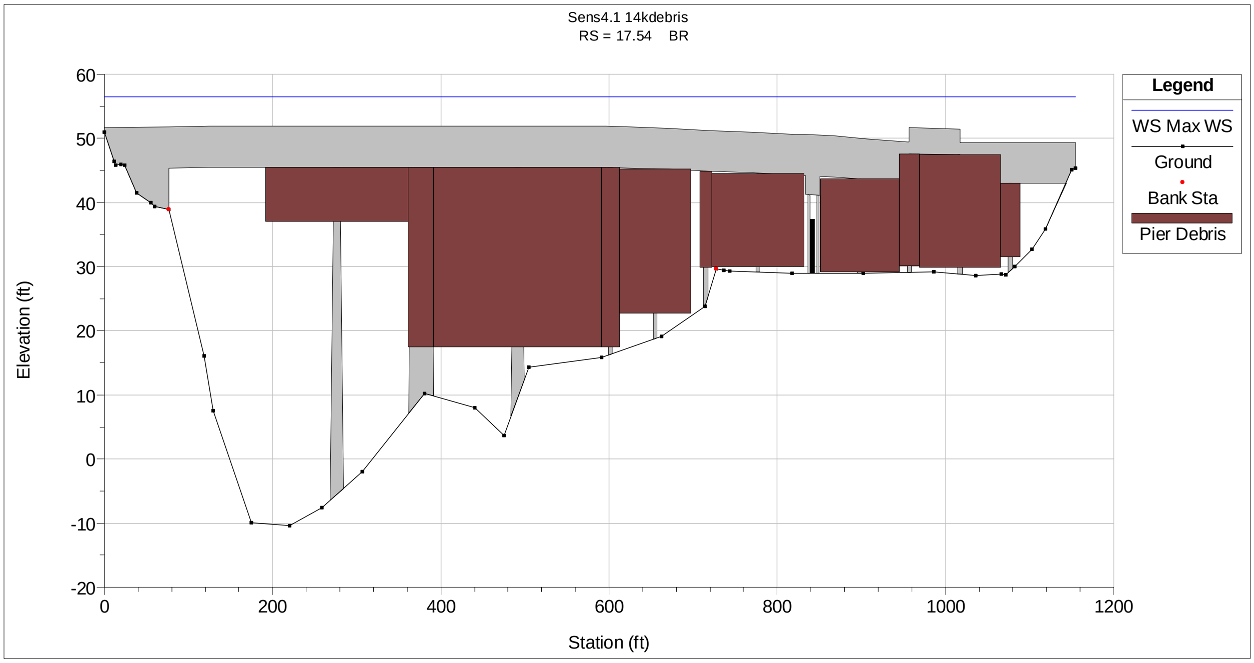


Figure A-10: BNSF Bridge – 14,000 sq. ft. of debris.

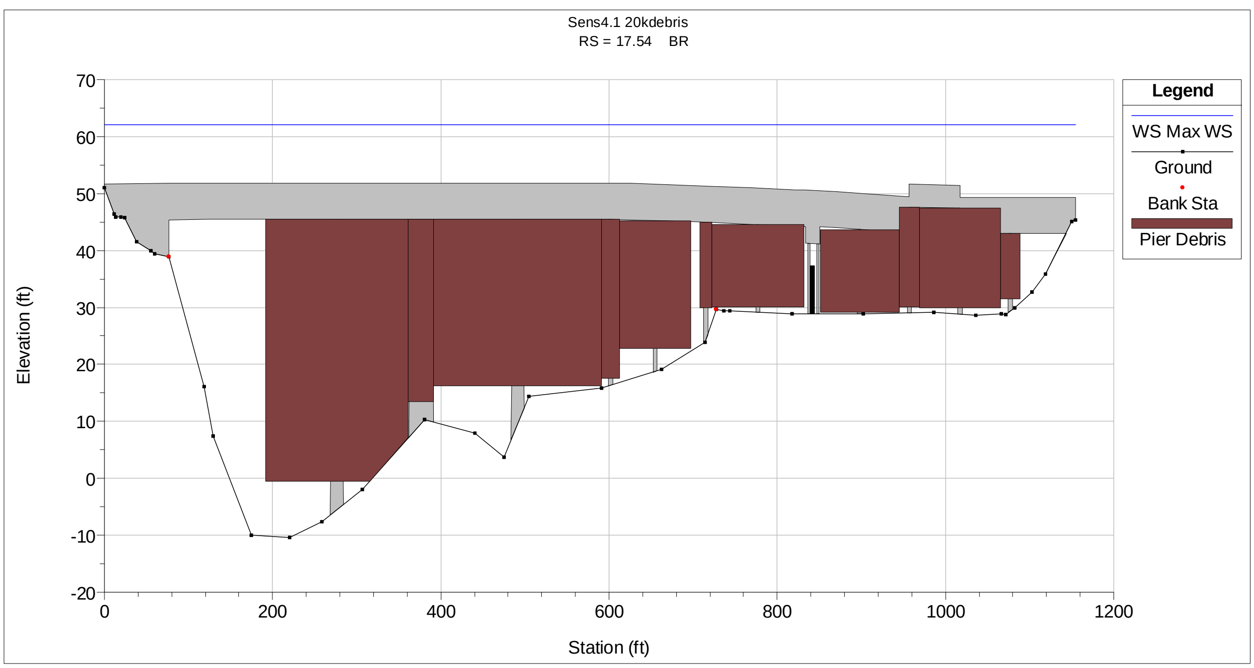
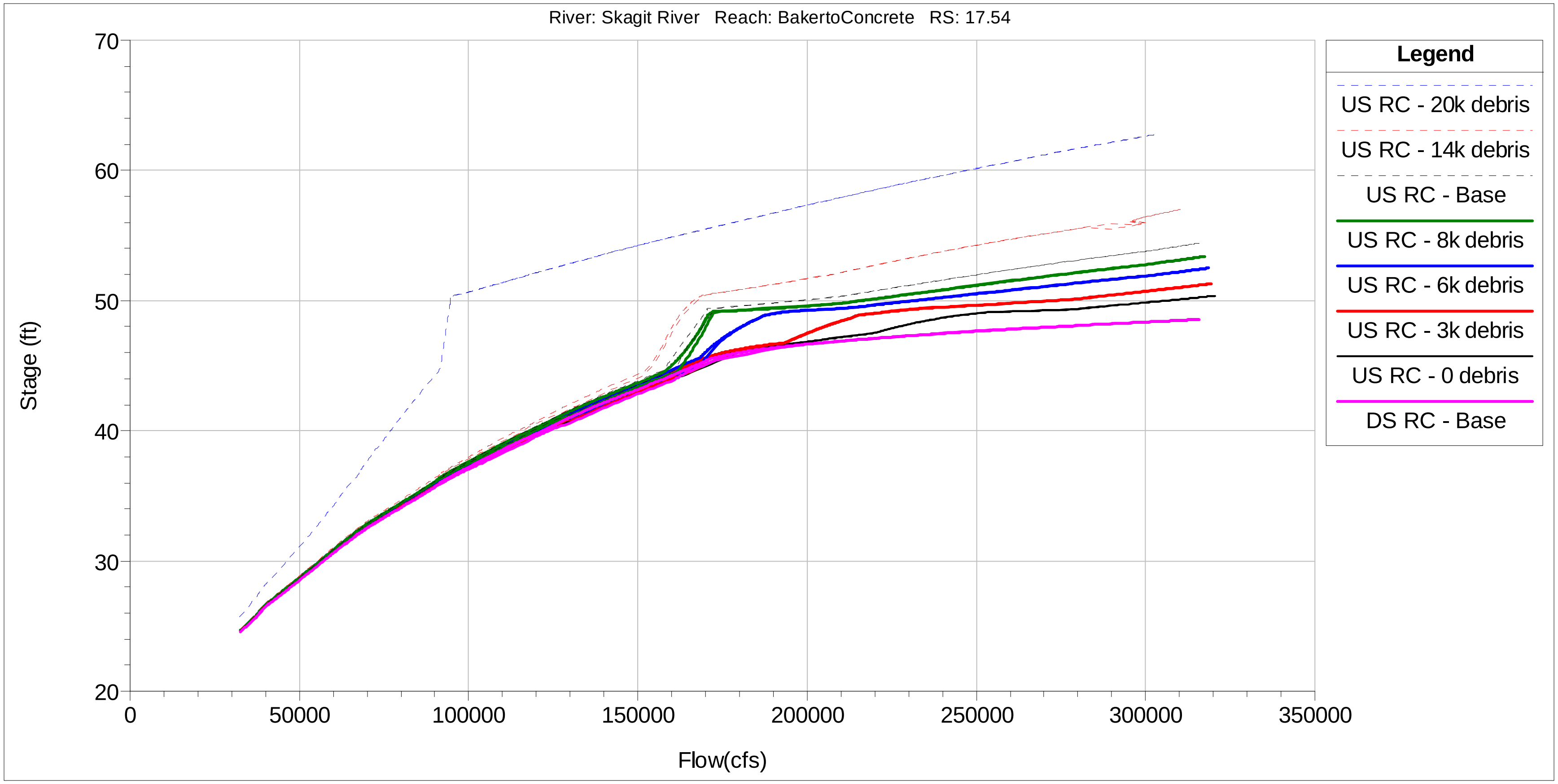
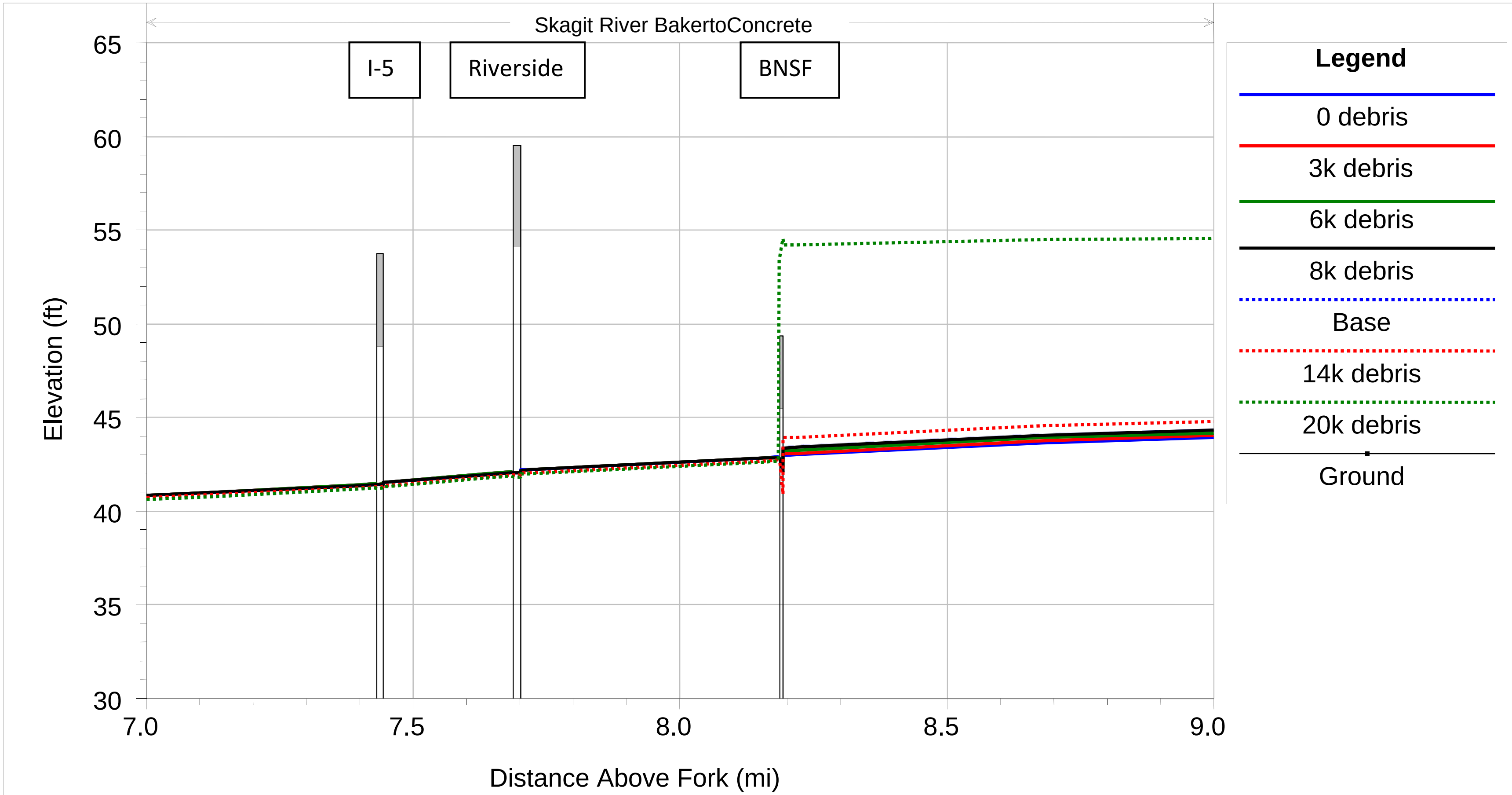


Figure A-11: BNSF Bridge – 20,000 sq. ft. debris.



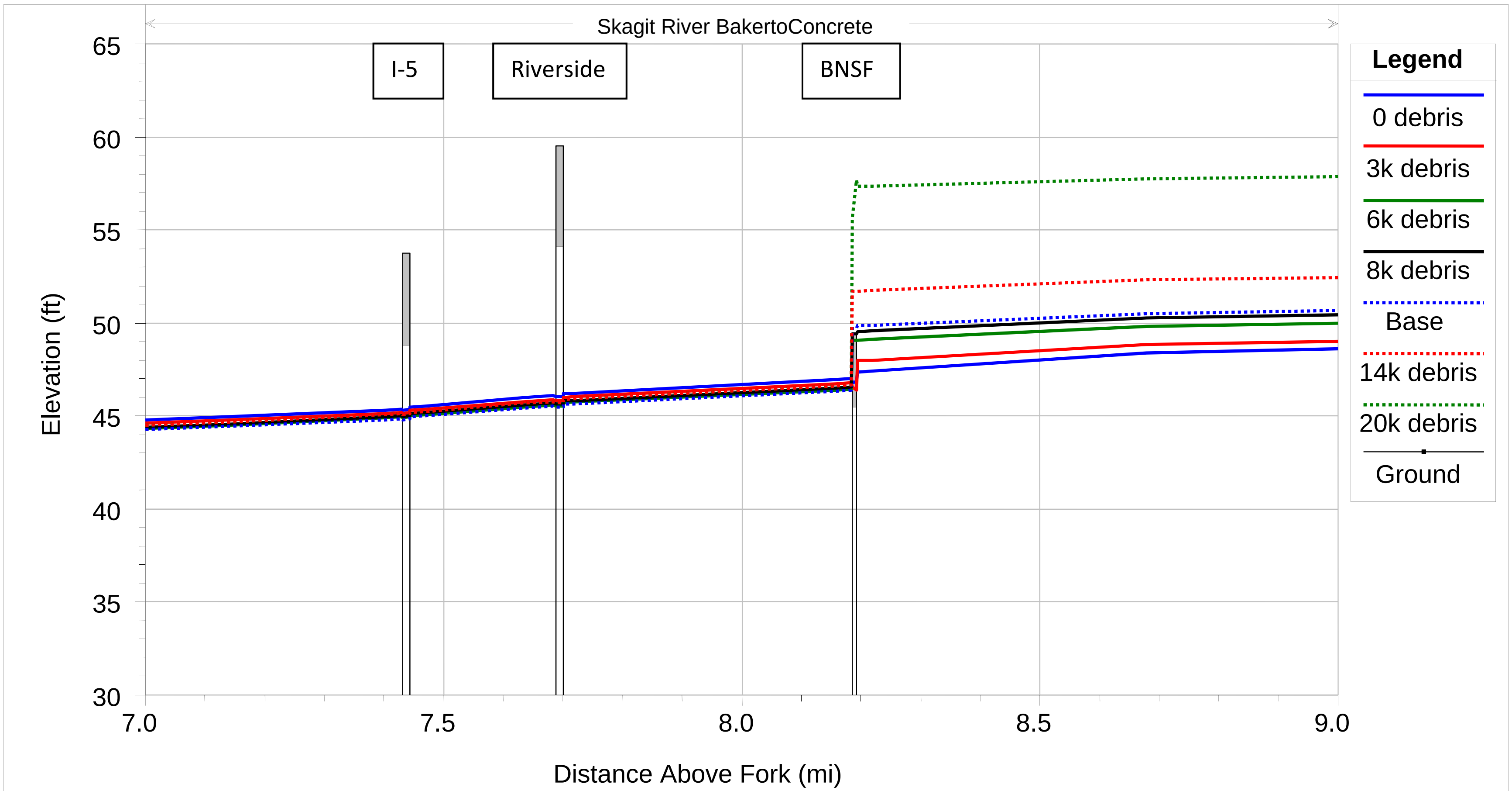
Notes: Base scenario has 10,000 sq ft of debris. Simulations assume no scour.

Figure A-12: Rating Curves – Debris Sensitivity.



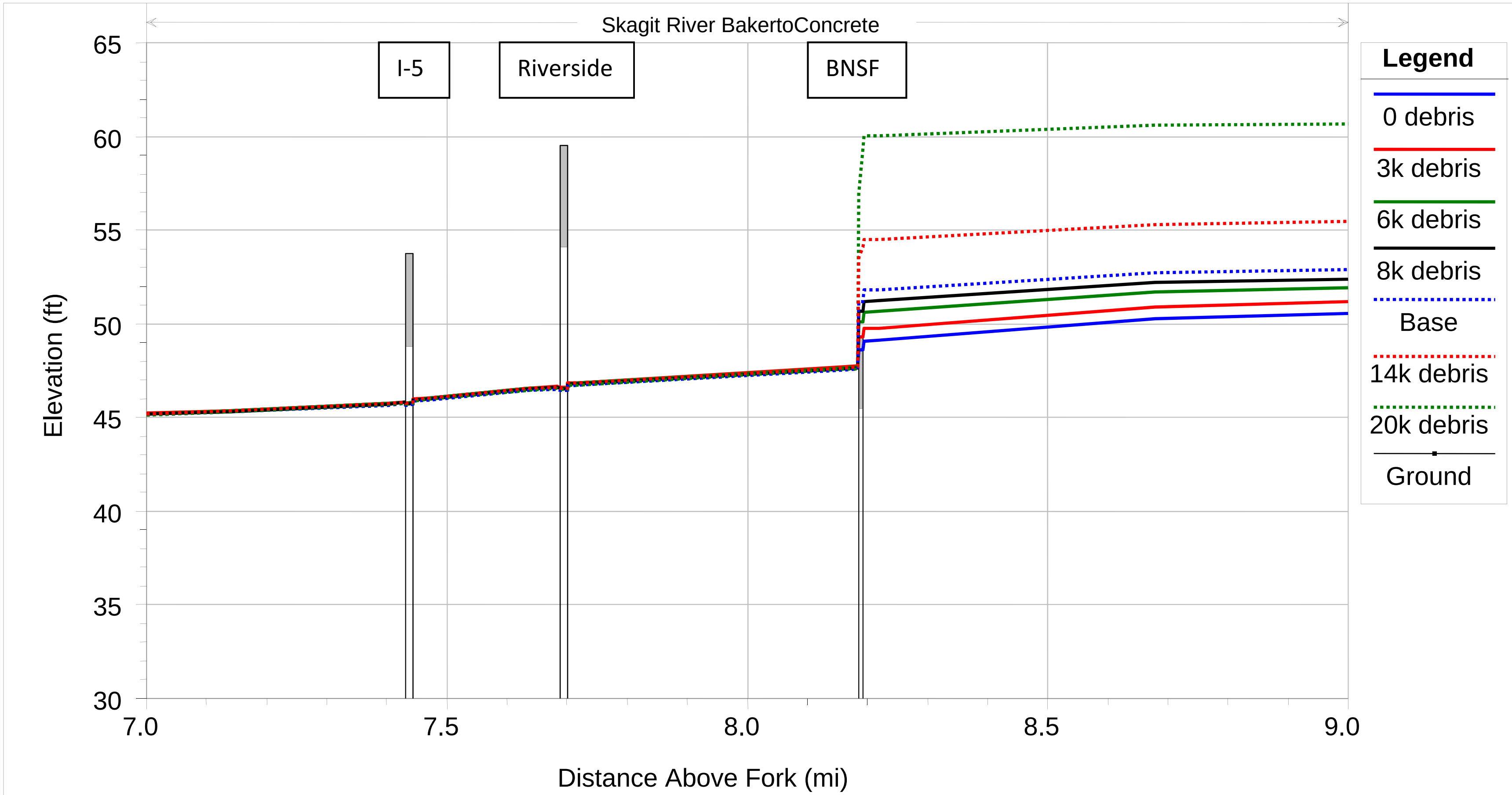
Notes: Base scenario has 10,000 sq ft of debris. Simulations assume no scour.

Figure A-13: 150,000 cfs Water Surface Profiles – Debris Sensitivity.



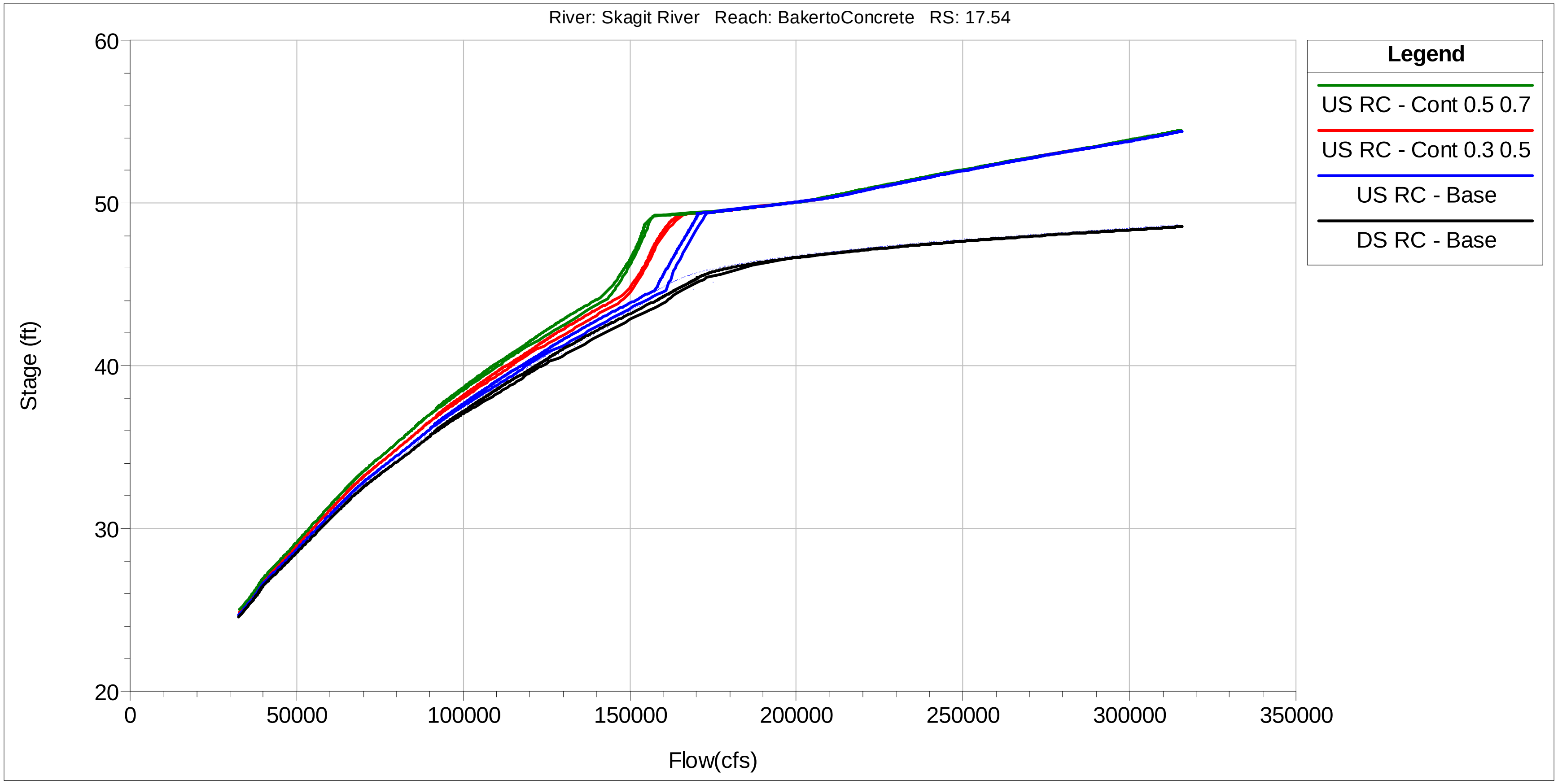
Notes: Base scenario has 10,000 sq ft of debris. Simulations assume no scour.

Figure A-14: 200,000 cfs Water Surface Profiles – Debris Sensitivity.



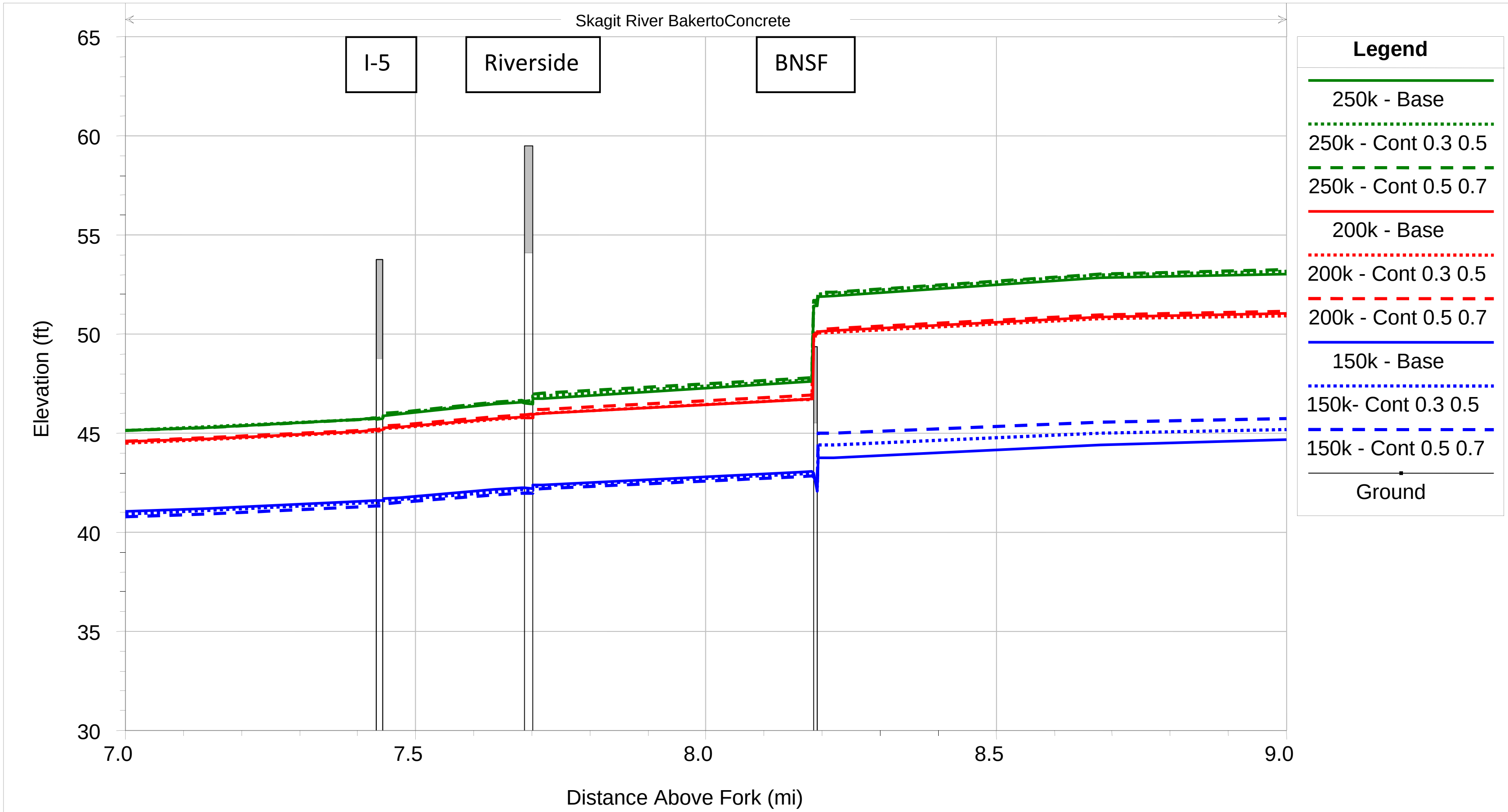
Notes: Base scenario has 10,000 sq ft of debris. Simulations assume no scour.

Figure A-15: 250,000 cfs Water Surface Profiles – Debris Sensitivity.



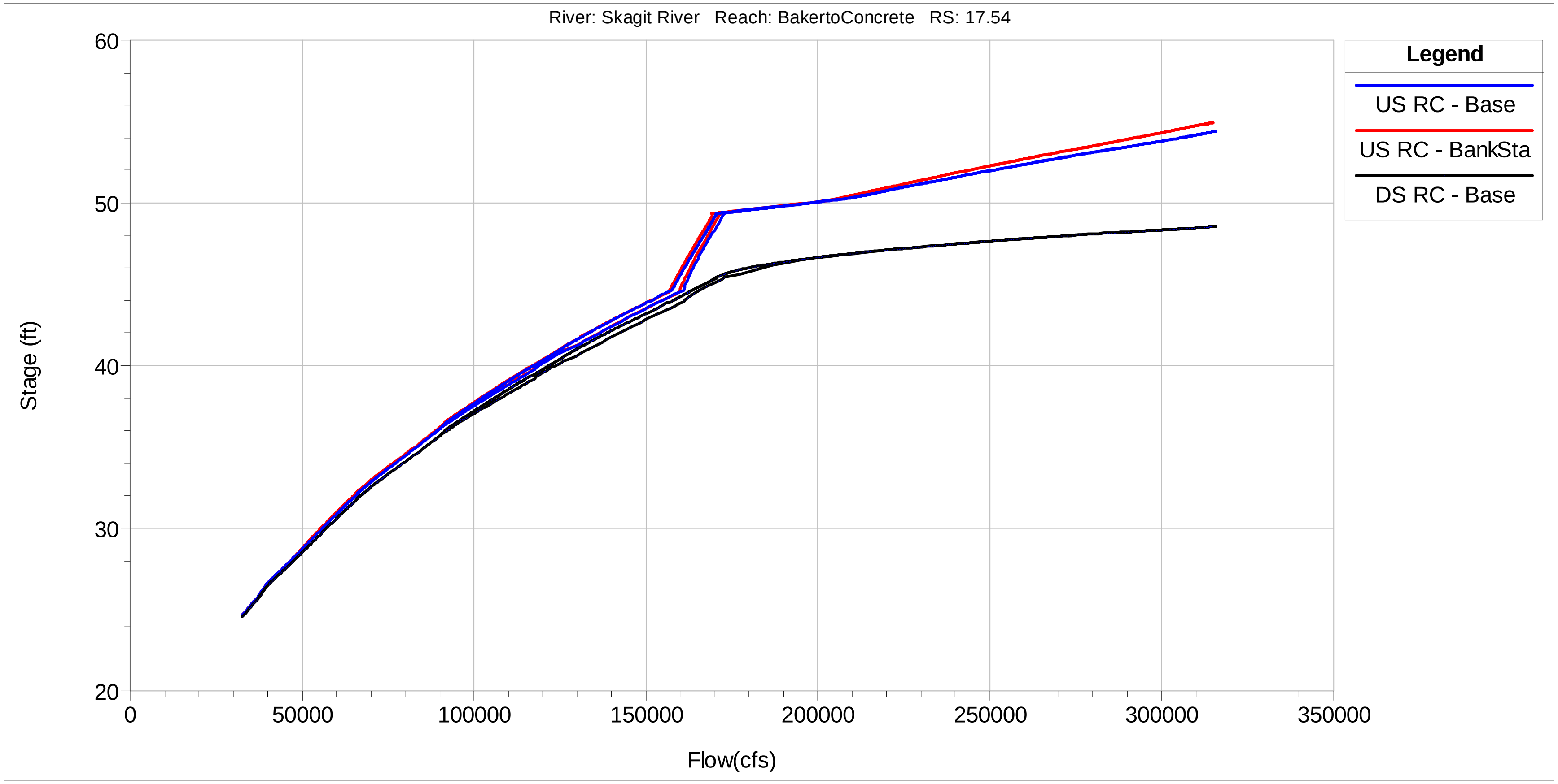
Notes: Base scenario with 0.1/0.3 contraction/expansion coefficients. Simulations assume no scour.

Figure A-16: Rating Curve – Contraction/Expansion Coefficient Sensitivity.



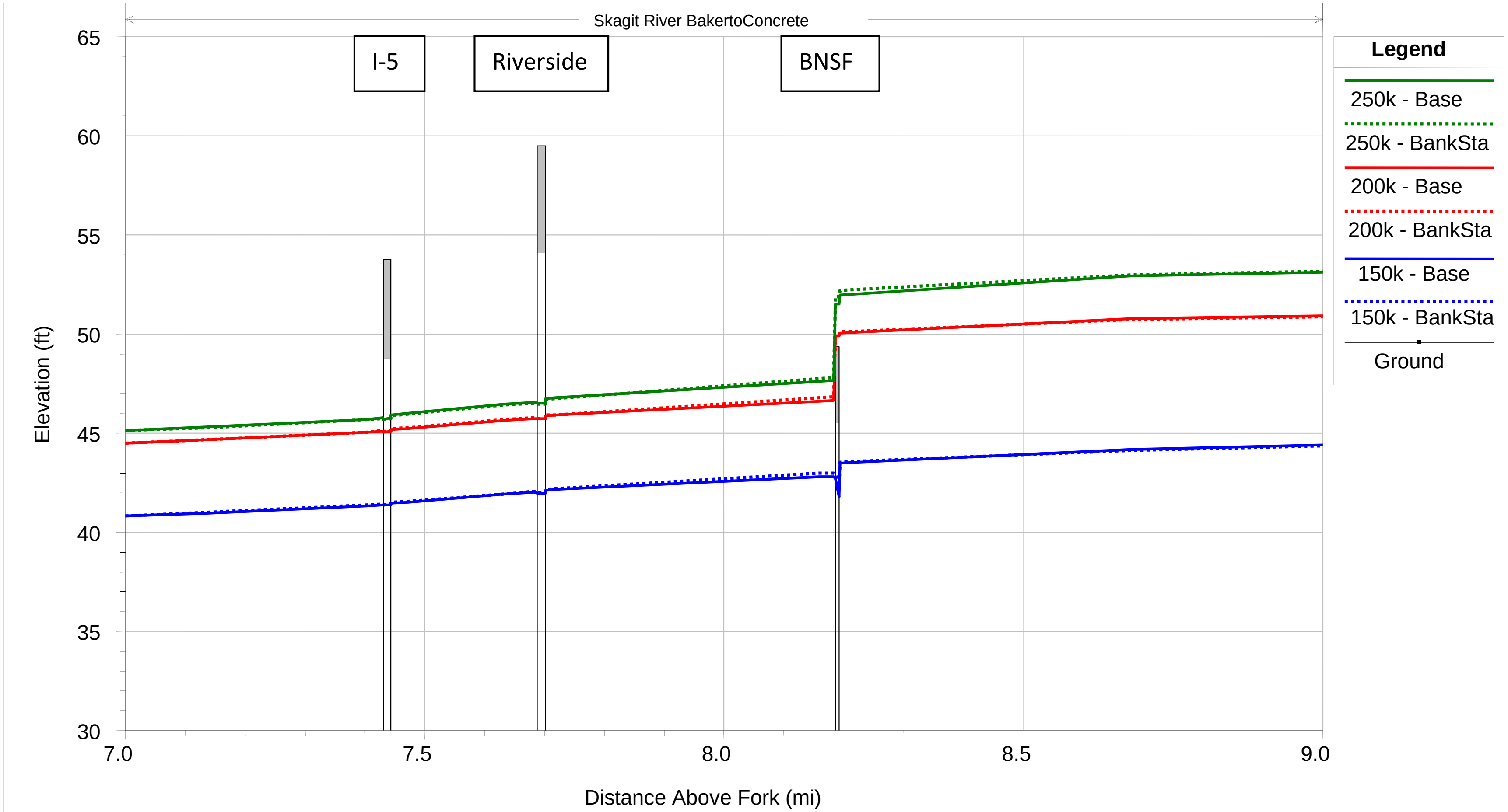
Notes: Base scenario with 0.1/0.3 contraction/expansion coefficients. Simulations assume no scour.

Figure A-17: 150,000 cfs, 200,000 cfs and 250,000 cfs Water Surface Profiles – Contraction/Expansion Coefficient Sensitivity.



Notes: Base scenario with right bank station at edge of low flow channel. Simulations assume no scour.

Figure A-18: Rating Curves – Bank Station Sensitivity.



Notes: Base scenario with right bank station at edge of low flow channel. Simulations assume no scour.

Figure A-19: 150,000 cfs, 200,000 cfs and 250,000 cfs Water Surface Profiles – Bank Station Sensitivity.

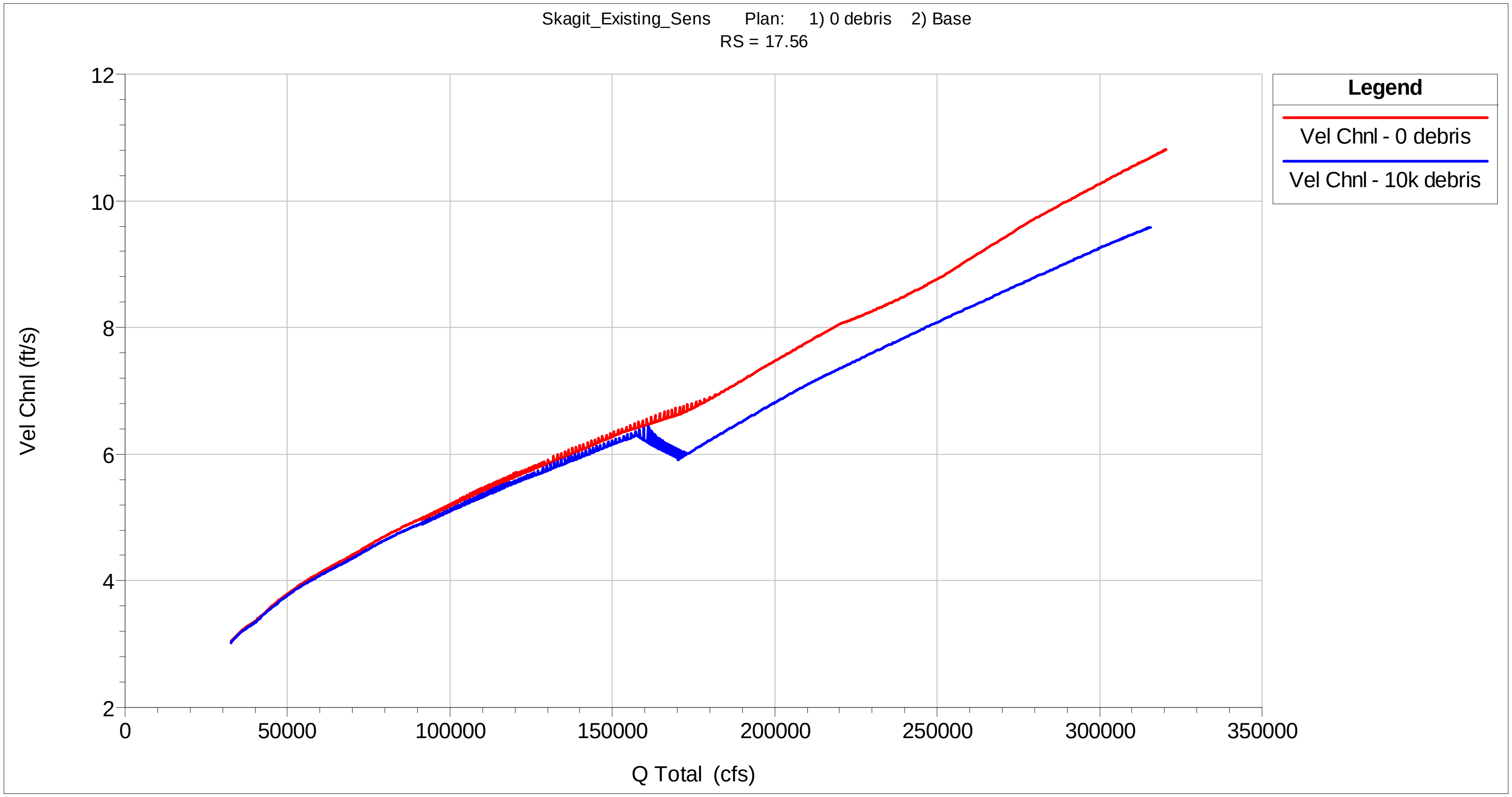


Figure A-20: Channel Approach Velocity

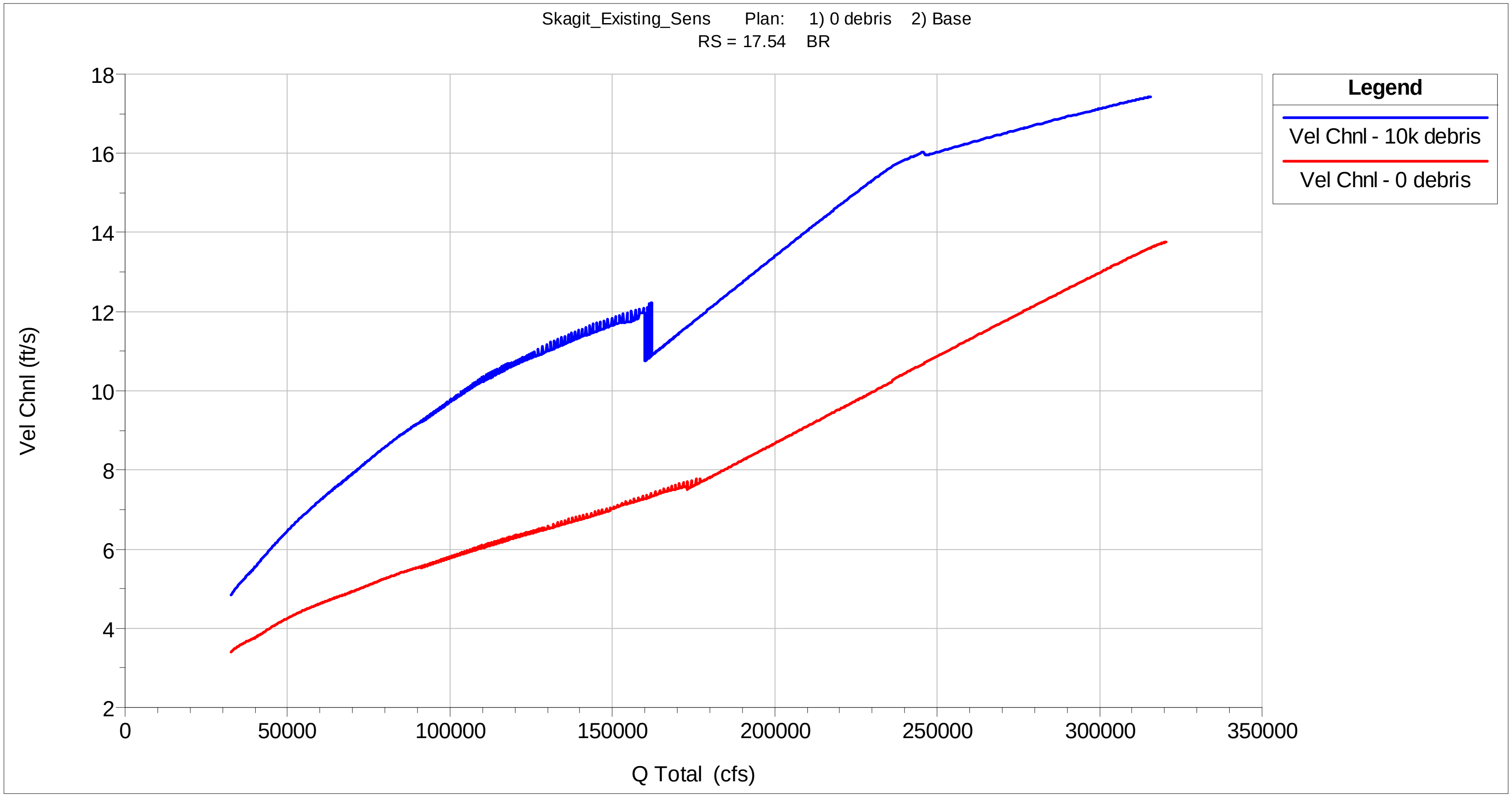


Figure A-21: Bridge Opening Channel Velocity

Figure A-22: BNSF Bridge at low flow (1993 – source and exact date unknown)

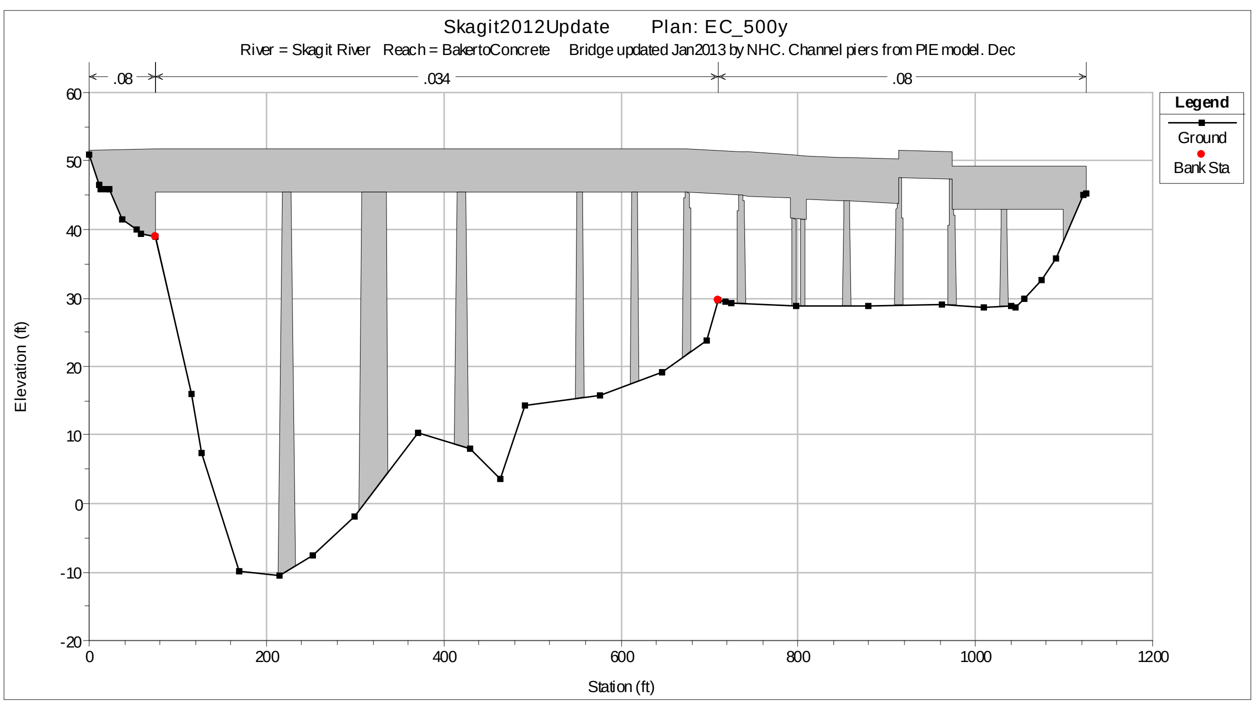
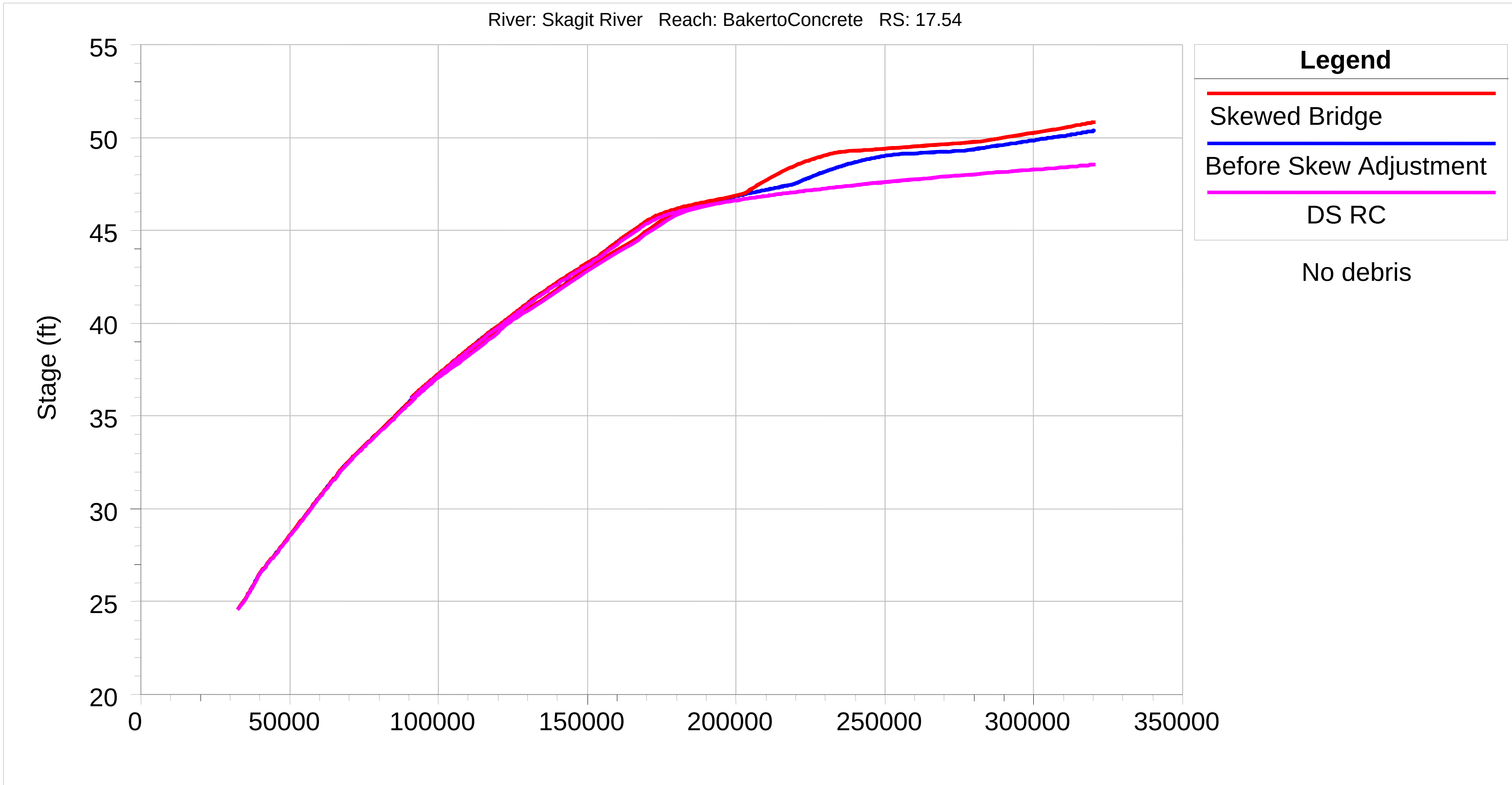
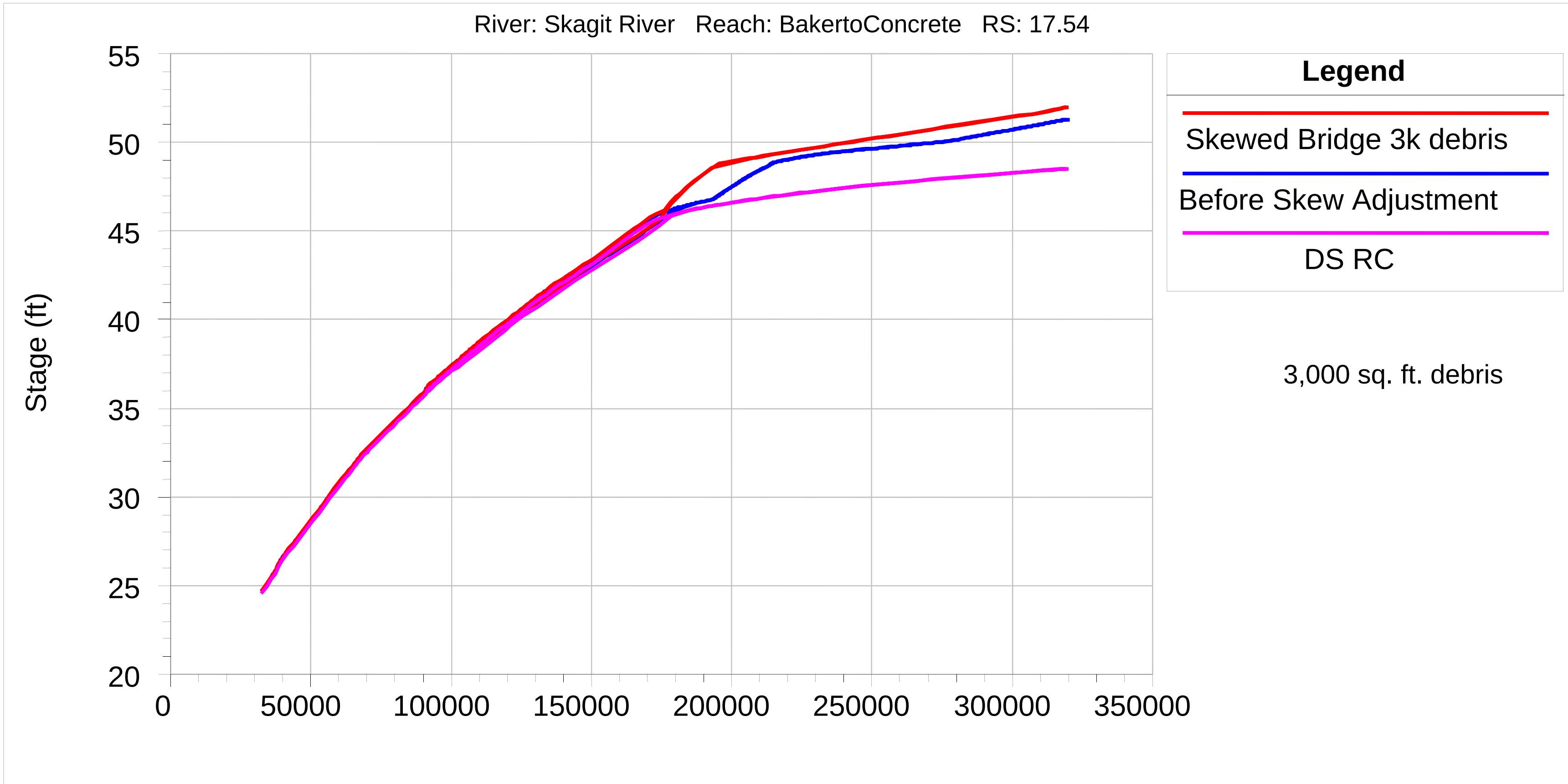


Figure A-23: Final BNSF bridge geometry after January 2013 refinements



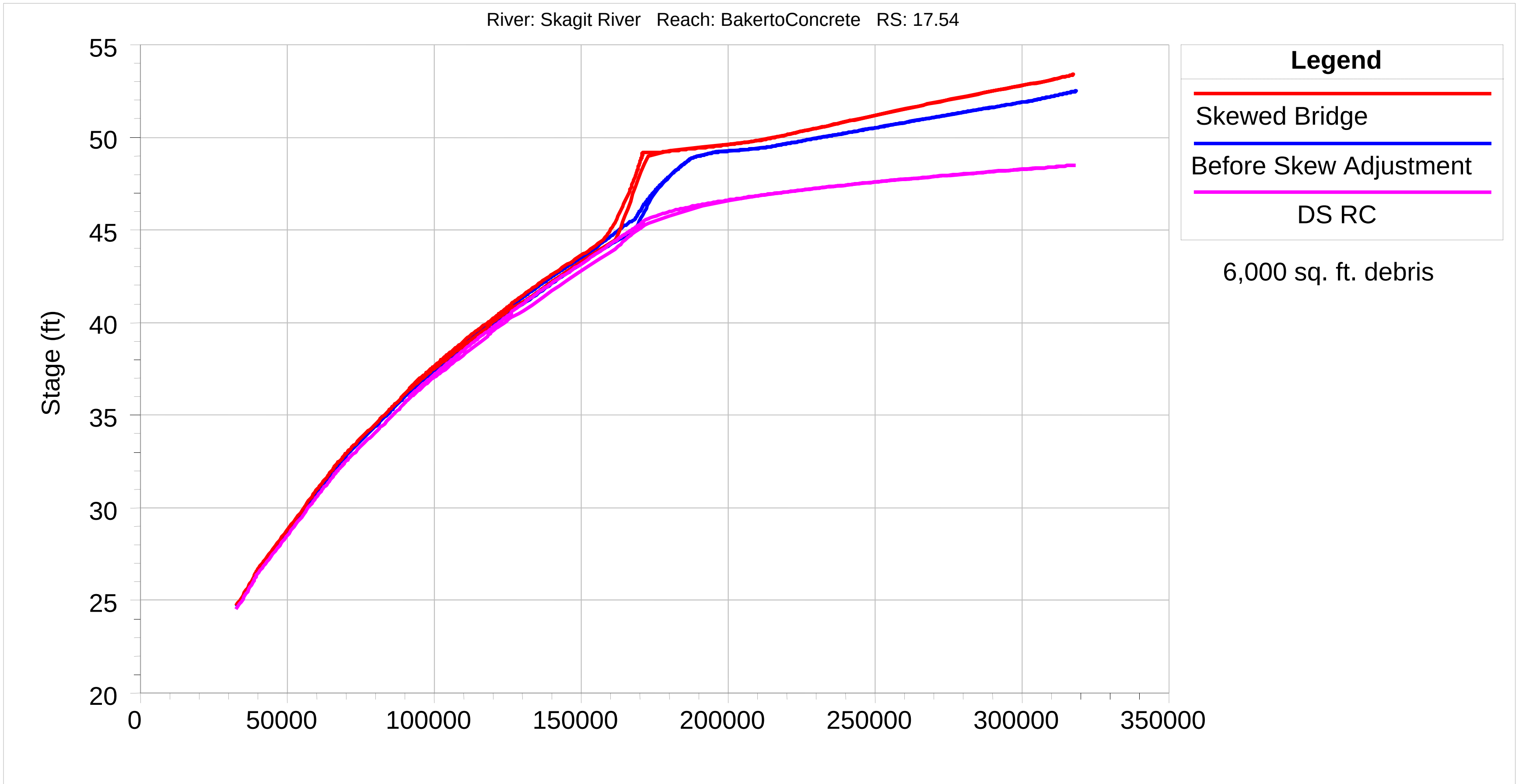
Note: Simulations assume no scour.

Figure A-24: Rating Curves – With and Without Skew Adjustment; No Debris.



Note: Simulations assume no scour.

Figure A-25: Rating Curves – With and Without Skew Adjustment; 3,000 sq. ft. of Debris.

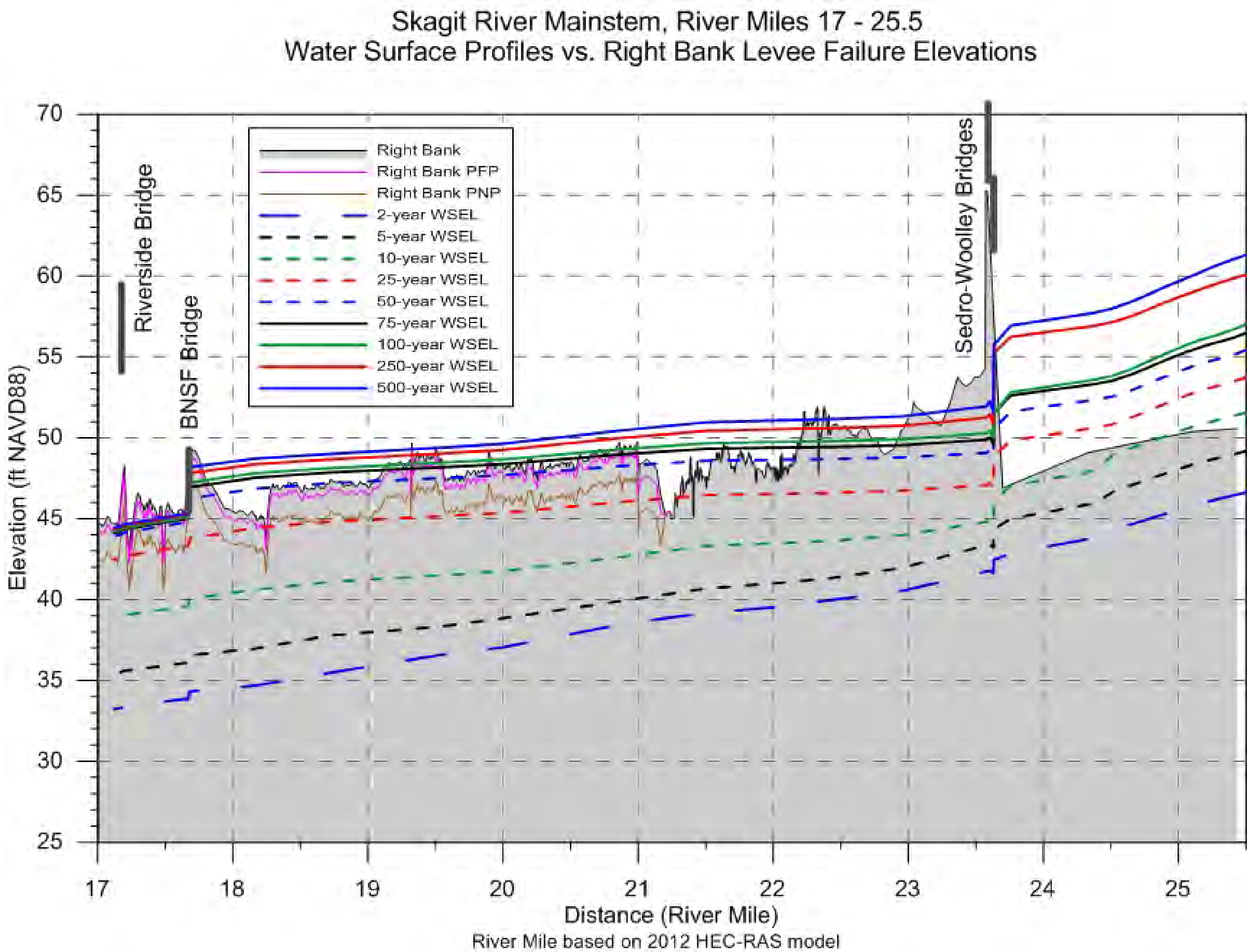


Note: Simulations assume no scour.

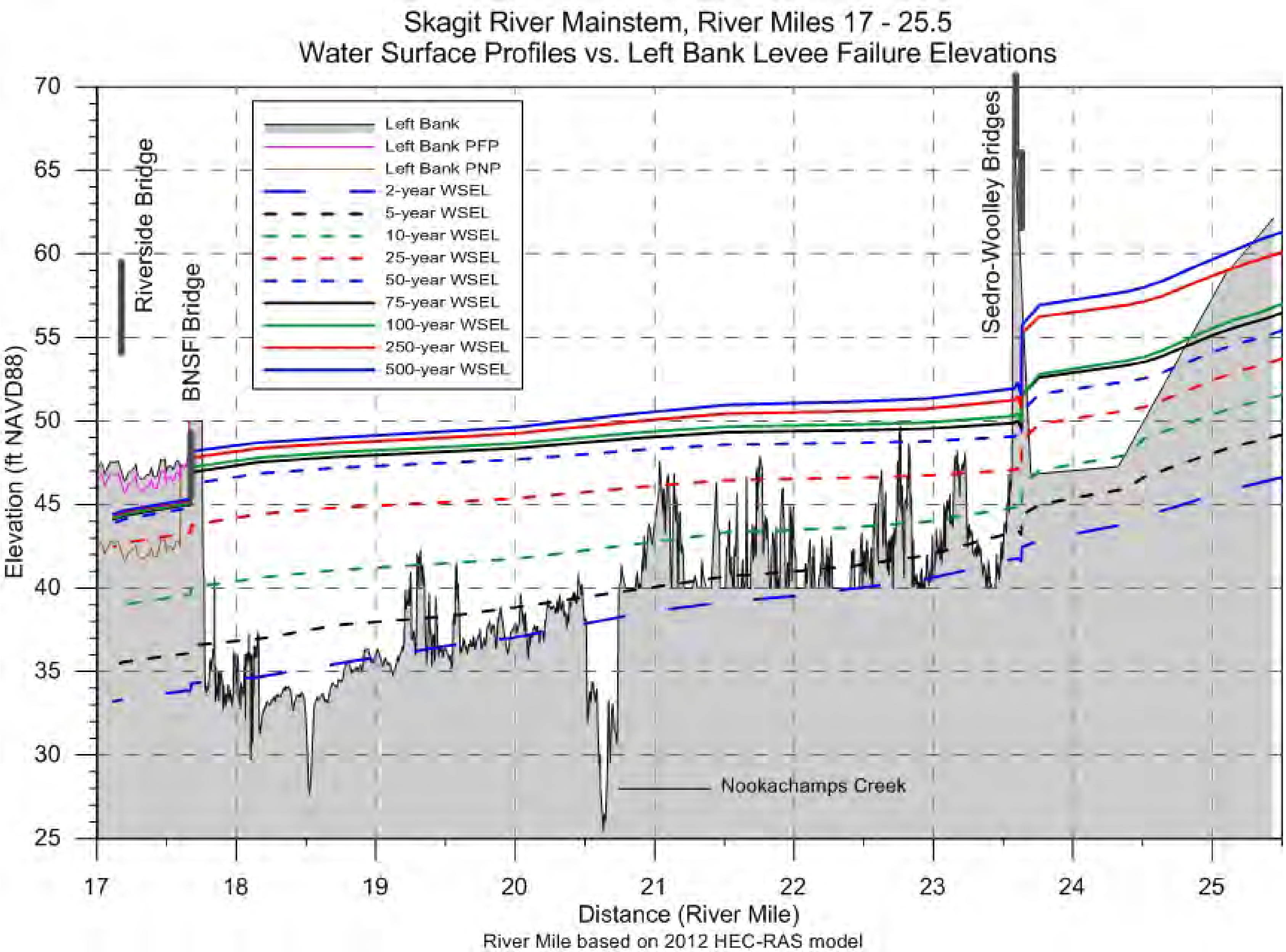
Figure A-26: Rating Curves – With and Without Skew Adjustment; 6,000 sq. ft. of Debris.

APPENDIX B

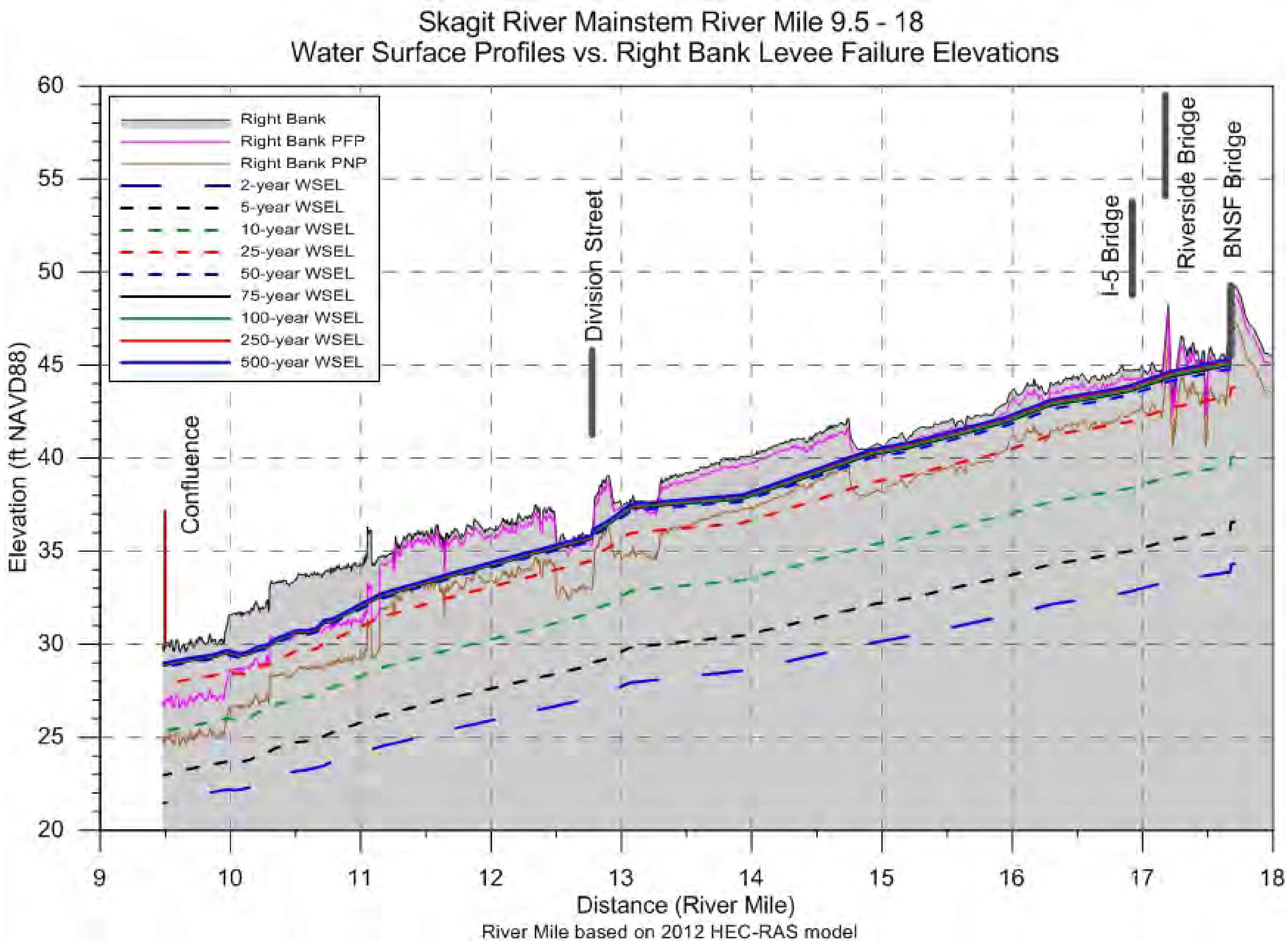
Water Surface Profiles for No Breach Scenario

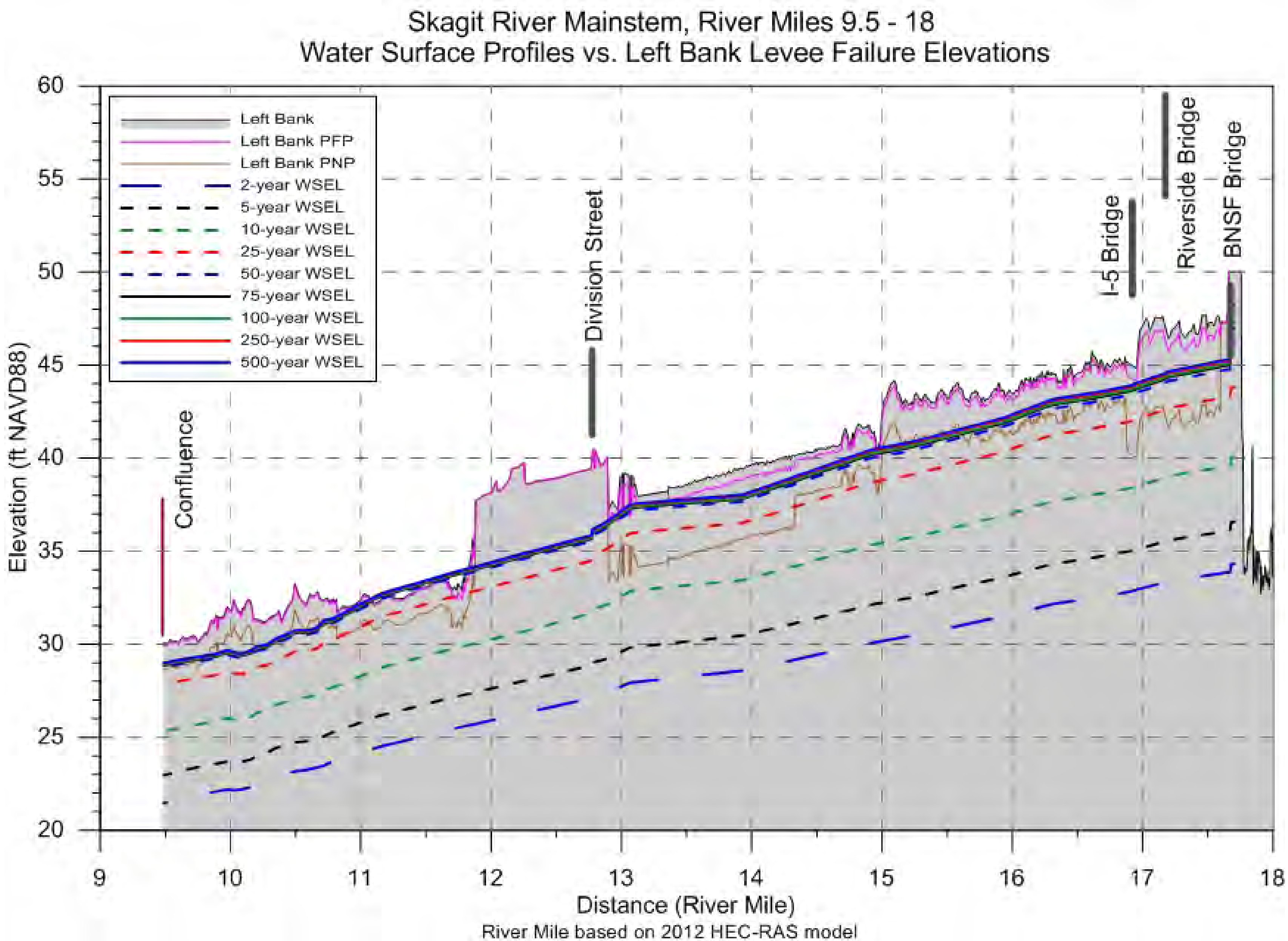


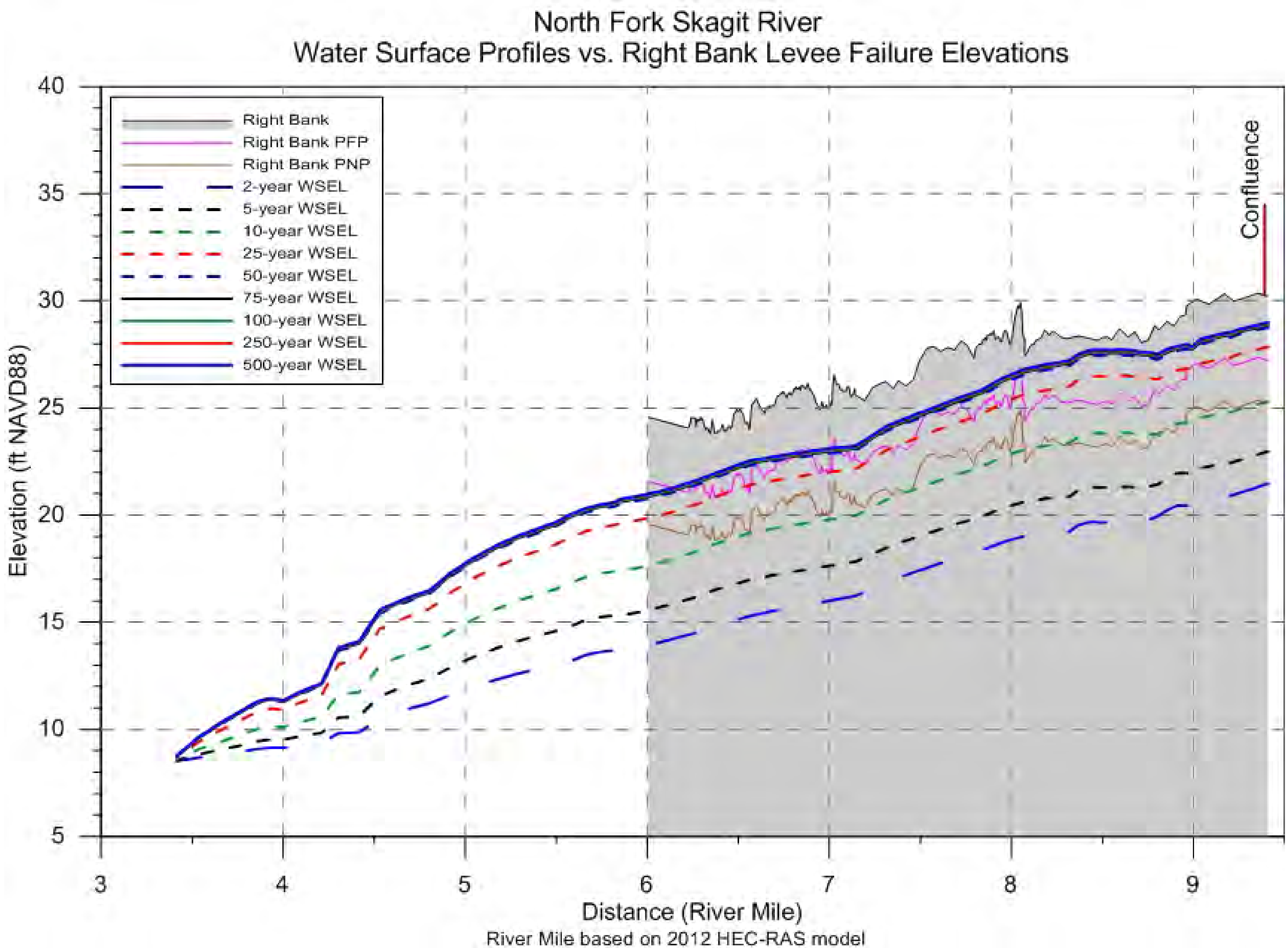
Skagit River Mainstem, River Miles 17 - 25.5
Water Surface Profiles vs. Right Bank Levee Failure Elevations
Right Bank
Right Bank PFP
Right Bank PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Riverside Bridge
BNSF Bridge
Sedro-Woolley Bridges
Elevation (ft NAVD88)
70
65
60
55
50
45
40
35
30
25
17
18
19
20
21
22
23
24
25
Distance (River Mile)
River Mile based on 2012 HEC-RAS model

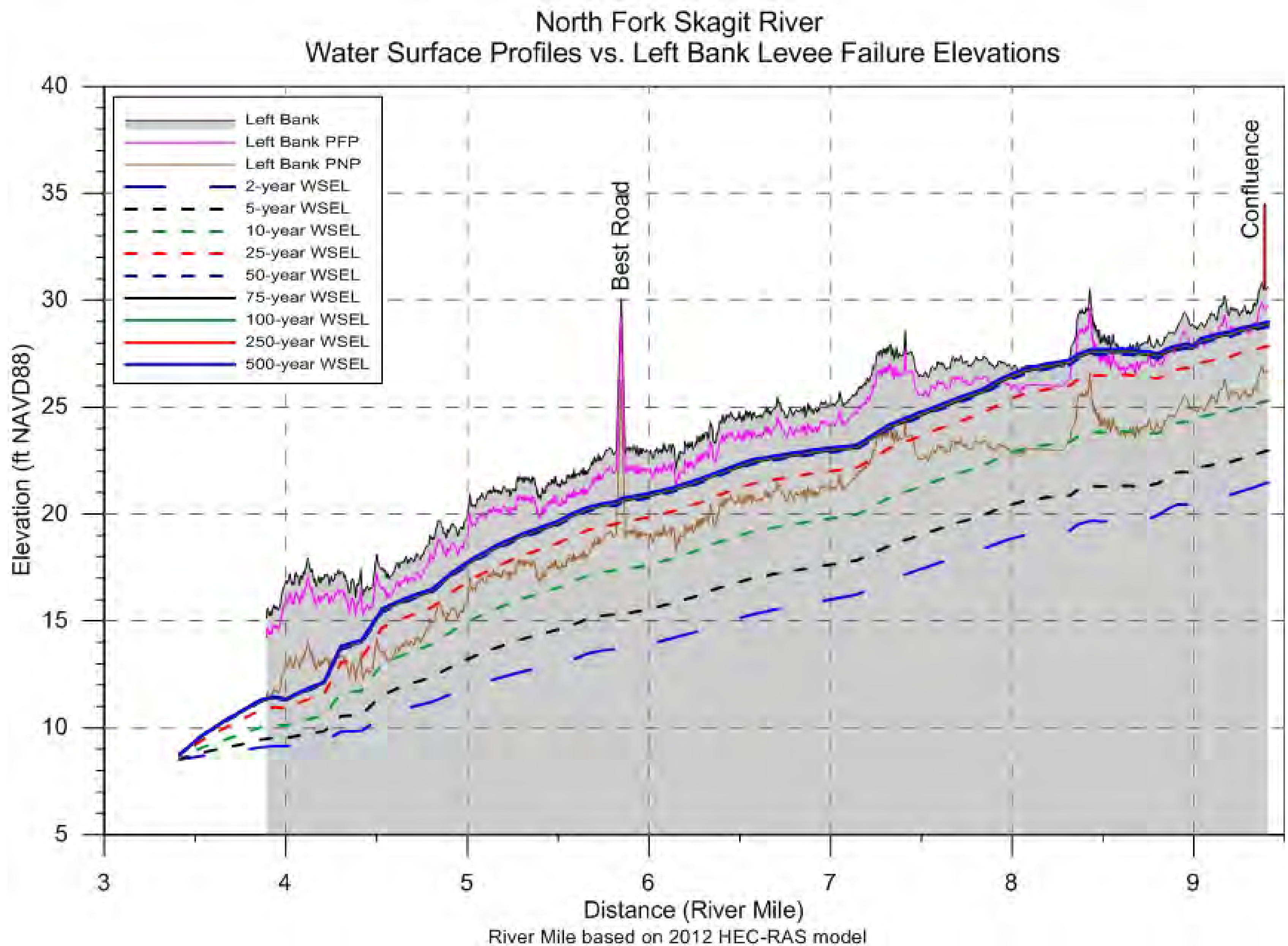


Skagit River Mainstem, River Miles 17 - 25.5
Water Surface Profiles vs. Left Bank Levee Failure Elevations
Left Bank
Left Bank PFP
Left Bank PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Riverside Bridge
BNSF Bridge
Sedro-Woolley Bridges
Nookachamps Creek
Elevation (ft NAVD88)
Distance (River Mile)
River Mile based on 2012 HEC-RAS model

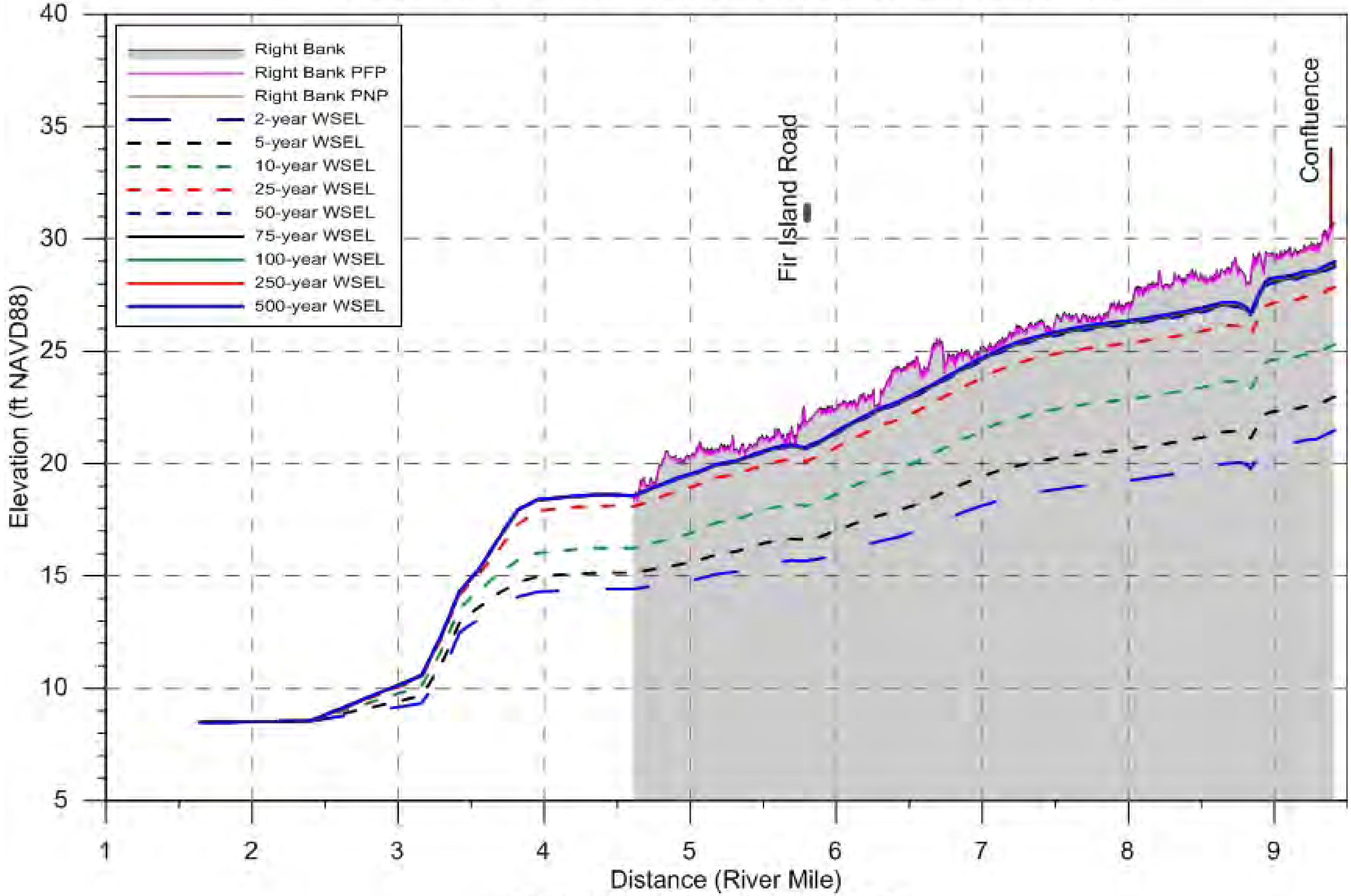

Skagit River Mainstem River Mile 9.5 - 18
Water Surface Profiles vs. Right Bank Levee Failure Elevations
Right Bank
Right Bank PFP
Right Bank PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Confluence
Division Street
I-5 Bridge
Riverside Bridge
BNSF Bridge
Elevation (ft NAVD88)
60
55
50
45
40
35
30
25
20
9
10
11
12
13
14
15
16
17
18
Distance (River Mile)
River Mile based on 2012 HEC-RAS model


Skagit River Mainstem, River Miles 9.5 - 18
Water Surface Profiles vs. Left Bank Levee Failure Elevations
Left Bank
Left Bank PFP
Left Bank PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Confluence
Division Street
I-5 Bridge
Riverside Bridge
BNSF Bridge
Elevation (ft NAVD88)
60
55
50
45
40
35
30
25
20
9
10
11
12
13
14
15
16
17
18
Distance (River Mile)
River Mile based on 2012 HEC-RAS model

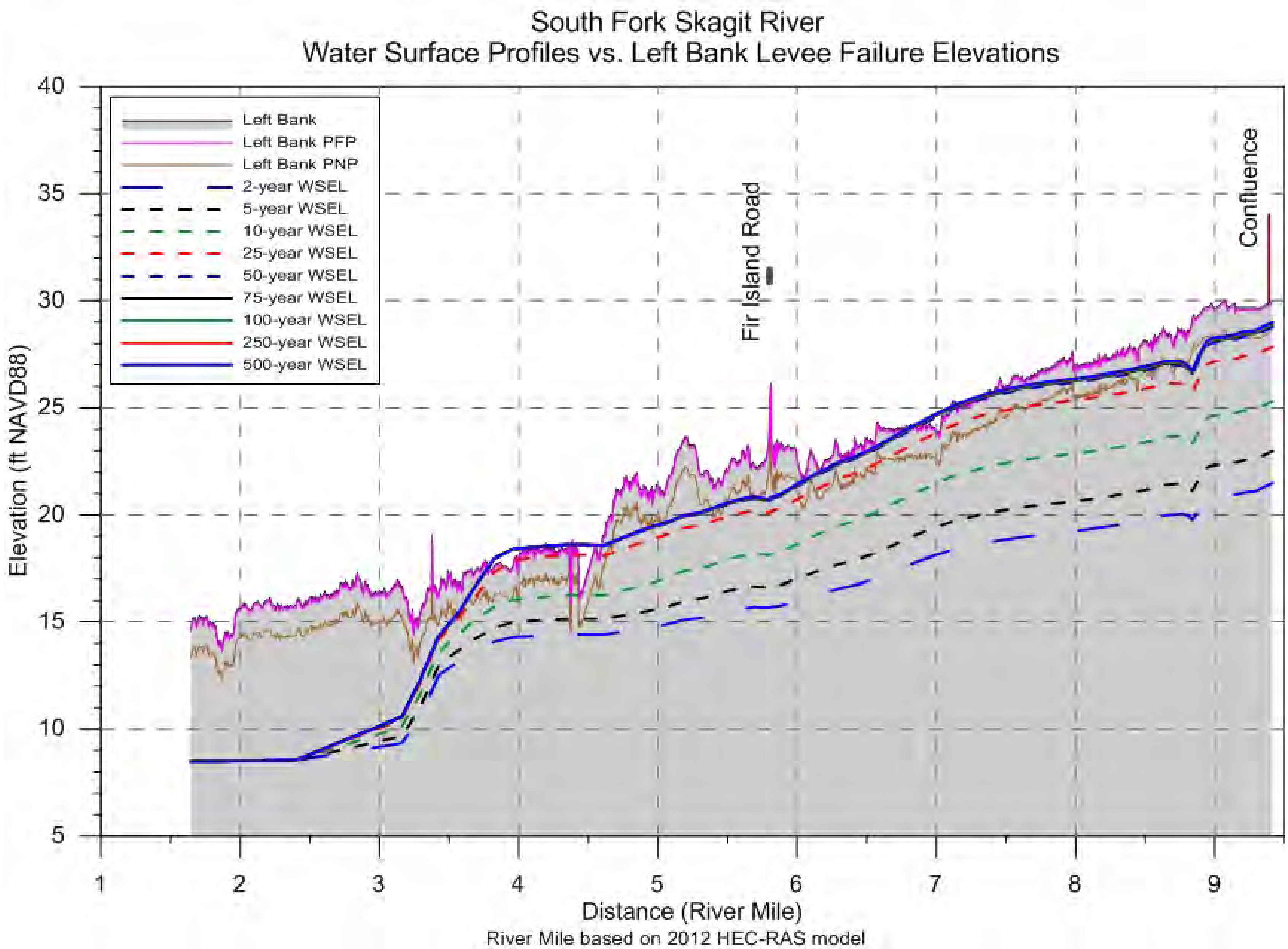

North Fork Skagit River
Water Surface Profiles vs. Right Bank Levee Failure Elevations
Right Bank
Right Bank PFP
Right Bank PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Confluence
Elevation (ft NAVD88)
40
35
30
25
20
15
10
5
3
4
5
6
7
8
9
Distance (River Mile)
River Mile based on 2012 HEC-RAS model


North Fork Skagit River
Water Surface Profiles vs. Left Bank Levee Failure Elevations
Left Bank
Left Bank PFP
Left Bank PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Best Road
Confluence
Elevation (ft NAVD88)
40
35
30
25
20
15
10
5
3
4
5
6
7
8
9
Distance (River Mile)
River Mile based on 2012 HEC-RAS model

South Fork Skagit River
Water Surface Profiles vs. Right Bank Levee Failure Elevations



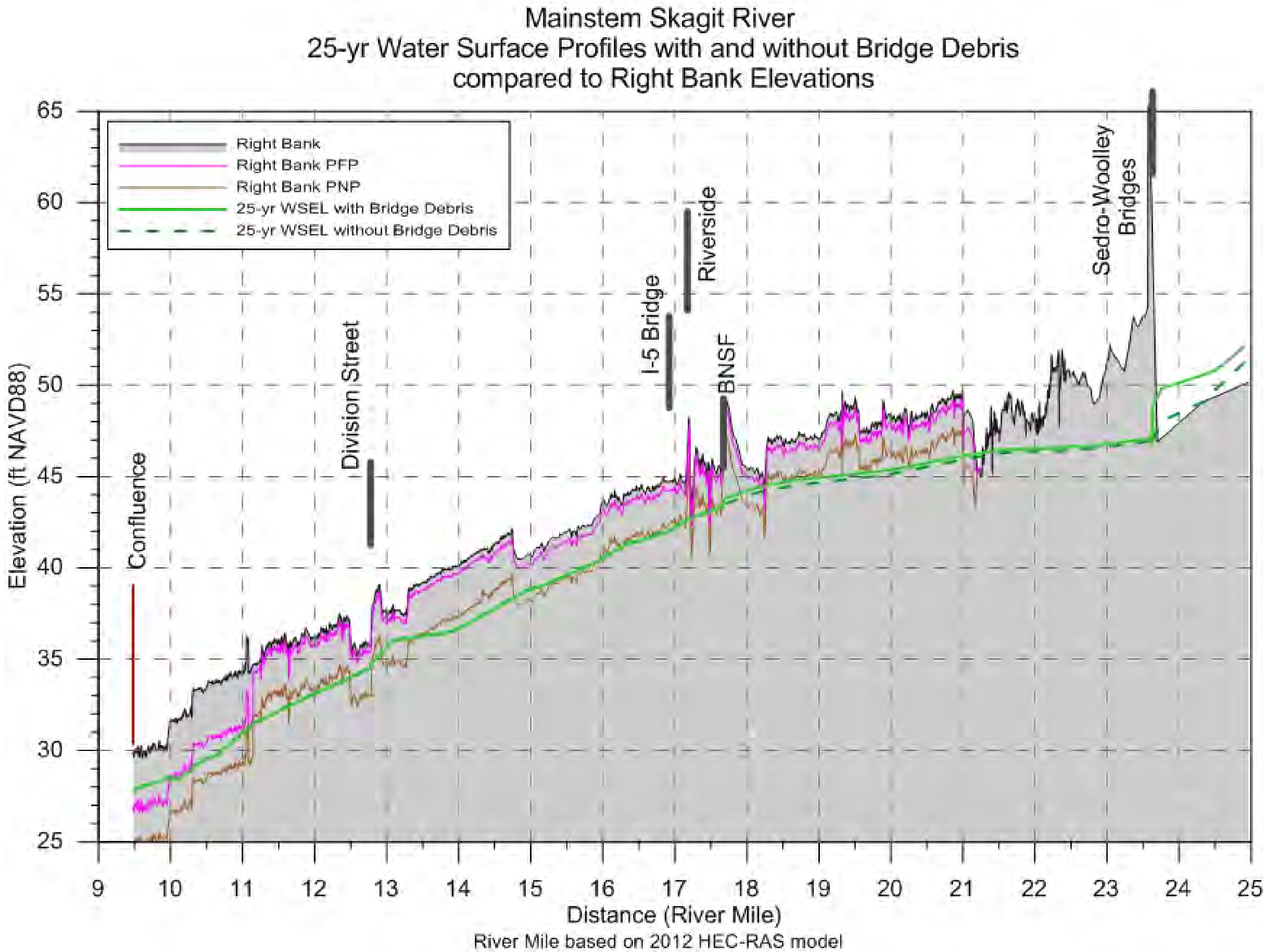
River Mile based on 2012 HEC-RAS model



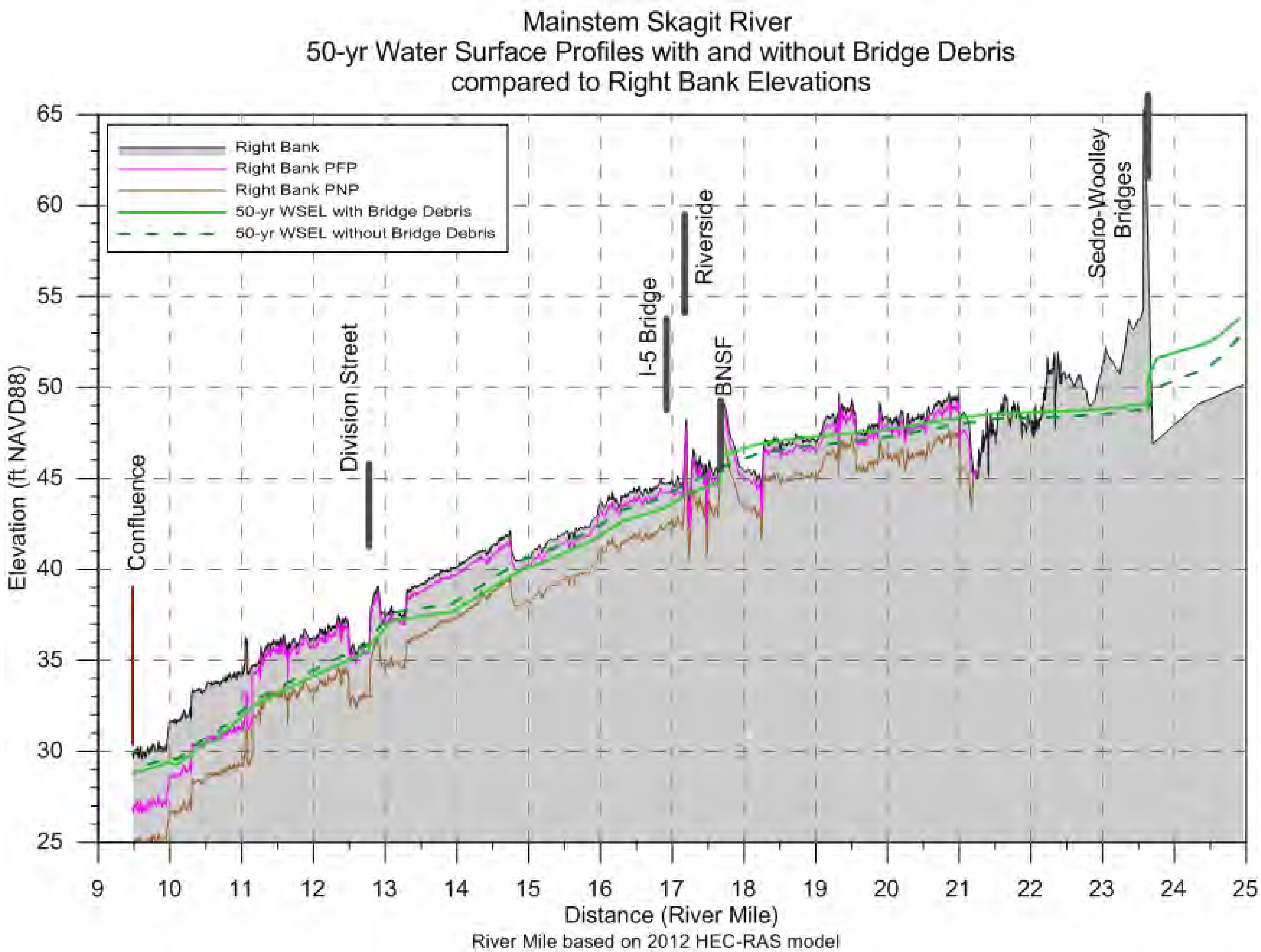
South Fork Skagit River
Water Surface Profiles vs. Left Bank Levee Failure Elevations
Left Bank
Left Bank PFP
Left Bank PNP
2-year WSEL
5-year WSEL
10-year WSEL
25-year WSEL
50-year WSEL
75-year WSEL
100-year WSEL
250-year WSEL
500-year WSEL
Fir Island Road
Confluence
Elevation (ft NAVD88)
40
35
30
25
20
15
10
5
1
2
3
4
5
6
7
8
9
Distance (River Mile)
River Mile based on 2012 HEC-RAS model

APPENDIX C

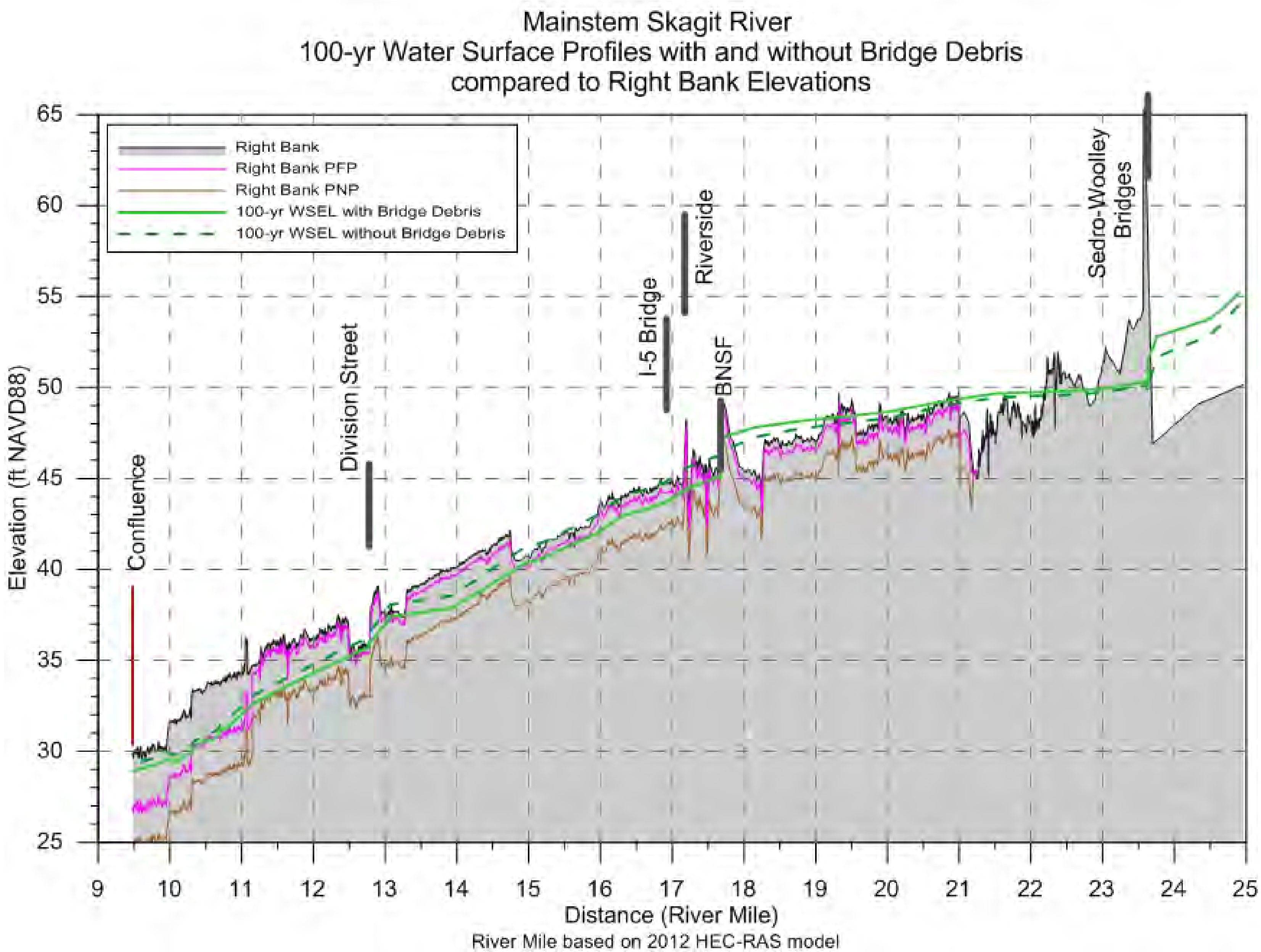
Water Surface Profiles for BNSF Bridge Debris Scenarios



Mainstem Skagit River
25-yr Water Surface Profiles with and without Bridge Debris
compared to Right Bank Elevations
Right Bank
Right Bank PFP
Right Bank PNP
25-yr WSEL with Bridge Debris
25-yr WSEL without Bridge Debris
Confluence
Division Street
I-5 Bridge
Riverside
BNSF
Sedro-Woolley Bridges
Elevation (ft NAVD88)
65
60
55
50
45
40
35
30
25
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
Distance (River Mile)
River Mile based on 2012 HEC-RAS model



Mainstem Skagit River
50-yr Water Surface Profiles with and without Bridge Debris
compared to Right Bank Elevations
Right Bank
Right Bank PFP
Right Bank PNP
50-yr WSEL with Bridge Debris
50-yr WSEL without Bridge Debris
Confluence
Division Street
I-5 Bridge
Riverside
BNSF
Sedro-Woolley Bridges
Elevation (ft NAVD88)
65
60
55
50
45
40
35
30
25
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
Distance (River Mile)
River Mile based on 2012 HEC-RAS model



Mainstem Skagit River
100-yr Water Surface Profiles with and without Bridge Debris
compared to Right Bank Elevations
Right Bank
Right Bank PFP
Right Bank PNP
100-yr WSEL with Bridge Debris
100-yr WSEL without Bridge Debris
Confluence
Division Street
I-5 Bridge
Riverside
BNSF
Sedro-Woolley Bridges
Elevation (ft NAVD88)
Distance (River Mile)
River Mile based on 2012 HEC-RAS model
65
60
55
50
45
40
35
30
25
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25

Skagit River Basin
Skagit River Flood Risk Management Feasibility Study

FINAL REPORT

HYDROLOGY TECHNICAL DOCUMENTATION

US Army Corps of Engineers

August 2013

HYDROLOGY TECHNICAL DOCUMENTATION
TABLE OF CONTENTS

APPENDICES

Important Note on Elevations and Vertical Datum

Elevations in this document are reported to a variety of vertical datums including NGVD29, NAVD88 and local datums, and are provided for general context or general information purposes only; elevations should be checked before being used for any other purpose.

1.0 Background

1.1 General

Authority for the Skagit River, Washington, flood risk management feasibility study is derived from Section 209 of the Flood Control Act of 1962 (Public Law 87-874). Section 209 authorized a comprehensive study of Puget Sound and Adjacent Waters, including tributaries such as the Skagit River, in the interest of flood risk management, navigation, and other water uses and related land resources. The current feasibility study was initiated in 1997 as an interim study under this statutory authority. Skagit County is the local sponsor of the feasibility study and is providing a combination of cash and in-kind services equaling 50 percent of the total study effort. The purpose of the study is to formulate and recommend a comprehensive flood risk management plan for the Skagit River floodplain that will reduce flood risk in Skagit County with a focus on downstream of Sedro-Woolley.

The authorization for the Skagit River Flood Risk Management Feasibility Study necessitated hydrologic and hydraulic analysis of the Skagit River basin. This allows for a basin-wide, systematic evaluation of the Skagit River. These analyses incorporate historical rainfall-runoff, reservoir operations, and flow along the major river systems to effectively evaluate the hydraulic performance of the flood management systems. The models can be used to assess the performance of the current systems or modified systems under a wide range of hydrologic conditions.

1.2 Purpose of Documentation

The main goal of the hydrologic analysis is to provide the hydrologic inputs necessary to adequately evaluate potential flood risk management measures. The main product components of this effort include:

- Description of the hydrologic analysis methodology
- Development of flows necessary to characterize the 2-, 5-, 10-, 25-, 50-, 75-, 100-, 250-, and 500-year flood events for the Skagit River Basin

1.3 Study Area

The study area encompasses the mainstem Skagit River from Skagit Bay to Ross Dam, the Baker River from the confluence with the Skagit to Upper Baker Dam, the Sauk River from the confluence with the Skagit to the Sauk River at Sauk gage, and the Cascade River from the confluence of the Skagit to the old Cascade River at Marblemount gage. The Skagit River basin has a drainage area of 3,115 square miles.

1.4 Study and Technical Review Chronology

Draft Hydrology Technical Documentation for the Skagit River Flood Risk Management Feasibility Study was produced by the Seattle District USACE in August 2004, with technical review by the Hydrologic Engineering Center. Hydrologic analyses for the study were subsequently revised and updated by the Seattle District primarily to incorporate additional hydrologic data and to account for revisions by the US Geological Survey to published peak discharges for historic floods. However the Hydrology Technical Documentation was not updated at that time. Further revisions to the hydrologic analyses and preparation of a March 2011 update to the Hydrology Technical Documentation were carried out by Northwest Hydraulic Consultants Inc. (NHC) under contract to the local sponsor, Skagit County (contract C20080424, Task Assignment 4, authorized 15 October 2009). Significant revisions or analyses conducted for the March 2011 update by NHC, in close consultation with the Seattle District, included:

- Use of computed probability flood quantiles throughout, consistent with requirements for subsequent risk and uncertainty analysis. (Earlier work incorporated an expected probability adjustment to flood frequency estimates).
- Analysis of the effects of seasonal variation in available flood control storage at Upper Baker and Ross reservoirs.
- Modification to "best" and "worst" case reservoir regulation scenarios to provide more realistic inputs for subsequent risk and uncertainty analyses.
- Reanalysis and downward adjustment of Nookachamps Creek coincident flows, incorporating hydrologic data either not used or not available for earlier work.
- Estimation of coincident flood hydrographs for Samish River, flows from which comingle with right bank Skagit River floodplain flows.

The present report is a further update to the March 2011 Hydrology Technical Documentation. Hydrologic analysis and preparation of the present August 2013 update were carried out by NHC under contract to the Seattle District USACE (contract W912DW-11-D-1006, Task Order No. 3). The principal revisions comprised:

- Updated analysis of the effects of seasonal variation in available flood control storage at Upper Baker and Ross reservoirs, including comprehensive update and revisions to Appendix G.
- Adoption of weighted regulated hydrographs to account for the effects of seasonal variation in flood control storage in place of previous "best" and "worst" case reservoir regulation scenarios.
- Updated routing of regulated and unregulated flows using the most recent (February 2013) HEC-RAS model of the lower Skagit River which includes revisions to the model representation of the BNSF railroad bridge at about

RM 17.6 and other model corrections and refinements. (Revisions to the HEC-RAS model are described in the study Hydraulic Technical Documentation)

The hydrologic analyses conducted by the USACE have relied on discharge data published by the USGS, including the USGS-published estimates of peak discharges for the historic floods of water years 1898, 1910, 1918 and 1922 on the Skagit River near Concrete. Particular attention has focused on the estimated magnitudes of these events since they have a significant influence on estimates of Skagit River flood quantiles. Reviews have been performed by County consultants (NHC 2010, NHC 2007, and PIE 2004), federal agencies (USGS 2010, FEMA 2010, USGS 2006, and FEMA 2006), and City of Burlington (PIE 2010 and PIE 2008). Reassessments of the magnitude of the historic floods were conducted by the USGS following the flood of October 2003 (USGS 2005), and again following the flood of November 2006 (USGS 2007). The USGS 2007 reevaluation resulted in a downward adjustment of about 5% in the estimated magnitude of the historic floods to produce the current published values which provide the basis for the updated hydrologic analyses presented both in the March 2011 report and in this report.

A chronological list of **selected** flood hydrology reports, reviews and reevaluations is provided in Appendix A. Many of the documents referred to in Appendix A can be found at www.skagitriverhistory.com.

2.0 General Basin Characteristics

The Skagit River basin is located in the northwest corner of the State of Washington (see Figure 1). The Skagit River drainage area is 3,115 square miles and the basin extends about 110 miles in the north-south direction and about 90 miles in the east-west direction between the crest of the Cascade Range and Puget Sound. The northern end of the basin extends 28 miles into Canada.

The Skagit River originates in a network of narrow, precipitous mountain canyons in Canada and flows west and south into the United States where it continues l35 miles to Skagit Bay. Skagit River falls rapidly from its source to an elevation of 1600 ft at the United States-Canadian Border. Stream profiles on Figure 2 show that within the first 40-miles south of the International Border, the River falls 1,l00 feet and that the remaining 500 feet fall is distributed along the 95 miles of the lower river.

The Skagit River crosses a broad outwash plain between Sedro-Woolley and the river mouth. Immediately downstream from Mount Vernon, the river divides into two principal distributaries, the North Fork and the South Fork. These two distributaries carry about 60 percent and 40 percent of the normal flows of the Skagit River, respectively. During floods, flows on the two distributaries are approximately equal.

The Skagit Valley, the 100,000-acre valley area downstream from the town of Concrete, contains the largest residential and farming developments in the basin. The 32-mile long valley between Concrete and Sedro-Woolley is made up of mostly cattle and dairy pasture land and wooded areas. West of Sedro-Woolley, the flood plain forms a large alluvial fan with an east-west width of about 11 miles and a north-south width of about l9 miles.

2.1 Topography

A major portion of the Skagit River basin lies on the western slopes of the Cascade Range. Most of the eastern basin is mountainous land above an elevation of 6,000 ft. The two most prominent topographical features in the basin are Mount Baker at an elevation of l0,778 feet on the western boundary of the Baker River basin, and Glacier Peak at an elevation of l0,568 ft in the Sauk River subbasin. In the eastern basin, 22 peaks are above an elevation of 8,000 ft. The upper reaches of nearly all tributaries are situated in precipitous steep-walled mountain valleys.

The Skagit River flows in a l-mile to 3-mile wide valley from Rockport to Sedro-Woolley. In this section, the valley walls are moderately steep timbered hillsides with few developments. Below Sedro-Woolley, the valley falls to nearly sea level and widens to a flat, fertile outwash plain that joins the Samish valley along the northeast side of the valley and extends west through Mount Vernon to La Conner and south to the Stillaguamish River near Stanwood.

2.2 Geology

The eastern mountainous region of the upper Skagit Basin consists of ancient metamorphic rocks, largely phyllites, slates, shales, schists, and gneisses together with intrusive granitic rocks and later andesitic lavas and pyroclastic deposits associated with Mount Baker and Glacier Peak. The valleys are generally steep sided and frequently flat floored. Valley walls are generally mantled with a mixture of rocky colluvium, and to a considerable elevation, by deposits of continental and alpine glaciation. These deposits are a heterogeneous mixture of sand and gravel together with variable quantities of silt and clay depending on the mode of deposition. Some of these deposits are highly susceptible to land sliding when saturated.

The floodplain of the Skagit River below Concrete is composed of sands and gravels that diminish to sands, silts, and some clays further downstream. Below Hamilton, fine-grained floodplain sediments predominate. The Baker River valley in the vicinity of the Baker Lake is geologically quite different from most of the other Skagit tributaries. This is largely due to the influence of Mount Baker, a volcanic cone rising to an elevation of l0,778 feet, that sets astride the western boundary of the Baker River basin.

Present bedrock exposures adjacent to Ross Lake consist of Chilliwack sediments, volcanics and granitics, Skagit gneiss, and Nooksack group phyllite. The continental ice movement and mountain glaciers sculpted the basic geological forms and rock types into the major landforms that are recognizable today. A large mass of metamorphic rock, known as the Skagit gneiss, forms the foundation rock for all three of the Skagit River Project hydroelectric plants. The age of its parent strata is presumed to be Paleozoic. The resistance to erosion provided by the massive gneiss is undoubtedly the reason for the narrow gorge of the Skagit River where the dams are located. Alpine glaciers have contributed to the steepness of the valley sides and to the depth of the valley bottoms. Over ten thousand years ago the upper Skagit Valley and the peaks were severely glaciated, removing not only the soil, but much of the loose rock. Many river channels created during the glacial melt have continued to aggrade, and as a result of that glacial action, the bedrock bottoms of most canyons are covered with glacial alluvium.

2.3 Sediment

Predicted rates of bed accumulation for 100 years in the Skagit River system vary in depth from 4 feet at the mouth of the 2 distributaries, the North and South Forks of the Skagit River, to 2 feet at Mount Vernon. The 2 feet of depth continues upstream to Burlington. The River annually transports about 10,000,000 tons of sediment of mostly glacial origin. Size of bed material, as determined by field observations and samples, varies from 1/4-inch to 3/4-inch gravel and coarse sand at Mount Vernon to medium and fine sand near the River mouths. From Burlington to Concrete, channel sediments are predominantly fine-to-coarse sands, gravels, and cobbles together with small quantities of silt and clay.

2.4 Climate

The major factors influencing the climate of the Skagit River basin are terrain, proximity of the Pacific Ocean, and the position and intensity of the semi-permanent high and low pressure centers over the north Pacific. The basin lies about 100 miles inland from the moisture supply of the Pacific Ocean. Westerly air currents from the ocean prevail in these latitudes bringing the region considerable moisture, cool summers, and comparatively mild winters. Annual precipitation throughout the basin varies markedly due to elevation and topography. Major storm activity occurs during the winter when the basin is subject to rather frequent ocean storms that include heavy frontal rains associated with cyclonic disturbances generated by the semi-permanent Aleutian Low. During the summer months, the weather is relatively warm and dry due to increased influence of the semi-permanent Hawaiian high-pressure system. A summary of precipitation, snowfall, and temperature data for twelve representative stations is provided in Table 1. The locations of climatological stations in or near the basin, station elevations, and periods of record are shown on Figure 3.

2.4.1 Temperature

Normal monthly mean temperature data for eight representative stations are presented in Table 2. The mean annual temperature for stations in or near the basin varies from 47.8 degrees Fahrenheit (^{o}F) at Upper Baker Dam to 51.0^{o}F at Anacortes. Normal monthly temperatures vary in January from 32.9^{o}F at Ross Dam to 40.3^{o}F at Anacortes, and in August from 66.1^{o}F at Ross Dam to 62.7^{o}F at Anacortes. The temperature extremes recorded in the basin are 109^{o}F at Newhalem and -14^{o}F at Darrington Ranger Station.

TABLE 1 - SUMMARY OF CLIMATOLOGICAL DATA (STANDARD UNITS)

	ELEV. (feet)	PERIOD OF RECORD	ANNUAL PRECIP. MEAN (inches)	ANNUAL PRECIP. GREATEST (inches)	ANNUAL PRECIP. LEAST (inches)	SNOW FALL MEAN (inches)	ANNUAL TEMP. MEAN ° F	ANNUAL TEMP. HIGHEST ° F	ANNUAL TEMP. LOWEST ° F
ANACORTES	34	1893-2005	26.20	39.43	15.89	4.5	51.1	95	4
BAKER LAKE	674	1926-1934	102.88	133.39	69.26	58.1	NA	NA	NA
CONCRETE FS	199	1920-2005	68.13	93.12	46.85	24.8	50.9	106	-1
DARRINGTON RS	554	1926-2005	79.64	104.89	51.20	40.3	49.1	105	-14
DIABLO DAM	895	1934-2005	77.07	115.34	45.86	55.0	48.6	106	-10
MARBLEMOUNT RS	352	1941-2005	77.23	101.2	50.36	NA	NA	NA	NA
MT. BAKER LODGE	4,154	‘26-’42 ‘46-60	109.85	142.33	74.13	525.3	40.1	91	-12
NEWHALEM	529	1924-2005	81.41	104.22	47.59	36.6	49.6	109	-6
ROSS DAM	1236	1960-2005	57.31	79.11	38.66	47.5	48.6	101	-10
SEDRO WOOLLEY	64	1896-2005	46.44	69.2	28.18	8.4	50.8	99	-2
SILVERTON	1,479	1942-1987	112.61	151.27	77.03	88.0	46.7	103	0
UPPER BAKER DAM	694	1961-2005	101.83	132.61	68.61	52	47.8	102	-5

Records through 2005. NOT AVAILABLE (NA). RS = Ranger Station FS = Fish Trap

TABLE 2 - NORMAL MONTHLY MEAN TEMPERATURE DATA (°F)

STATION	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
Anacortes	40.3	42.4	45.5	49.8	54.9	59.0	62.3	62.7	58.8	51.5	44.7	40.5	51.0
Concrete	37.0	39.8	43.8	49.0	54.7	59.1	63.6	64.2	59.8	51.5	42.4	37.3	50.2
Darrington RS	35.4	38.9	43.8	49.4	55.8	60.3	65.2	65.4	59.8	50.3	41.0	35.6	50.1
Diablo Dam	33.6	36.7	41.5	47.5	54.4	59.7	64.8	65.8	59.8	49.9	39.7	34.3	49.0
Newhalem	34.6	37.2	41.8	47.6	54.1	58.9	63.9	64.6	59.4	49.8	40.2	35.1	48.9
Ross Dam	32.9	35.7	40.6	46.6	53.6	59.3	65.1	66.1	59.7	49.8	39.3	33.8	48.5
Sedro Woolley	39.1	41.8	45.6	49.9	55.1	59.3	62.8	63.5	58.8	51.2	43.9	39.3	50.9
Upper Baker Dam	33.4	36.5	40.8	46.5	52.8	57.6	62.4	63.0	57.9	49.2	39.5	34.2	47.8

Climatological normals based on record period 1971-2000

TABLE 3 - NORMAL MONTHLY MEAN PRECIPITATION DATA (INCHES)

STATION	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
Anacortes	3.69	2.49	2.21	1.86	1.63	1.51	1.06	1.04	1.36	2.25	4.14	3.81	27.05
Concrete	9.99	7.56	6.92	4.86	3.71	3.01	1.83	1.69	3.23	6.20	11.37	11.02	71.39
Darrington RS	11.16	9.43	8.39	5.32	3.96	3.00	1.80	1.80	3.51	7.12	13.34	12.15	80.98
Diablo Dam	11.38	8.45	7.12	4.72	3.30	2.49	1.85	1.74	3.23	7.47	14.36	12.76	78.87
Mount Vernon	4.22	2.85	2.81	2.53	2.42	1.95	1.20	1.34	1.70	2.89	4.83	3.96	32.70
Newhalem	11.62	8.75	7.10	4.71	3.53	2.80	2.07	1.82	3.26	7.32	13.46	13.06	79.50
Ross Dam	8.84	6.47	5.14	3.01	2.15	1.65	1.39	1.22	2.19	5.23	10.51	9.64	57.44
Sedro Woolley	5.77	4.11	4.15	3.76	3.03	2.85	1.77	1.62	2.68	3.97	6.88	5.97	46.56
Upper Baker Dam	14.34	11.05	9.75	6.42	5.06	3.69	2.64	2.11	4.27	9.09	16.47	15.70	100.59

Climatological normals based on record period 1971-2000.

2.4.2 Precipitation

Normal monthly mean precipitation data for nine stations are presented in Table 3 preceding this page. Average annual precipitation over the Skagit basin varies by about 150 inches. Mean annual precipitation is 40 inches or less near the mouth of the Skagit River and in the portion of the basin in Canada that lies in topographic rain shadows. Average precipitation of l80 inches or more falls on the higher elevations of the Cascade Range in the southern end of the basin and over the higher slopes of Mount Baker. The annual precipitation over the basin above the town of Mount Vernon averages 92 inches with approximately 75 percent of this amount falling during the 6-month period, October-March. The mean monthly precipitation at stations in or near the basin ranges from 1.04 inches in August at Anacortes to 16.47 inches in November at Upper Baker Dam. The mean annual precipitation at Upper Baker Dam and Diablo Dam is 100.59 inches and 78.87 inches, respectively. The maximum-recorded precipitation for one month was 4l.95 inches at Silverton in January 1953. Storm studies indicate that 5 to 6 inches of rainfall in a 24-hour period have occurred over much of the basin.

The locations of precipitation stations presented in Table 3 together with several other stations in the Skagit basin vicinity are shown on Figure 3. A basin normal annual isohyetal map is shown on Figure 4.

2.4.3 Snowfall

Snowfall in the Skagit River basin is dependent upon elevation and proximity to the moisture supply of the ocean. The mean annual snowfall at stations in the basin varies from 4.5 inches at Anacortes to 525.3 inches at Mount Baker Lodge, with a maximum recorded value of 1,140 inches at Mount Baker Lodge during the July 1998 through June 1999 season. Snow surveys have been made within the Skagit River basin since 1943. Locations of snow courses in the basin are shown on Figure 3.

2.4.4 Wind

Surface wind speeds in the basin are the result of the pressure gradient between high- and low-pressure cells, storm intensity, and topographic effects. Prevailing winds in the lower basin are generally from the southerly quadrant from September through May and from the northerly quadrant from June through August. In the upper valleys above Concrete, the airflow is subject to a topographic funneling effect and is generally up the valley in the winter and down slope in the summer. A diurnal change in direction often occurs in the summer. Occasionally in the winter, cold continental air from eastern Washington or eastern British Columbia will flow through mountain passes creating cold east winds down the valley. In the winter season, storm winds will vary from 20 to 30 miles per hour (mph). During extreme events, winds will exceed 60 mph for short durations with 100 mph gusts occurring over mountain peaks.

2.4.5 Storms

Flood-producing storms occur chiefly during the winter season but are not uncommon in late fall or early spring. The sharp increase in frequency, duration, and severity of storms in late fall is a result of a southward displacement and renewed activity of the semi-permanent Aleutian low-pressure system. Frequently, a series of waves develop along the polar front. As the waves move landward, the unstable, moist air masses are orographically lifted by the mountains. This results in widespread, often heavy, precipitation that increases with elevation. Winter storms in the Pacific Northwest are typically of this basic type, having similar origins, air mass trajectories, and a moisture source in the Pacific Ocean. These storms sometimes follow in quick succession. On mountain slopes, storm precipitation is often heavy and continuous as a result of the combination of frontal and orographic affects. The November 1909, November 9-12, 1990, November 21-25, 1990, November 27-30, 1995 storm, and the October 16-21, 2003 storms are described below.

2.4.5.1. November 1909 Storm

November l909 was a month of above-average precipitation with a period of almost continuous moderate-to-heavy precipitation during the last 2 weeks of the month as a series of low-pressure systems moved across the Pacific Northwest. The fastest moving storm was the last one of the series which caused heavy rain on the 28th and 29th. During the 66-hour period beginning at 6 a.m. on the 27th and ending at midnight on the 29th, total storm precipitation amounts were 9.2 inches at Goat Lake, 8.3 inches at Skagit Powerplant, 5.9 inches at Concrete, and 2.5 inches at Sedro-Woolley. Maximum 24-hour amounts were 5.6, 5.8, 3.8 and l.3 inches, respectively, at these stations. The mean basin and maximum 24-hour precipitation for this storm period were 6.7 inches and 3.6 inches, respectively.

2.4.5.2. November 9-12 and 21-25, 1990 Storms

Precipitation amounts in Western Washington during the month of October were as much as 200 percent of normal. The snowpack was also 200 percent of normal and the snowline was at about 2000 feet mean sea level with an excess of 2 inches of water in the pack above 2,500 feet. The conditions, therefore, were primed to saturation in advance of the actual rainfall for the November 9-12 event. From November 9th through 12th, western Washington was dominated by a warm, moist subtropical air mass whose source region was an area just north of the Hawaiian Islands. During this entire period, the polar jet was vigorous, strong, and extraordinarily persistent. The core of the jet was generally oriented southwest to northeast and aimed at southern British Columbia and northern Washington. Maximum winds in the core of the jet were always in the excess of 100 knots and at times were in the 170-190 knot range.

Heavy and intense rains fell in western Washington during the 3-day period of November 8th through the 10th. Due to the strength and location of the core of the polar jet stream and the resulting wind structure at lower levels, the rains were highly orographic in nature. Heaviest rainfall centered in the Cascade Mountains from the Snoqualmie basin northward into Canada. The rainfall distribution can be seen in Table 4.

TABLE 4 – PRECIPITATION DURING THE NOVEMBER 8-11, 1990 STORM (INCHES)

River	Precipitation Station	November 8	November 9	November 10	November 11	Total
Sauk	Darrington	0.9	4.2	1.2	0.1	5.8
Skagit	Marblemount	0.9	6.1	2.5	0.1	9.6
Skagit	Diablo	4.0	7.3	1.0	0	12.3

Prior to the event, the freezing level was about 4,000 feet in western Washington but quickly jumped to 9,000-10,000 feet with the arrival of the tropical air mass. The freezing level stayed above 9,000 feet until November 13th and then dropped to about 3,000 feet late on November 14th. Warm air and rain falling on the snowpack melted an average of about 2 inches of water from the snowpack in the mountainous regions between 2,500 feet and 5,500 feet. Snowmelt, therefore, contributed significantly to the severity of flooding.

There was still substantial standing water left over from this first event in the basin when the second flood hit from November 21-26. A persistent low pressure system in the Gulf of Alaska generated a series of frontal systems that tracked across the Pacific Northwest from November 21st through the 26th. Normally there is a pool of heavy cold air that follows these frontal systems and forces them over the Cascades and into the Rocky Mountains. In this event, however, these frontal systems lacked sufficient cold air to drive them swiftly through the region. As a result, the systems were slow moving and stalled in the Cascades, allowing the orographic rains to continue much longer than normal. The cumulative rainfall for this event was greater than the first event but the first event had periods of much greater intensity. The rainfall distribution for this event can be seen in Table 5.

TABLE 5 – PRECIPITATION DURING THE NOVEMBER 21-25, 1990 EVENT

River	Precipitation Station	Nov. 21	Nov. 22	Nov. 23	Nov. 24	Nov. 25	Total
Sauk	Darrington	1.4	1.9	3.3	4.1	0.6	11.3
Skagit	Marblemount	1.0	2.5	1.0	2.2	0.3	6.0
Skagit	Diablo	2.8	3.5	5.8	3.2	0.2	15.5

Although the snowpack had built back up after the first event, the freezing level stayed quite low during the week of the event. Hence, although an average of 2 to 3 inches of water melted from the snowpack in the lower parts of the basins, the snowpack above 4,000 feet actually increased during the event. Snowmelt, therefore, did not contribute significantly to the severity of this event.

2.4.5.3. November 27-30, 1995 Storm

November 1995 was the wettest November on record at several locations in the Pacific Northwest. Flooding resulted from a combination of saturated ground, heavy rains, high freezing levels, and melting snow. Heavy rains that began on November 27 resulted from three storms that carried moisture laden, semi-tropical air into the Pacific Northwest. These storms were fed by a very strong polar jet stream that helped produce strong orographic precipitation on south and west facing slopes of the Olympic and Cascade Mountains. The heaviest rainfall from the first storm was in the central and northern Cascades, while the Olympics and southern Cascades felt the brunt of the last two systems. Four-day precipitation totals (November 27-30) at the NWS stations, Skagit River near Marblemount, and Sauk River near Darrington, were 7.5 inches and 5.7 inches, respectively. Inches of snow-water runoff during the November 1995 storm at Stevens Pass in the Skykomish River basin and at Corral Pass in the Green River basin, from snow pillow data, are listed in Table 6.

TABLE 6 - CHANGE IN SNOW-WATER EQUIVALENT FOR THE NOVEMBER 27-30, 1995 STORM

Date of Snow Observation	Stevens Pass Elev. 4070 ft	Corral Pass Elev. 6000 ft
Nov. 28	5.30 in	4.00 in
Nov. 29	4.70 in	4.00 in
Nov. 30	3.40 in	3.50 in
Dec. 1	4.40 in	4.00 in
Dec. 2	5.40 in	4.60 in

2.4.5.4 October 16-21, 2003 Storm

Prior to this event, northwest Washington experienced the driest summer on record and September precipitation about 50% of normal. As a result, soil conditions were relatively dry when the first storm made landfall on October 15th. The storm was made up of two events: the first between October 15th and 18th and the second one between October 19th and 23rd. Both storms were charged with tropical moisture that was transported into the area by the jet stream. These types of event have been typically called "pineapple express" events due to the long southwesterly moisture fetch. Being of tropical origin, the air contained very high concentrations of precipitable water (around 1.5 inches). The combination of high precipitable water and high speed jet stream results in very heavy precipitation on favorable slopes. Freezing levels were also very high, so precipitation during these events fell as rain at all elevations in the basins.

Measurements made at NRCS SNOTEL sites within the Skagit and Nooksack Basins on October 15 showed that 6 of the 9 stations had no snow and the remaining sites had only a few tenths of an inch of snow water equivalent. On October 20, prior to the onset of the heaviest rainfall, the snow water equivalent only increased by a few tenths of an inch. Low snow water equivalent is typical for this time of year. On October 21 after the heaviest precipitation, the snow water equivalent was relatively unchanged, indicating that snowmelt or rain-on-snow did not contribute toward the magnitude of the flood event.

Record 24-hour rainfall totals were recorded at Ross Dam (5.63 inches) and Upper Baker Dam (6.60 inches) on October 16th. Both records are noteworthy because each of these gages has a record length greater than 35 years. Other noteworthy 24-hour rainfall totals include 5.3 inches at Ross Dam on October 20th (second wettest 24-hour period of record), 6.8 inches at Darrington on October 20th (second wettest 24-hour period of record), and 6.82 inches at Diablo Dam (wettest 24-hour period of record in October). This suggests that the heavy rainfall during the first storm event on October 16th was sufficient to prime the basin for the flooding that resulted following the arrival of the second storm event on October 20th. This resulted in large instantaneous peak flows in the upper basin including a 124-year recurrence flow at the Sauk River at Sauk gage (119,000 cfs), a 72-year recurrence flow at the Thunder Creek near Newhalem gage (17,600 cfs), a 70-year recurrence flow for the inflow to Upper Baker Dam (37,000 cfs), and a 50-year recurrence flow for the inflow to Ross Dam (45,000 cfs). The regulated peak flow at Concrete of 166,000 cfs corresponds to roughly a 30-year event. The unregulated event is estimated to be roughly 206,000 cfs, which corresponds to roughly a 25-year event (see Section 6.0, Table 22).

While the maximum 24-hour rainfall totals associated with the 1990 and 1995 events were lower than the maximum 24-hour totals during the 2003 event, the rainfall amounts preceding these events were much greater than the rainfall amounts preceding the 2003 event. For example, the fall months of both 1990 and 1995 were quite wet with November 1990 (31.3 inches) and November 1995 (30.9 inches) being the wettest two months of record at the Upper Baker Dam gauge. Although the intensity of the short-duration rainfall associated with the 1990 and 1995 events was less than similar duration rainfall during the 2003 event, the consistently wet conditions preceding these events resulted in larger overall runoff volumes and hence longer duration peak flows, which results in a higher peak flow at Mt. Vernon relative to the 2003 event. There was also no snowmelt component to the 2003 event due to the lack of preceding precipitation and the earliness of the season, which helped to keep the flood volumes down. The volumes of water seen in the peak 3-day period for the 2003 event were not nearly as unusual as the instantaneous peak flows. These 3-day volumes for the Sauk River at Sauk gage, the inflow to Upper Baker Dam, and the inflow to Ross Dam have recurrences of 10-year, 25-year, and 14-year, respectively.

2.4.6 Channel Characteristics

2.4.6.1 International Border to Gorge Dam

The Skagit River from the United States-Canadian Border to Gorge Dam flows through the three Skagit River Plants (Ross, Diablo and Gorge) in a hydraulically-connected reservoir waterway.

2.4.6.2. Gorge Dam to Newhalem.

The 15,000-feet long reach from Gorge Dam to the Gorge Powerhouse is usually dry during normal hydropower operations. During flooding, however, local runoff generally fills the limited storage space in Gorge Lake prior to the flood peak, causing Gorge to spill into the normally dry channel between the dam and Gorge Powerhouse. When the channel is filled below Gorge, releases from Ross can be routed to Newhalem in a half hour or less provided the spill gates at Diablo and Gorge are opened when the release is made at Ross.

2.4.6.3 Newhalem to Concrete

The 39.6 miles long Skagit River reach from Newhalem to Concrete falls approximately 8 feet per mile. The upper half of the reach contains a steep rugged channel located between narrow rock canyon walls in many places. Most of the channel bed is composed of large irregular-shaped boulders, rocks, and cobbles. The River flows in a series of water drops and deep pools. The lower half of the reach is much more placid with a wider flatter channel with smaller rocks and gravel materials. Hydraulic travel time from Newhalem to Concrete is approximately eight hours at the higher range of flows that occur during flood conditions.

2.4.6.4 Concrete to Mount Vernon

The 38.4 mile long reach from Concrete to Mount Vernon falls approximately 150 feet (an average of about 3.9 feet per mile). River gradients range from 5.3 feet per mile near Concrete to l.5 foot per mile below Sedro-Woolley. Hydraulic velocities vary according to the location along the river, ranging from 5 feet per second to 10 feet per second. This reach is comparatively placid with a wide, gravel-lined channel with mostly small cobbles and gravels, soil embankments, and numerous side channels, oxbows and overbank erosion scars created during large floods of the past. Travel time through this reach varies with the rate of discharge, decreasing from 15-20 hours at low flow to between 10-15 hours at higher discharges. There is a wide range of hydraulic travel times between Concrete and Mount Vernon and the above values are occasionally exceeded.

2.4.6.5 Mount Vernon to Skagit Bay

From Mount Vernon, the Skagit River flows approximately 6 miles to the point at which it splits into the North and South Fork distributaries. The North and South Fork then each flow approximately 8 miles, west and south respectively, to discharge into Skagit Bay. During moderate (10-year return period) flood conditions, tidal influence is felt approximately 7 miles upstream from the bay on the North Fork and 5 miles on the South Fork. The river gradient from Mount Vernon to Skagit Bay is approximately 2 feet per mile. Upstream from the tidally-affected reach, hydraulic velocities range from about 3 feet per second to 9 feet per second, depending on location and discharge. The Skagit River downstream from Mount Vernon is fully confined by levees on both banks. The North and South Forks are similarly confined until they approach Skagit Bay. The channel bed material from Mount Vernon downstream is predominantly sand.

2.4.7 Streamflow Characteristics

The Skagit River basin is subject to rain and snowmelt runoff during the fall and winter, and snowmelt runoff during the spring. Spring snowmelt runoff is caused predominantly by melting of the winter snowpack and is characterized by a relatively slow rise and long duration. Some minor contribution to the rate and peak of the snowmelt is occasionally provided by warm spring rains, but the spring rain-on-snow impact is usually not significant. Highest mean monthly snowmelt discharges are usually reached in June. The resulting runoff occasionally inundates low areas adjacent to the river but rarely reaches the major damage stage. The maximum-recorded spring snowmelt discharge at Mount Vernon was 92,300 cfs in April of 1959.

Power reservoirs are normally refilled during the annual spring snowmelt runoff; and as a result, the spring peak discharges are generally reduced. The Skagit River and all of its major tributaries usually have low flows during August and September after the high-elevation snowpack has melted and the baseflow has partially receded.

With the advent of heavy precipitation in the fall and winter, the Skagit River experiences a significant flow increase. Floods and the highest daily and highest instantaneous peak discharge of the year usually occur during this period. Heavy rainfall and warm winds during typical 1-3 day winter storms causes streamflows to rise rapidly in a matter of hours to flood levels. Streamflows recede rapidly within hours after the storms have moved eastward through the region, although base flows and basin soil moistures usually remain high for several days. Several minor rises usually occur each winter, while major floods are more intermittent. Winter rain-type floods usually occur in November or December but may occur as early as October or as late as February.

The Skagit River, which receives the effect of the initial lifting of Pacific air over the Cascade Range, varies in seasonal streamflow throughout the basin, generally due to the basin’s heavy winter precipitation, spring snowmelt runoff, dry summers and topographical and elevation differences. The average annual runoff at the following stations reflects the runoff variation throughout the basin; Skagit River at the Newhalem streamgage, 50.8 inches; Sauk River Near Sauk streamgage, 82.4 inches; Baker River at Concrete streamgage, 121.1 inches; Skagit River near Concrete streamgage, 74.4 inches; and Skagit River near Mount Vernon, 72.7 inches. The 999 square mile watershed above Ross dam, located in the lee of western mountains that shield the basin from winter storms, has an annual runoff of only 45.6 inches. Average annual runoff at Ross and Upper Baker Dams is approximately 32 percent of the average annual runoff at Mount Vernon.

Maximum and minimum extremes in recorded annual runoff at Mount Vernon during the 1941-1999 period are 16,752,595 acre-feet in l991 and 7,608,893 acre-feet in l944 or 101.6 and 46.1 inches, respectively, for the 3,093 square mile basin.

2.4.8 Streamgage Stations

The locations of U.S. Geological Survey streamgaging stations in the Skagit River basin are shown on Figure 1 and a summary of both active and inactive gaging stations, along with their periods of record, is provided in Appendix B. A summary of streamflow data from selected long-term stations is provided in Table 7. Mean monthly streamflows for the Skagit River system are provided in Table 8.

TABLE 7 - SUMMARY OF STREAMFLOW DATA (CFS) 1/

STREAMGAGE	DRAIN. AREA MI^2	PERIOD OF RECORD	YEARS OF RECORD	AVERAGE ANNUAL DISCHARGE	MAXIMUM ANNUAL DISCHARGE	MINIMUM ANNUAL DISCHARGE	MAX. INST.	MIN. INST.
Skagit River at Newhalem	1,175	1909-14, 1921-2005	91	4,395	6,251	2,627	63,500	54
Sauk River near Sauk	714	1912, 1929-2005	78	4,332	6,048	2,662	106,000	572
Baker River below Anderson	210	1911-25, 1929-31, 1956-59	22	2,073	2,600	1,540	36,800	219
Baker River at Concrete	297	1911-15, 1944-2005	67	2,649	3,469	1,865	36,600	30
Skagit River near Concrete	2,737	1925-2005	81	15,010	21,270	9,512	166,000,	2,160
Skagit River near Mt. Vernon	3,093	1941-2005	65	16,560	23,140	10,500	152,000	2,740

1/ Data from USGS Water Resource Data through Water Year 2005. All years listed represent water years.

TABLE 8 - MEAN MONTHLY STREAMFLOWS (CFS)

STREAMGAGE	PERIOD	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
Skagit River at Newhalem	1909-14, 1921-2004	3,130	4,014	4,062	4,123	4,082	3,756	4,170	5,890	7,314	6,129	3,646	2,781
Sauk River near Sauk	1912, 1929-2004	2,867	4,479	4,624	4,163	3,789	3,256	3,957	6,468	7,894	5,611	2,791	2,091
Baker River at Concrete	1911-15, 1944-2004	2,490	3,353	2,883	2,737	2,485	2,101	1,974	2,774	3,716	3,274	2,116	1,823
Skagit River near Concrete	1925-2004	11,240	15,550	15,850	14,850	13,790	12,150	13,800	20,230	24,430	19,120	10,830	8,563
Skagit River near Mt. Vernon	1941-2004	12,420	18,100	18,610	17,650	16,720	14,320	15,070	20,360	24,570	20,130	11,730	9,469

Notes: 1/ Data from USGS Water Resource Data through Water Year 2004

2.4.9 Floods

Major floods on the Skagit River are the result of winter storms moving eastward across the basin with heavy precipitation and warm snow-melting temperatures. Several storms may occur in rapid succession, raising antecedent runoff conditions and filling various river storage areas. Frequently, a low-elevation snowpack forms over large parts of the basin. Heavy rainfall and warm snow-melting complete the flood producing sequence. Minor floods usually last about three days, rising to damage proportions in a day or less, reaching a flood crest in the next several hours, and receding rapidly in 24 hours or less. Floods of this variety have flood peaks less than 125,000 cfs below Concrete and are expected approximately every 10 years. Minor floods described above become major floods when the primary flood ingredient, intense storm rainfall, is extended for a longer period of time, or multiple storm systems occur in rapid succession. Several minor rises usually occur every year, but major floods occur with less regularity. However, two major floods have occurred in a single season, while several years have passed without a significant flood event. Winter rain-type floods usually occur in November or December but may occur as early as October or as late as February.

In l923, Mr. J. E. Stewart of the USGS collected data and reported on several very large historical floods in the Skagit River basin. Data collected and conclusions reached, together with information concerning floods of record through l957, are published in USGS Water Supply Paper l527. Mr. Stewart concluded that great floods occurred in l8l5 and l856 prior to the arrival of white settlers, and that the larger flood of l8l5 was probably as large as the greatest flood on the Skagit River within the last several hundred years. The published magnitudes of these floods, which are based on high water marks, have a high degree of uncertainty and have been classified by the USGS as “estimates”. There is also some concern that large woody debris jams that developed over decades, may have affected these high water marks. As a result of this high uncertainty, the floods of 1815 and 1856 are not considered in the analyses presented in this report.

Mr. Stewart also documented and estimated the magnitudes of a number of other large floods which occurred prior to the widespread establishment of stream gages within the basin. The most significant of these events were the large floods which occurred in water years 1898, 1910, 1918, and 1922. Estimates for the magnitudes of these floods were based on a variety of high water information, including both eyewitness reports of flood levels and natural indicators of high water levels, such as mud marks.

The estimated magnitudes of the historical floods of 1898, 1910, 1918 and 1922 have been the subject of considerable review, analysis and discussion, as described in Section 1.4. The analyses present in this report rely on peak discharge data for these floods as currently published by the USGS.

Between l920 and late 1950, prior to completion of present storage facilities at Ross and Upper Baker, incidental flood reduction was provided to varying degrees by storage operations at the initial power reservoirs. Regulation of 74,000 acre-feet and 120,000 acre-feet of flood control storage at Upper Baker and Ross since 1977 and 1953, respectively, have reduced all floods to some degree. Peak discharges for selected flood events, including the currently published peak discharges for the historical floods, are listed in Table 9.

Flood volume, channel storage, and Concrete to Mount Vernon local inflow have a marked effect on the routing and attenuation of flood peaks between Concrete and Mount Vernon. For example, during the two large floods in November 1990, the first flood peak attenuated between Concrete and Mount Vernon while the second flood increased in the same reach.

Skagit River flood peaks usually attenuate between Concrete and Mount Vernon. However, floods with high peaks and large volumes will generally fill the channel storage, and combined with runoff from the 356 square mile local area between Concrete and Mount Vernon, will cause the peak discharge to increase as it moves downstream.

During dry summer weather, soil moistures in the Skagit basin become substantially depleted. With the beginning of fall and winter rainfall, soil moistures are recharged; however, there is often a noticeable loss of runoff volume during the initial floods of the season until the various loss parameters are fully satisfied.

TABLE 9 - SUMMARY OF HISTORICAL FLOODS (CFS)
(Flows from USGS Records Except as Noted)

STATION	Skagit River near Concrete		Skagit River near Mt Vernon	
PERIOD OF RECORD	October 1924-Present		October 1940-Present	
	2,737 square miles		3,093 square miles	
	Peak Discharge		Peak Discharge	
Date	cfs	cfs / sq. mi.	cfs	cfs / sq. mi.
1815	510,000	186.3	--	--
1856	340,000	124.2	--	--
16 Nov 1896	--	--	--	--
18-19 Nov 1897	265,000	96.8	--	--
16 Nov 1906	--	--	180,000	58.2
18 Nov 1908	--	--	--	--
29-30 Nov 1909	245,000	89.5	--	--
21 Nov 1910	--	--	--	--
29-30 Dec 1917	210,000	76.7	--	--
12-13 Dec 1921	228,000	83.3	--	--
27 Feb 1932	147,000	53.7	--	--
13 Nov 1932	116,000	43.4	--	--
22 Dec 1933	101,000	36.9	--	--
25 Jan 1935	131,000	47.9	--	--
27 Nov 1949 1/	154,000	56.3	114,000	36.9
10 Feb 1951 1/	139,000	50.8	144,000	46.6
3 Nov 1955 2/	106,000	38.7	107,000	34.6
23 Nov 1959 2/3/	89,300	32.6	91,600	29.6
20 Nov 1962 2/3/	114,000	41.7	83,200	26.9
13 Jul 1972 2/3/	91,900	33.6	80,600	26.1
4 Dec 1975 2/3/	122,000	44.6	130,000	42.0
27, 28 Dec 1980 2/3/	148,700	54.3	114,000	36.9
9-12 Nov 1990 2/3/	148,800	54.4	142,000	45.9
22-26 Nov 1990 2/3/	146,000	53.3	152,000	49.1
28-30 Nov 1995 2/3/	160,000	58.5	141,000	45.6
17-21 Oct 2003 2/3/	166,000	60.7	129,000	41.7
6-7 Nov 2006 2/3/	145,000	53.0	125,000	40.4

1/ Ross Dam began storing water in March 1940.
2/ Includes effect of 120,000 acre-feet of flood storage established at Ross Dam in 1953
3/ Upper Baker Dam began storing water in July 1959 (74,000 acre-feet of flood storage at Upper Baker began in 1977)

2.4.9.1. Flood Runoff From Uncontrolled Watersheds

Runoff from the uncontrolled watersheds in the Skagit Basin has a major effect on flooding in the lower Skagit Valley. Flood control at Ross and Upper Baker is sufficient to control floods in the lower valley (with the lower valley defined as within the levee system from Burlington to the mouths) with exceedance frequencies of four to five percent (20-25 year event), but flood runoff from the Skagit's uncontrolled watersheds during events greater than approximately 4 percent (25-year event) exceedance frequency at Mount Vernon is sufficient to produce major flooding in the valley regardless of the flood control regulation at Ross and Upper Baker. The floods of November 1990 and November 1995 were 5 to 6 percent (16-20 year event) exceedance frequency events that raised the river to the tops of the main levees.

Flood control storage at Ross and Upper Baker is sufficient to store inflow while releasing only the minimum outflow for up to a two percent exceedance (50-year) event. The contribution from the uncontrolled watersheds for this event (50-year), however, is still large enough to deliver 175,000 cfs to the Mount Vernon area, which exceeds the current levee capacity. This will likely mean that the lower Skagit Valley will have flooded due to levee failures as a result of runoff from the uncontrolled watersheds. The magnitude of the uncontrolled watershed runoff is implied by the following runoff data for the river. Ross and Upper Baker reservoir watersheds are 39 percent of the total Skagit River drainage area at Mount Vernon (the remaining 61 percent of the total area is uncontrolled), and their combined annual runoff is 32 percent of the average annual runoff of the Skagit River at Mount Vernon. Uncontrolled runoff is 68 percent of the average annual runoff at Mount Vernon.

2.4.9.2. November 1949 Flood

The flood of November l949 is a good example of a flood crest flattening while moving downstream. Channel storage had a marked effect on the sharpness of the peak between Concrete and Mount Vernon. The peak discharge of 154,000 cfs at Concrete was reduced to 114,000 cfs at Mount Vernon. An absence of precipitation in the lower basin at the time of this flood partially explains the reduction in crest in the lower reaches of the channel. The Sedro-Woolley precipitation gage indicated that very little rain fell in the lower part of the basin.

2.4.9.3. February 1951 Flood

The February l95l flood had a peak discharge of l39,000 cfs at Concrete, a recorded peak of l50,000 cfs at Sedro Woolley, and a peak of l44,000 cfs at Mount Vernon. Reservoir storage reduced the peak discharge at Concrete about l3,000 cfs. However, due to the long duration of the peak discharge between Concrete and Mount Vernon, channel storage and attenuation had little effect on reducing the peak stage in the lower reaches. The flood remained near its peak for 6 hours at Mount Vernon. The duration of this peak was more significant than its magnitude because it minimized the effectiveness of natural storage in the Nookachamps Creek area, and dikes failed because they lacked sufficient cross-sectional dimensions to withstand a long period of high water.

2.4.9.4. November 1990 Floods

The 1990 floods broke through the Fir Island levee and inundated most of the interior farmland. Both events required extensive flood fighting in the vicinity of Mount Vernon. For example, during the November 1990 flood event, the peak discharge of 149,000 cfs at Concrete increased to 152,000 cfs at Mount Vernon, while the discharge of 160,000 cfs at Concrete during the November 1995 flood was reduced to 141,000 cfs at Mount Vernon. During the 1990 and 1995 floods, the stages at Mount Vernon were nearly equal, 37.34 feet and 37.37 feet, respectively. A major levee failure at Fir Island during the November 1990 flood increased the river slope and velocity below Mount Vernon, causing an artificially low crest stage at the Mount Vernon gage. The month of November 1990 included significant floods on November 9-11 (the first flood) and November 24-25 (the second flood). The first flood was slightly larger in volume than the second flood, but peak discharges were similar during both floods, having approximately a 5 percent exceedance frequency at the Concrete streamgage. Total flood storage used at both projects amounted to approximately 194,000 acre-feet during the first flood and approximately 153,900 acre-feet during the second flood. The above volumes include 112,000 acre-feet stored in Ross and 82,000 acre-feet stored in Upper Baker during the first and 100,000 acre-feet stored in Ross and 53,900 acre-feet stored in Upper Baker during the second flood. Inflow to both projects peaked on November 10, 1990 (first flood) as follows; 46,000 cfs at 2400 hours at Ross, and 33,000 cfs at 1000 hours at Upper Baker. Outflows at both projects were regulated to a minimum of 5,000 cfs through the main part of the flood.

A major levee break occurred during the first flood on the eastside of Fir Island, the major farming region between the North and South Forks of the Skagit River about 3 miles downstream from Mount Vernon. The failure occurred about 12-14 hours before the peak at Mount Vernon, inundating most of Fir Island with major damage consequences. The Fir Island levee failure caused the Skagit River to fall abruptly. Many requests were received by the Seattle District USACE Reservoir Control Center (RCC) from flood engineers at Mount Vernon to hold the stored floodwater and limit the rate of storage discharges to provide time for recession of the river’s uncontrolled streamflows. (The RCC is responsible for directing flood control operations at both Upper Baker and Ross Dams). The hydraulic relief provided by the Fir Island levee failure was probably instrumental in preventing failure of other major levees in the vicinity. Emergency repairs to the Fir Island levee were made between the first and second floods, but time was insufficient to fully stabilize the levee and the levee failed again during the second flood. Flood peaks between Concrete and Mount Vernon are normally reduced by attenuation and limited local inflow. This relation was reversed during the second flood due to significant local inflow, saturated soil conditions, and remaining pondage from the first flood.

2.4.9.5. November 1995 Flood

Flows on the Skagit River reached 160,000 cfs at Concrete and 141,000 cfs at Mount Vernon during the November 28-30, 1995 flood. Concrete was above zero damage stage for four days and above major damage (90,000 cfs) for one and a half days. Mount Vernon was above zero damage stage for approximately 4 days and above major damage for approximately 3 days. As a result of the reservoir regulation and sandbagging efforts, levees at Mount Vernon and Fir Island were able to withstand the flood without failing. Runoff stored at Ross and Upper Baker are estimated to have reduced flood levels by about 5 feet and 2 feet at Concrete and Mount Vernon, respectively.

RCC took control of Ross flood control storage at 0555 hour on the 28^{th} when the National Weather Service was forecasting a storm that would produce record-level flooding. Ross filled to an elevation of 1602.38 feet on November 30, using 118,623 acre-feet of the total active flood-control storage of 120,051 acre-feet. Ross inflow peaked at about 46,500 cfs at 1400 hours on November 29^{th}. Outflows from Ross were regulated to no more than 13,500 cfs until after the Skagit River near Concrete had peaked and receded to 90,000 cfs on the afternoon of the 30^{th}. Efforts to increase discharge from Ross and pass inflow were delayed nearly two days by the high inflow and the limitation on discharge of 26,000 cfs-28,000 cfs through the Project.

RCC took control of Upper Baker flood control storage on November 28^{th} at 1135 hours when the reservoir was at elevation 707.9 feet. Upper Baker Dam filled to an elevation of 719.1 feet on November 30, using 63,800 acre-feet of the 74,000 acre-feet of total flood-control storage at Upper Baker. Peak inflow into Upper Baker was 31,000 cfs.

This flood set a new crest-stage record at the Skagit River near Concrete gage despite the regulation at Ross and Upper Baker. The Concrete gage reached a crest of 41.57 feet. The Mount Vernon gage reached a crest of 37.34 feet, approximately equal to the record stage of 37.37 feet during the November 25, 1990 flood.

Reservoir inflow caused Ross Lake to fill to elevation 1602.38 feet, which is within 0.12 feet of the maximum full flood control pool. Upper Baker started to evacuate storage at 1800 hours on November 30, nearly a day after the river crested at Concrete. The flood storage evacuation was delayed until the flood recession at Concrete receded below 90,000 cfs in response to reports from the field flood engineers indicating that levees were still holding but a prolonged duration of high river flow was likely to cause failure. At Mount Vernon, the river was 0.5 feet above major damage stage for an extra half day, but the initial height was reduced due to this special evacuation.

2.4.9.6. October 2003 Floods

The floods of October 2003 started with a smaller peak followed by a larger peak. The first flood peaked at 94,700 cfs at Concrete and 73,500 cfs at Mount Vernon on October 17^{th} and 18th. This exceeded the major damage stage for 6 hours at Concrete but did not get above major damage at Mount Vernon. The second flood was significantly larger and spread more completely across the upper basin and peaked at 166,000 cfs at Concrete and 129,000 cfs at Mount Vernon on October 21st. Concrete was above zero damage stage for 57 hours and above major damage (90,000 cfs) for 33 hours. Mount Vernon was above zero damage stage for 64 hours and above major damage for 47 hours. As a result of the reservoir regulation and sandbagging efforts, levees at Mount Vernon and Fir Island were able to withstand the flood without failing.

This flood set a new crest-stage record at the Skagit River near Concrete gage despite the regulation at Ross and Upper Baker. The Concrete gage reached a crest of 42.21 feet, about 0.6 feet greater than the flood of November 1995. The Mount Vernon gage reached a crest of 36.2 feet, which is a foot lower than the peaks seen for the November 1995 and November 25, 1990 floods.

3.0 Hydrologic Study of the Skagit River Basin

This section summarizes the hydrologic analysis that has been completed for the Skagit Flood Risk Management Feasibility Study. Determining hydrology for the Upper Skagit River basin above Concrete (River Mile 54.1) is necessary to perform the hydraulic analysis of each of the proposed alternatives. The major flood damage centers are located from Sedro-Woolley (River Mile 22.4) downstream to the mouths of the North and South Forks.

3.1 Upper Skagit River Basin Above Concrete, WA to Ross Dam

The Upper Skagit River Basin has 1,214 square miles of drainage area behind dams that currently have reservoir storage space set aside for flood control and 1,523 square miles that is uncontrolled. The Upper Skagit River from Concrete, WA to Ross Dam has many tributaries flowing into it. Most of the large tributaries and drainage areas have a long record of stream gage information (see Appendix B). These gaged areas include the Baker River, Skagit River above Ross Dam, Cascade River, Sauk River, and Thunder Creek. Additionally, there are gages with long periods of record for the Skagit River at Newhalem and the Skagit River at Marblemount that provide information on the local flow in between these two areas.

3.2 Baker River

The Baker River, the second largest tributary in the basin, drains the north central portion of the Skagit Basin. The Baker River rises in rugged mountains in the upper Baker Basin and drains 298 square miles of watershed through a narrow rocky channel that flows about 30 miles to the right bank of the Skagit River at RM 56.5. The basin ranges in elevation from 170 to 10,775 feet with approximately two-thirds of the basin located below an elevation of 4,000 feet.

The Baker River Basin features several significant peaks including Mount Baker (10,775 feet), Mount Shuksan (9,127 feet), Mount Challenger (8,236 feet), Mount Blum (7,680 feet), Whatcom Peak (7,574 feet), and Bacon Peak (7,066 feet). Mount Baker is the second most heavily glaciated volcano in the Cascade Range to Mount Rainier with a volume of snow and ice of 0.43 cubic miles. The basin is mostly forested below 5,500 feet as the main land owners in the basin are the US Forest Service, North Cascades National Park, Washington State Department of Natural Resources, and Puget Sound Energy. Above 5,500 feet, only scrub vegetation exists with little or no vegetation on rock outcrops, glaciers, and permanent snowfields. The watershed is fairly steep with slopes from 20 to 40 percent over most of its area except in the vicinity of the channel and valley floor. Lake Shannon and Baker Lake occupy roughly 16 linear miles of the Baker River Valley. The average annual precipitation over the basin is roughly 130 inches.

The Baker River is regulated by two hydroelectric dams on the Baker River that are owned by Puget Sound Energy (PSE). These dams are named Upper and Lower Baker Dams. Upper Baker Dam is a concrete gravity structure that is 330 feet high and 1,230 feet long. The dam is located at River Mile 9.29 and was completed in 1959. At normal full pool elevation of 727.77 feet NAVD88, the reservoir extends 9 miles upstream and contains a surface area of 4,980 acres. There are 180,128 acre-feet of active storage between the normal full pool and the minimum power pool at an elevation of 677.77 ft NAVD88. A maximum of 4,650 cfs can be run through the turbines and the spillway can release up to 48,000 cfs at normal full pool and 60,000 cfs at the maximum design pool. When PSE first received its FERC license in 1956, a volume of 16,000 acre-feet was required to be set aside for flood control to make up for lost valley storage. In 1977, an additional 58,000 acre-feet of flood control storage was authorized by Section 209 of Public Law 87-874. The flood control operating policy requires that a minimum of 5,000 cfs be released from the project to maintain the necessary flood control space for large flood events.

Lower Baker Dam is a semi-gravity concrete arch structure 285 feet high and 530 feet long. It is located at river mile 1.2 and was completed in 1925. At normal full pool elevation of 442.35 feet NAVD88, the reservoir extends 7 miles upstream and contains a surface area of 2,278 acres. There are 116,770 acre-feet of active storage between the normal full pool and the minimum power pool at elevation 373.75 feet NAVD88. A maximum of 4,100 cfs can be run through the turbines and the spillway can release up to 40,000 cfs at normal full pool. There currently is no authorized flood control storage behind Lower Baker Dam. The current restriction during flood control operations is that Lower Baker Dam cannot draw down the reservoir while Upper Baker is storing water for flood risk management.

FERC issued PSE a new, 50-year operating license for the Baker River Hydroelectric Project in October 2008. The timing of flood control storage required at Upper Baker under the terms of the current license is shown in Table 10.

TABLE 10 - UPPER BAKER FLOOD CONTROL STORAGE REQUIREMENTS

	UPPER BAKER ELEVATION (NAVD 88)	**ACTIVE FLOOD STORAGE**
DATE	**FEET**	**acre-ft**
October 1	727.77	0
October 15	724.53	16,000
November 1	724.53	16,000
November 15	711.70	74,000
March 1	711.70	74,000
April 1	727.77	0

Under the terms of its new license, PSE is required to "develop means and operational methods to operate the Project reservoirs in a manner addressing imminent flood events". These methods may include "additional reservoir drawdown below the maximum established flood pool". Section 4.1.2 of the license Settlement Agreement further states that "PSE and Skagit County shall seek an agreement with the ACOE [i.e. USACE] to amend the ACOE Baker River Project Water Control Manual" to reflect a specified reservoir drawdown protocol when a flood event is imminent. It is anticipated that any operational changes to address "imminent floods" would take place after about 2012; the nature and impact of any such changes is not yet known, and are not considered in the hydrologic analyses in this report.

There are three locations on the Baker River where there is useful flow information for hydrologic analysis. Daily flows into the Upper Baker Dam area have been calculated since October 1926. Prior to Upper Baker Dam being built there was a gage (Baker River below Anderson Creek) at this site. Since construction of the dam, the daily flows can be calculated from the daily reservoir elevation and outflow information. The Baker River at Concrete gage has operated from 10/1/1910-2/28/1915 and 9/1/1943 to present. This has a mixed record of pre-dams and post-dams flows and can be influenced by the backwater of the Skagit River during large flood events so care has to be taken when utilizing this data.

There is also some limited local inflow data into Lower Baker Dam. Table 11 shows the runoff per square mile for Upper and Lower Baker inflows for the most recent major flood events for which there was full hourly data. The earlier October 2003 event was oriented more towards the Upper Basin than would be typical so it was not weighted as strongly when determining the factor to use as a ratio of Lower Baker to Upper Baker inflows. It is for this reason that the local inflow to Lower Baker dam is determined to be roughly 0.76 times the runoff per square mile as the Upper Baker inflow on average.

TABLE 11 – RATIO OF LOWER BAKER INFLOWS TO UPPER BAKER INFLOWS

Flood Event	Upper Baker Peak 24-hour Flow (cfs)	Upper Baker Runoff per Square Mile	Lower Baker Peak 24-hour Flow (cfs)	Lower Baker Runoff per Square Mile	Lower Baker to Upper Baker Runoff Ratio
11/10/1990	28255	131.4	8677	105.8	0.81
11/29/1995	24664	114.7	7315	89.2	0.78
10/17/2003	34540	160.7	5606	68.4	0.43
10/21/2003	28024	130.3	8590	104.8	0.80
12/24/2005	13161	61.2	3044	37.1	0.61
11/06/2006	28594	133.0	9188	112.0	0.84
Average					**0.71**
Average w/o 10/17					**0.77**

3.3 Sauk River

The Sauk River is the largest tributary of the Skagit River and flows into it on the left bank at River Mile 67.2. The Sauk River flows mostly north and is over 50 miles in length. It has a drainage area of 732 miles, which is over 25% of the total drainage area of the Skagit River at Concrete. This represents just over 50% of the uncontrolled drainage area in the basin. It is for this reason that the Sauk River is the largest contributor to the flooding that occurs on the Skagit River. Table 12 shows the Sauk's contribution in the last 3 major flood events on the Skagit River.

TABLE 12 - SAUK RIVER CONTRIBUTION TO SKAGIT RIVER FLOODING

Flood Event	Skagit River at Concrete Regulated Peak Flow (cfs)	Contribution from Sauk River at Sauk Flow (cfs)	Percent Contribution
11/10/1990	149,000	66,900	45%
11/29/1995	160,000	73,597	46%
10/21/2003	166,000	106,000	64%
11/06/2006	145,000	84,900	59%
100-year	214,000	111,000	52%

The elevations in the basin range from 210 feet to 10,541 feet. The Sauk River is designated a Wild and Scenic River. The rivers banks are mostly lined with grass and low brush and the overbank areas are mostly made up of forests. There are two large tributaries that flow into the Sauk from Glacier Peak. The largest is the Suiattle River (346 square mile drainage area), which flows in from the west at River Mile 13.2 and is over 40 miles in length. The White Chuck River (86.2 square mile drainage area) flows in from the west at River Mile 31.9.

There are two locations on the Sauk River that have useful flow information for this analysis. The Sauk River at Sauk gage has operated from 4/1/1911-7/31/1912 and 8/1/1928 to present. This gage is the most useful because it measures most of the drainage area (714 square miles) of the Sauk and has a long period of record. The Sauk River above Whitechuck River near Darrington has operated from 10/1/1917-9/30/1922 and 10/1/1928 to present. This gage provides the earliest hints of when the Sauk River might peak and shows the relative contribution from the upper basin.

3.4 Cascade River and Local Flow from Marblemount to Concrete

The Cascade River flows into the Skagit River at River Mile 78.1, just upstream of the town of Marblemount, and has a drainage area of 185 square miles. The Cascade River runs for 29 river miles north and east from South Cascade Glacier on Sentinel Peak to the Skagit River. The basin ranges in elevation from 185 to 8,300 feet. The Cascade River is classified as a Wild and Scenic River. It is mostly forested and the river opens from a canyon where the floodplain is roughly 400 feet wide at River Mile 3.3 to 2,800 feet at the mouth.

The local flow from Marblemount to Concrete covers the flows that enter the Skagit River from River Mile 78.7 to River Mile 54.1. The major creeks that flow into this area are Corkindale Creek, Rocky Creek, Illabot Creek, Bark Creek, and Jackman Creek. This reach has a local drainage area of 173 square miles.

There is one location on the Cascade River that has useful flow information for this analysis. The Cascade River at Marblemount gage operated from 10/1/1928-10/10/1979, and from 6/1/2006 to present. This gage is the most useful because it measures most of the drainage area (172 square miles) of the Cascade River and has a long period of record.

The local flows from Marblemount to Concrete can be calculated by subtracting gage data from the Skagit River at Marblemount, Sauk River at Sauk, and Baker River at Concrete from the Skagit River at Concrete gage but there are many potential sources of error with this approach. The main problem is that it is difficult to accurately time each flow for every event and the calculation sometimes results in negative flows. This may also be impacted by routing effects in this area as there is some storage available in the floodplain. The number of years that all of the gages are working simultaneously is limited, which limits the dataset that is available for use.

There are 9 years prior to October 1979 where there is enough data for all of the gages to allow for an estimate of local flow from Marblemount to Concrete when the Cascade River at Marblemount gage was active. The post-2006 data for the Cascade River at Marblemount was not available at the time the analysis described here was performed. Table 13 shows the comparison of the runoff per square mile of drainage area for the local flow and the Cascade River during the peak winter flow on the Skagit River at Concrete. This shows that the Cascade River is very similar in runoff per square mile of drainage area to the local flow. Although it appears that the Cascade River has slightly less runoff than the local flow, a look at the whole record shows that the Cascade River has slightly more runoff than the local flow. This discrepancy shows some of the inaccuracy of the local calculation. It is for these reasons that the local flow from Marblemount to Concrete is derived assuming that it has the same runoff per square mile of drainage area as the Cascade River.

TABLE 13 – COMPARISON OF RUNOFF PER SQUARE MILE OF DRAINAGE AREA BETWEEN MARBLEMOUNT TO CONCRETE (MMCC) LOCAL AND CASCADE RIVER

Year	Cascade River 1-day Peak Winter Flow (cfs)	MMCC Local Related 1-day Flow (cfs)	Cascade River Runoff Per Square Mile	MMCC Local Related Runoff Per Square Mile	Cascade to MMCC Local Ratio
1944	3210	5850	19	34	55%
1947	6640	8660	39	50	77%
1948	6280	7120	37	41	88%
1949	2340	2500	14	14	94%
1950	10200	11420	59	66	89%
1951	8870	14220	52	82	62%
1977	5860	4280	34	25	137%
1978	4420	5810	26	34	76%
1979	3700	3030	22	18	122%
Average	**5724**	**6988**	**33**	**40**	**82%**

3.5 Local Flow from Newhalem to Marblemount

There are 8 creeks that flow into the Skagit River between the stream gages at Newhalem and Marblemount. These drainages are Newhalem Creek, Goodell Creek, Thornton Creek, Damnation Creek, Alma Creek, Copper Creek, Bacon Creek, and Diobsud Creek. This local flow enters the Skagit River from River Mile 93.7 to River Mile 78.7 and has a drainage area of 206 square miles. These creeks run through steep, heavily forested basins to enter the Skagit.

This local flow can be determined by subtracting the Skagit River at Newhalem gage from the Skagit River at Marblemount gage. The Skagit River at Newhalem gage has flow data from 12/21/1908 to 5/31/1914 and 10/1/1920 to present. The Skagit River at Marblemount gage has flow data from 9/1/1943 to 7/7/1944, 10/1/1946 to 9/30/1951, and 5/20/1976 to present. The local flow can be determined, therefore, for 34 years of concurrent record.

3.6 Thunder Creek and Local Flow from Ross Dam to Newhalem

Thunder Creek flows into the Skagit River on the left bank at River Mile 102.2, just upstream of Diablo Dam. Thunder Creek runs north for 15 river miles from the glaciers of Mount Torment to the Skagit River and has a drainage area of 108 square miles. The basin ranges in elevation from 1,220 to 8,815 feet. The basin is heavily forested.

There is one location on Thunder Creek that has useful flow information for this analysis. The Thunder Creek near Newhalem gage has been in operation from 10/1/1930 to present. This gage is the most useful because it measures most of the drainage area (105 square miles) of Thunder Creek and has a long period of record.

The local flow from Ross Dam to Newhalem has a drainage area of 176 miles of which Thunder Creek represents 60%. Other creeks in this area include Horsetail Creek, Sourdough Creek, Stetattle Creek, Pyramid Creek, and Gorge Creek. The small sample of available data shown in Table 14 indicates that the local flow has roughly the same runoff per square mile as Thunder Creek, so the Thunder Creek gage is used to estimate this local flow.

TABLE 14 – RATIO OF ROSS DAM TO NEWHALEM LOCAL TO THUNDER CREEK

Flood Event	Thunder Creek Peak 24-hour Flow (cfs)	Thunder Creek Runoff per Square Mile	Ross Dam to Newhalem Local Peak 24-hour Flow (cfs)	Ross Dam to Newhalem Local Runoff per Square Mile	Ross Dam to Newhalem Local to Thunder Creek Runoff Ratio
11/29/1995	7872	75	13090	74	0.99
10/17/2003	6622	63	12901	73	1.16
10/21/2003	12667	121	17682	100	0.83
Average					**1.00**

3.7 Skagit River Above Ross Dam

Ross Dam is located at River Mile 105.2 on the Skagit River. Flows in this upper basin originate from Allison Pass in British Columbia and flow 57.1 river miles down to Ross Dam. The river crosses the U.S./Canada border at River Mile 127.0. The drainage area above Ross Dam is 999 square miles.

Ross Dam is a concrete arch dam that has a maximum height of 540 feet with a base width of 208 feet and a top width of 33 feet. The dam was built in 1949 and first had space available for flood control storage in 1954. At normal full pool elevation of 1,602.5 feet NGVD 47, the reservoir extends 23 miles upstream and contains a surface area of 11,700 acres. There are 1,434,796 acre-feet of active storage between the normal full pool and the lowest sluice outlet at an elevation of 1,265 feet. There are two sluice outlet systems, a high level sluice located near the center of the dam at an elevation of 1,340 feet and a low level sluice along the right abutment of the dam. The discharges of the high and low sluices at the normal full pool are 4,130 cfs and 4,400 cfs, respectively. There are two overflow spillway sections that are symmetrically located on either side of the dam. Each spillway section contains six bays at a spillway crest elevation of 1,582 feet with six radial gates of modified monocoque design. Each spillway gate is 20.5 feet high and 20 feet wide. The spillway capacity at normal full pool is 90,000 cfs and can reach 121,000 cfs at the top of the surcharge storage pool elevation of 1,608 feet. The

Seattle District prepared a plan requiring 200,000 acre-feet of flood control storage that was incorporated on 2/20/1950 with the understanding that further studies were needed to refine this number. Subsequent studies resulted in decreasing the flood control storage to 120,000 acre-feet. Eight hours before the natural flow on the Skagit River at Concrete is predicted to hit 90,000 cfs, outflows from the project can be reduced to 0. The timing of the flood control storage availability can be seen in Table 15.

TABLE 15 - ROSS FLOOD CONTROL STORAGE REQUIREMENTS

	ROSS LAKE ELEVATION	**ACTIVE FLOOD STORAGE**
DATE	**FEET(SCL Datum*)**	**acre-ft**
October 1	1,602.50	0
October 15	1,600.80	20,000
November 1	1,598.84	43,000
November 15	1,597.37	60,000
December 1	1,592.11	120,000
March 15	1,592.11	120,000

*SCL Datum is 1.79 ft above NGVD29

A gage existed at the dam site before the dam was built and daily pool elevations and outflows are available since Ross Dam has been in place. From this data, daily flow records are derived for the inflow to Ross Reservoir from 1/1/1919 to present.

4.0 Skagit River near Concrete Frequency Analysis

The hydrologic analysis hinges on flows developed for the Skagit River near Concrete. This location is the focal point for several reasons. There has been a stream gage (USGS gage #12149000) running continuously at this location since October 1924 and there are 4 additional significant historical peaks that have been determined for this location. The stream gage encompasses 88% of the total drainage area of the Skagit River (2,737 square miles). The stream gage is located upstream of any development that could influence the gage other than the dams upstream. It is also in a fairly confined area so there is less likely to be errors associated with the rating of the gage. This provides a firm foundation to determine the magnitude and recurrence of floods in the Skagit River Basin.

4.1 Developing a Consistent Record

In order to perform a frequency analysis correctly, the watershed conditions need to be consistent during the period of record. This is not the case for the Skagit River near Concrete gage because reservoirs have been added throughout the period of record (see Table 19), which have had varying effects on reducing floods in the upper basin. Developing a frequency curve that only included the current watershed condition with the current flood control storage would restrict us to only using the flow data from 1977 to present.

This period does not include the larger earlier floods that could greatly influence the upper part of the Concrete frequency curve. When developing low recurrence flood events (such as a 1% chance of recurrence (100-year event)), it is important to use as much data as possible including historical data unless there is evidence that this data is not indicative of the extended record.

The USGS has published peak discharges for 6 major historical floods (ungaged events). The peak discharges for these historical floods were determined by Stewart in the 1920's and published in 1961 with Bodhaine in USGS Water Supply Paper 1527. These data were revised downward slightly in Scientific Investigation Report 2007-5159 by Mark Mastin of the USGS in 2007. The data for the latest 4 historical floods (water years 1898, 1910, 1918, and 1922) from this report are used for this analysis. The following table summarizes the historical events for the Concrete gage.

TABLE 16 - HISTORICAL FLOODS FOR THE SKAGIT RIVER AT CONCRETE

Date of Historical Flood Event	USGS published Discharge at Concrete (cfs)
1815	510,000
1856	340,000
11/19/1897	265,000
11/30/1909	245,000
12/30/1917	210,000
12/13/1921	228,000

The latest four historical flood events (in water years 1898, 1910, 1918, 1922) are all documented as flooding events in early photographs and/or newspaper articles. The earliest historical flood events (1815, 1856) were also likely large events, but the magnitude of these floods is difficult to determine. The USGS has recently downgraded these flows to "estimates" due to the fact these estimates are based on single high water marks that were obtained long after these events occurred. There are also concerns that there could have been large debris jams in the past that accumulated over decades that could have created an artificial dam break flood. This would represent a changed watershed condition that would be hard to account for. Consequently, the 1815 and 1856 floods are not used in the unregulated frequency curve calculations.

4.1.1 Methodology Used to Estimate Unregulated Peak Annual Discharge from Regulated Discharges for the Skagit River Near Concrete

Although the period of record of streamflow data at the USGS gage 12194000 Skagit River near Concrete location dates to 1924, data collected at this gage reflect the effects of regulation at upstream reservoirs. For instance, by the late 1920's, construction of Gorge and Diablo dams on the Skagit River and Lower Baker dam on the Baker River had been completed. As such, use of the observed data from the Skagit River near Concrete gage to estimate unregulated discharge at this location involves adjusting these data for the effects of upstream regulation. See Figure 5 for location of dams.

The methodology used to account for the effects of regulation was largely dictated by data availability. For instance, the estimated unregulated discharge record was calculated using a daily time-step since this is the shortest time-step at which streamflow data are available over an appropriately long period of record.

The effects of regulation on the Skagit River discharge at Concrete were determined by calculating the effects of regulation from the five upstream hydroelectric power dams within the basin. The effects of regulation were determined independently for the three dams located on the mainstem Skagit River and for the two dams located within the Baker River sub-basin. The effects of regulation from these two sub-basins were then combined to produce an estimate of the overall impact of regulation on the Skagit River discharge at Concrete at a daily time-step. Adjustment of the regulated Skagit River streamflow record at Concrete using the time-series' of estimated effects of upstream regulation resulted in a synthetic time-series of unregulated Skagit River discharges at

Concrete. The following sections provide further details regarding how the regulated streamflow record at Concrete was adjusted to produce a synthetic record of unregulated discharge. For diagrams of these methods, see Appendix C.

4.1.1.1 Methodology Used to Estimate the Effects of Regulation from the Skagit Project

The Skagit Project consists of three dams owned by Seattle City Light located on the mainstem Skagit River – Ross, Diablo, and Gorge. Ross dam, which is the furthest upstream, impounds the largest reservoir and has the most significant impact to streamflow in the downstream reaches of the Skagit River. The drainage area contributing runoff to Ross reservoir is 999 square miles. Diablo and Gorge dams impound significantly smaller reservoirs and have a relatively smaller impact on streamflow.

The effects of regulation from these three dams were estimated by comparing the record of observed streamflow in the Skagit River downstream of these dams with a synthetic record of unregulated streamflow. Regulated streamflow downstream of these dams is best represented by data from USGS gage 12178000, which is located in the Skagit River at Newhalem and is several miles downstream of Gorge dam. The gage at Newhalem has a contributing drainage area of 1,175 square miles and has a continuous record dating back to 1920. A synthetic record of unregulated streamflow at this gaging location was estimated using a combination of a natural (unregulated) streamflow record for the Skagit River at the present location of Ross dam (999 mi^2 drainage area) and an estimated synthetic record of tributary inflow to the Skagit River between Ross dam and the Newhalem gaging site (tributary area of 176 mi^2). The record of natural streamflow in the Skagit River at the Ross dam site was obtained from Seattle City Light. Runoff from a significant portion of the tributary area between Ross dam and Newhalem is reflected in the streamflow record of Thunder Creek (USGS gage 12175500), which measures discharge from a 105 mi^2 area that is tributary to Diablo reservoir. Runoff from the remaining tributary area between Ross dam and Newhalem (71 mi^2) was estimated using data from the Thunder Creek gage and the estimated relationship between runoff in the Thunder Creek sub-basin relative to the 71 mi^2 area that is currently ungaged.

A review of USGS stream gaging stations was performed to locate suitable gaging records that could be used to estimate runoff from the 71 mi^2 drainage area between Thunder Creek and Newhalem. Long-term streamflow records from Stetattle Creek and Newhalem Creek appear to provide the most appropriate data. A 50-year streamflow record is available from Stetattle Creek (USGS station 12177500), which represents a 22 mi^2 drainage area (tributary to Gorge reservoir) located to the north of the Skagit River near the town of Diablo. The Stetattle Creek drainage is part of the 71 mi^2 tributary area to the Skagit River between Thunder Creek and Newhalem. Discharge in Stetattle Creek is considered representative of local inflows entering the Skagit River between Thunder Creek and Newhalem from similarly oriented tributary sub-basins. Mean annual runoff in the Stetattle Creek drainage is about 114 inches.

A 38-year record is available from Newhalem Creek (USGS 12178100), representing a 27.9 mi^2 drainage located to the south of the Skagit River near the town of Newhalem. Newhalem Creek enters the Skagit River just downstream of the USGS gage Skagit River at Newhalem (USGS 12178000) but should be reasonably representative of local inflows entering the Skagit River between Thunder Creek and Newhalem from similarly oriented tributary sub-basins. Mean annual runoff in the Newhalem Creek drainage is about 86 inches. Combined mean annual runoff from the Stetattle Creek and Newhalem Creek drainages, which is about 100 inches, should be representative of local runoff from the 71 mi^2 area between Thunder Creek and Newhalem (it appears as if the tributary area to the Skagit River between Thunder Creek and Newhalem is evenly split between drainages oriented similar to the Stetattle and Newhalem Creek sub-basins). It should be noted that an estimate of the mean annual runoff from this 71 mi^2 area based on the difference between observed discharge in the Skagit River at Newhalem, Thunder Creek, and Skagit River at Ross dam also yields 100 inches. By comparison, mean annual runoff in the Thunder Creek drainage is about 80 inches, or 20 percent less than runoff generated from the tributary area between Thunder Creek and Newhalem. Based on this comparison, the following relationship provides a reasonable estimate of tributary inflows to the Skagit River from the 71 mi^2 area between Thunder Creek and Newhalem:

Tributary inflows from the 71 mi^2 area = (71 mi^2/105 mi^2) * (100"/80") * Thunder Creek discharge;

Which yields: Tributary inflows from the 71mi^2 area = 0.85 * Thunder Creek discharge.

The following relationship was therefore used to create the synthetic record of unregulated mean daily discharge in the Skagit River at Newhalem (1,175 mi^2):

Mean daily natural discharge in the Skagit River at the Ross dam site (999 mi^2) + mean daily discharge in Thunder Creek (105 mi^2) + 0.85 * mean daily discharge in Thunder Creek (estimated runoff from 71 mi^2)

It should be noted that the values calculated using the above relationship were adjusted slightly to account for the approximate travel time in the natural (unregulated) Skagit River between Ross dam and Newhalem (estimated travel time of 2.3 hours). The resulting time-series is a synthetic representation of the mean daily unregulated discharge in the Skagit River at Newhalem for the period 1930 through 2007. The record begins in 1930 because this is the first year of operation of the Thunder Creek stream gage. Finally, the estimated effect of Skagit Project regulation on the mainstem Skagit River was calculated by taking the difference between the record of mean daily regulated discharge observed at Newhalem (USGS 12178000) and the synthetic record of mean daily unregulated discharge at this location. The effect of regulation on Skagit River discharge at Concrete was estimated by adjusting the time-series to account for an approximate eight-hour travel time from Newhalem to Concrete.

4.1.1.2 Methodology Used to Estimate the Effects of Regulation from the Baker River Project

The Baker River Project consists of two dams owned by Puget Sound Energy (PSE) located on the Baker River within the Baker River sub-basin. Upper Baker dam, which is the furthest upstream, impounds a larger reservoir and has a relatively greater influence on streamflow in the downstream reaches of the Skagit River relative to Lower Baker dam. The drainage area contributing runoff to Upper Baker reservoir (Baker Lake) is 215 mi^2 and the overall drainage area contributing runoff to Lower Baker reservoir (Lake Shannon) is 297 mi^2 (this figure includes the 215 mi^2 drainage to Upper Baker reservoir).

The effects of regulation from these two dams were estimated by comparing the record of observed streamflow in the Baker River downstream of both dams with a synthetic record of unregulated streamflow. A continuous record of regulated streamflow downstream of these dams is best represented by data from USGS gage 12193500, which is located in the Baker River less than one mile downstream of Lower Baker dam and just upstream of the confluence of the Baker and Skagit Rivers (a continuous record for this gage extends back to 1943). It is noted that data from this gage on occasion are affected by backwater from the Skagit River during high Skagit River flows. While PSE maintains a record of mean daily discharge from Lower Baker dam, these data are unfortunately not available over a continuous and suitably long-term record. Furthermore, a comparison of PSE's discharge data from Lower Baker dam with data from the USGS gage during several recent high flow events suggests that use of the USGS data to estimate the effects of Baker River regulation on Skagit River flows has a relatively small impact on the synthetic time-series of unregulated Skagit River flows. This is discussed in further detail in Section 4.1.1.3.

A synthetic record of unregulated streamflow at the Baker River at Concrete gaging site was estimated using a combination of a natural (unregulated) streamflow record for the Baker River at the present location of Upper Baker dam (215 mi^2) and an estimated synthetic record of tributary inflow to the Baker River between Upper Baker dam and USGS gage 12193500 (tributary area of 82 mi^2). The record of natural streamflow in the Baker River at the Upper Baker dam site was obtained from PSE.

A review of streamflow data from the Baker River near Concrete (USGS 12193500) shows a mean annual runoff of 122 inches from the Baker River basin for the period 1943 – 1999. The record of natural Baker River flows at the Upper Baker dam site for this period suggest a mean annual runoff upstream of Upper Baker dam of about 130 inches. Runoff from the 82 mi^2 area tributary to the Baker River downstream of Upper Baker dam can be estimated using the following relationship:

Runoff from 82 mi^2 area = [(122"*297 mi^2)-(130"*215 mi^2)]/82 mi^2 = 101 "/year

Based on this relationship, mean daily discharge from the 82 mi^2 tributary area downstream of Upper Baker dam can be estimated from natural discharge in the Baker River at the Upper Baker dam site as follows:

Inflows from 82 mi^2 area = (82 mi^2/215 mi^2) * (101”/130”) * natural discharge in the Baker River at the Upper Baker dam site;

Which yields: Inflows from 82 mi^2 area = 0.30 * natural discharge in the Baker River at the Upper Baker dam site.

The following relationship was therefore used to create the synthetic record of unregulated mean daily discharge in the Baker River at Concrete (297 mi^2):

Mean daily natural discharge in the Baker River at the Upper Baker dam site (215 mi^2) + 0.30 * mean daily natural discharge in the Baker River at the Upper Baker dam site (estimated runoff from 82 mi^2).

It should be noted that the values calculated using the above relationship were adjusted slightly to account for the approximate travel time in the natural (unregulated) Baker River between Upper Baker dam and Concrete (estimated travel time of 1.7 hours). The resulting time-series is a synthetic representation of the mean daily unregulated discharge in the Baker River at Concrete for the period 1926 through 2007. The record begins in 1926 because this is the first year of record of natural streamflow in the Baker River at the Upper Baker dam site. Finally, the estimated effect of regulation from the Baker River Project on the Baker River was calculated by taking the difference between the record of mean daily regulated discharge observed at Concrete (USGS 12193500) and the synthetic record of mean daily unregulated discharge at this location. The effect of regulation on Skagit River discharge at Concrete was estimated by adjusting the time-series to account for an approximate one-half hour travel time between the Baker River gage near Concrete and the Skagit River gage near Concrete.

4.1.1.3 Estimated Unregulated Peak Annual 1-day Discharges in the Skagit River at Concrete

A synthetic record of the mean daily unregulated discharge in the Skagit River at the Concrete gaging site was constructed by adjusting the observed record of mean daily Skagit River discharge (USGS 12194000) using the time-series of estimated mean daily regulation effects for the Baker River and Skagit hydroelectric projects. The resulting time-series has a record from 1925 through 2007. A synthetic record of peak annual mean daily unregulated discharge in the Skagit River at Concrete was constructed by selecting the peak annual discharges from the time-series of mean daily unregulated discharge.

As noted previously, estimates of the effects of regulation from the Baker River Project were made using Baker River discharge data collected at the USGS gage at Concrete. These data are occasionally affected by backwater from the Skagit River during high Skagit River flows. As such, Baker River discharge reported at the USGS gage may be artificially high during these periods. Use of the USGS data to estimate the effects of Baker River regulation in these circumstances may result in an underestimate of the benefits of flood control at the Baker River Project, which would therefore result in an underestimate of the unregulated discharge in the Skagit River at Concrete. The potential effect of this on the synthetic record of unregulated Skagit River peak flows was investigated using the three highest Skagit River flow events at Concrete since 1925 (November 1990, November 1995, and October 2003). These three events were selected because discharge records of the Baker River at Concrete are available from both the USGS and PSE (PSE's record reflects discharge from Lower Baker dam). Note that for these events only, the estimated unregulated discharge in the Skagit River at Concrete was determined using Lower Baker dam discharge data obtained from PSE (Baker River USGS data were not used to estimate unregulated Skagit River discharge for these three events). Use of the USGS data to estimate the peak mean daily unregulated discharge in the Skagit River at Concrete during these events would have resulted in peak discharges that are roughly 2 percent lower in 1990, 3 percent lower in 1995, and 4.5 percent lower in 2003 relative to the values computed using PSE's Lower Baker dam discharge data. However, it should be noted that these three events represent the largest mean daily Skagit River peaks at Concrete since 1921. Most of the annual Skagit River peaks at Concrete are much lower than these three peaks and as a result the backwater impacts to the Baker River gage at Concrete are expected to be relatively lower and in many cases negligible. As such, use of the Baker River USGS data is expected to have a relatively small impact to the estimated annual unregulated Skagit River peaks at Concrete.

4.1.2 Determining the Relationship between Historical 1-day Flows and Historical Peak Flows

The historical data contains only instantaneous peak flows so a relationship between peak and 1-day flows is needed to convert this data to 1-day data. Without a similarly sized unregulated basin to draw from, an estimate needs to be made from the existing data. A comparison was made between unregulated 1-day flows and the regulated 1-day flows to determine which floods were minimally affected by regulation. This filtering of the floods was done to identify those floods where the unregulated and regulated 1-day flows were within 5% of each other (there were 18 winter floods that met this criteria). It was then assumed that the observed peak and 1-day flows for those events were representative of unregulated conditions. In addition, there is enough data for the November 1990, November 1995, October 2003, and November 2006 floods to determine the unregulated hourly data for the entire duration of these storms, so peak and 1-day unregulated flows can be derived for these events. Regression of peak against one-day flow using all of these data results in a peak to 1-day relationship for unregulated flows with a correlation coefficient (R^2) of 0.98.

4.2 Winter Flood Frequency Curve

Floods in the Skagit Basin can be classified as either spring snowmelt, or winter or late fall rainfall or rain-on-snow events. For the majority of time, the unregulated peak flow at Concrete recorded in any water year will occur within the time period of October through March. These winter (or late fall) floods are driven primarily by heavy rainfall. Snowmelt may or may be a significant contributor to flood magnitude or volume and is not a necessity for a winter flood. However, winter events have the potential to produce the highest peak flows and volumes when significant low elevation snowfall is present, followed by rising freezing levels, rain, and wind. The hydrograph produced by a winter flood event shows relatively quick rising and falling limbs compared to the broader, higher volume spring runoff hydrograph. It is very unusual to observe a regulated spring snowmelt peak flow at Concrete that exceeds 90,000 cfs (major damage level). Hydropower reservoirs are refilling during the spring runoff, and usually decrease the spring peaks. All observed floods that have caused significant damage have been winter rainfall or rain-on-snow flood events. The winter type flood events comprise the majority of annual flood flows, and define the upper end (high return interval portion) of the frequency curves. It is for these reasons that a winter frequency curve is used to define the flood flow frequency for the Skagit Flood Risk Management Study.

The program HEC-FFA was used to perform the flood frequency analysis. This program computes flood frequencies in accordance with the publication titled "Guidelines for Determining Flood Flow Frequencies, Bulletin 17B of the US Water Resources Council". The flood frequency is determined by fitting a Log-Pearson Type III distribution. A generalized skew of 0 is used for the analysis of the peak events, –0.04 is used for the 1-day, and –0.12 is used for the 3-day analysis. The adopted skew used by the program is close to the actual skew of the data due to the long length of records at this site.

The results of flow frequency analyses presented in this report are for computed frequency estimates. An expected probability adjustment, normally applied in accordance with Corps' guidelines contained in EM1110-2-1415 (Engineering and Design – Hydrologic Frequency Analysis), is not appropriate in this instance since a risk-based approach to analysis and design has been adopted per EM1110-2-1619 (Risk-Based Analysis for Flood Damage Reduction Studies).

Frequency curves for unregulated and regulated flows are provided in Appendix D.

4.3 Hypothetical Unregulated Hydrographs for Skagit River near Concrete

Unregulated hypothetical flood hydrographs for the 2-, 5-, 10-, 25-, 50-, 75-, 100-, 250-, and 500-year events were developed for the Skagit River near Concrete using statistical frequency peak and volume analyses. The hydrograph shapes were roughly based on the October 2003 event. The hydrographs were then balanced to match the necessary 1-day and 3-day volumes. That is, the area of the hydrograph defined by the 100-year peak and 1-day value was shaped so that the 24 hourly discharge values summed and averaged are equal to the 100-year 1-day discharge. The same was applied to the flood hydrographs defined by the peak, 1-day and 3-day values. These hydrographs can be seen in Appendix E.

4.4 Regulated Frequency Curve at Concrete

A consistent frequency curve is now developed for the Skagit River near Concrete gage but does not represent the existing condition. This requires developing a regulated frequency curve at Concrete that reflects the influence of flood storage and hydropower operations at Seattle City Light and Puget Sound Energy Reservoirs. There are several steps necessary to develop the existing condition regulated frequency curve at the Skagit River near Concrete gage. These steps include using the data that we have available that reflect the existing flood control operation and then converting the rest of the data set to reflect what the flows would have been if the existing flood control had been available.

4.4.1 Data Available with Existing Flood Control Operation

The existing flood control operation for the upper basin is that up to 74,000 acre-feet at Upper Baker Dam and up to 120,000 acre-feet at Ross Dam are available for flood control storage. The seasonal variation in flood control storage is shown in Tables 10 and 15 for Upper Baker Dam and Ross Dam, respectively. This storage at Ross Dam has been available since 1954. For Upper Baker Dam, 16,000 acre-feet has been available since 1956 and the additional 58,000 acre-feet has been made available since 1977. Even though the current flood storage requirements were not fully implemented until 1977, a closer examination of the record from 1956-77 shows that there were only two floods in that period that significantly exceeded the 90,000 major damage threshold. This study assumed that all regulated peaks from water year 1956 to present essentially show the effects of current flood control requirements. The 1-day, 3-day, and other regulated flow durations at Concrete may have changed due to changing storage requirements, but is unlikely that regulated peak flows from water year 1956 to 1976 would have changed significantly with the present flood storage conditions. The regulated median plotting positions for the 1956 to present data is used to develop the lower magnitude and more frequent events (i.e. the 2- and 5-year flood events).

4.4.2 Development of Regulated Lower Frequency Events

To develop the lower frequency events, unregulated flows for the 10-, 25-, 50-, 75-, 100-, 250-, and 500-year flood events for the Skagit River near Concrete need to be converted to flows that are regulated with the existing flood control requirements. This requires relating the unregulated Concrete flows to each of the upper basin flows, regulating the flows through Ross and Upper Baker Dams, and routing these flows back down to Concrete.

4.4.2.1 Unregulated Skagit River near Concrete to Upper Basin Flow Regressions

To relate the upper basin flows to the unregulated Skagit River near Concrete flows, regressions are developed that relate the observed upper basin gage's 1-day flow to the corresponding unregulated Skagit River near Concrete peak 1-day winter event for the concurrent period of record. These upper basin flows include Upper Baker and Ross Dam inflows, Newhalem to Marblemount Local, Thunder Creek, and Cascade and Sauk Rivers (see Appendix F). The remaining upper basin flows are derived from these as is detailed in Section 3.

The 1-day time period is the duration which has the greatest influence on flood peaks both upstream and downstream. This is because there is storage in the floodplain that can attenuate peak flows as they move downstream so flooding is more related to the volume of flows moving through the system. Instantaneous peaks are also more difficult to determine for the inflows to Upper Baker and Ross Dams. Peak and 3-day volumes for each of the upper basins are derived from their peak to 1-day and 3-day to 1-day regressions for winter floods. (See Appendix F for all regressions).

4.4.2.2 Development of Hypothetical Hydrographs for Upper Basins

The regressions provide 1-day peak flows for each of the upper basins. Regressions are then developed for each of the upper basins to relate their winter peak 1-day flows to their coincident instantaneous peak and 3-day flows (see Appendix F). The upper basin hypothetical hydrographs are then shaped to match these peak, 1-day, and 3-day flows using the October 2003 upper basin hydrograph shapes as a guide. The timing for when each of the upper basin tributaries peaked is determined by evaluating this relationship for past events. Table 17 shows the timing for each of the tributaries.

TABLE 17 – TRIBUTARY TIME OF PEAK IN HOURS BEFORE SKAGIT RIVER NEAR CONCRETE PEAKS

	Ross Inflow	Thunder Creek	Ross to Newhalem Local	Newhalem to Marblemount Local	Marblemount to Concrete Local	Sauk River at Sauk	Upper Baker Inflow	Lower Baker Inflow
11/10/90	4		4	7	10	2	6	22
11/24/90	15			19			21	
11/29/95	3	7	-2	8	23	4	10	7
12/13/98	5	6	8	8	15	11		
11/12/99	-3			2	-2	14		
01/08/02	1	1	-4	2	-1	10		
01/26/03	-5		-5	1	8	6		
10/17/03	4.25	5.5	13.25	17	25.75	3.25	11.25	12.25
10/21/03	4.25	7.25	5.25	10.75	14.25	4	11.25	13.25
12/11/04	1	5	8	8	23	0	7	9
12/24/05	-11	3		6			8	
11/06/06	5		6	10		6	15	15
Average of All Events	2.0	5.0	3.7	8.2	12.9	6.0	11.2	13.1
Average of Large Events *	5.9	6.6	5.3	12.0	18.3	3.9	12.4	13.9
Timing Used	4.0	7.0	5.0	12.0	15.0	4.0	11.0	13.0

* Large events are the WY 1991, 1996, 2004, 2007 events.

To ensure that these upper basin flows are correct, the upper basin flows are routed without flood control regulation through a HEC-RAS unsteady flow model (see the Hydraulic Technical Documentation for more information) down to the Skagit River near Concrete for each of the events. These routed flow volumes are then compared with the corresponding unregulated flows that were derived for Concrete in Section 4.2. The upper basin flows are then scaled as necessary to match the unregulated flows at Concrete as closely as reasonably possible. Particular emphasis was given to matching the one-day unregulated flows at Concrete. Due to the complexity of the system, and the desire to maintain nested upper basin flow hydrographs over the full range of events, an exact match to the Section 4.2 unregulated flows was generally not possible. Differences between routed flows and unregulated flows from frequency analysis ranged from: +0.6% to -5.2% for peak flows; +0.4% to +3.4% for one-day volumes; and -6.2% to +8.6% for three-day volumes. The one-day scaled flows are listed in Table 18 below.

The HEC-RAS hydraulic model extends 0.5 miles upstream of Marblemount on the Skagit River and 0.5 miles on the Baker River above its confluence with the Skagit. For the purpose of flow inputs to the HEC-RAS model, for modeling of unregulated conditions, the Ross Dam Inflow, Thunder Creek, and local inflows above Marblemount are lumped into a single input hydrograph. Similarly, on the Baker River, the Upper Baker Dam and Lower Baker Dam inflows are lumped into a single input hydrograph.

TABLE 18 – SCALED UPPER BASIN 1-DAY COINCIDENT FLOWS (IN CFS) DERIVED FROM REGRESSION WITH UNREGULATED SKAGIT RIVER NEAR CONCRETE 1-DAY COMPUTED PEAK FLOWS

Location	2-year	5-year	10-year	25-year	50-year	75-year	100-year	250-year	500-year
Unregulated Skagit River near Concrete	68000	105000	134000	174000	207000	227000	242000	294000	336000
Ross Dam Inflow	9990	20340	26440	35320	41560	46090	49590	60710	69250
Thunder Creek	2100	4340	5620	7330	8690	9640	10300	12600	14390
Ross Dam to Newhalem Local w/o Thunder Ck	1880	3640	4760	6170	7340	8140	8770	10630	12160
Newhalem to Marblemount Local	10060	14910	20390	26750	31570	35050	37220	45630	51680
Cascade River at Marblemount	4920	7320	9570	11910	14530	15830	16990	20170	23350
Marblemount to Rockport Local	2960	4590	5770	7150	8750	9530	10230	12140	14070
Rockport to Concrete Local	2040	3070	3980	4930	6030	6570	7050	8370	9700
Sauk River at Sauk	22630	36040	49390	59900	71400	79670	85790	102200	115900
Upper Baker Dam Inflow	11620	16160	20410	27240	29790	32320	34390	40550	46420
Lower Baker Dam Inflow	3440	5070	6050	7960	8830	9580	10190	12010	13760

4.4.2.3 Determining Low Frequency Regulated Peak Flows for Skagit River near Concrete

To determine the regulated flows for Skagit River near Concrete, the existing flood control regulation is used to alter the upper basin flows. The inflows to Upper Baker and Ross Dams are routed using the existing flood control authority, to come up with regulated outflows at these two dams. Local flows with routing are added to the outflow from Ross Dam and Upper Baker Dam to determine the corresponding flows for the Skagit River at Marblemount and Baker River at Concrete gages. These flows are the upstream inputs to the upstream hydraulic model (see Hydraulic Technical Documentation). These flows are then routed with the necessary local flows to Skagit River near Concrete to produce the regulated hydrograph for that event. This is run for the 10-, 25-, 50-, 75-, 100-, 250-, and 500-year events. Further details of the analysis, including the technique for accounting for seasonal variation in flood control storage, are provided in Section 4.4.2.4.

4.4.2.4 Detail of Methods to Model Existing Flood Control Regulation

Dam construction in the Skagit basin began in 1924 with the Low Gorge dam. Additional dam construction continued until 1961 with the completion of High Gorge Dam. All dams were designed and built as hydropower generation structures. As the magnitude of Skagit Basin flooding problems became more evident, flood control storage was later required in Ross and Upper Baker Reservoirs. No flood control storage is currently required in Diablo, Gorge, or Lower Baker Reservoirs. The following table is a synopsis of dam construction and important flood control storage requirements in the Skagit Basin.

TABLE 19 - SYNOPSIS OF DAM CONSTRUCTION AND FLOOD CONTROL EVENTS

Year	Significant Construction or Flood Control Event
1924	Low Gorge Dam completed
1925	Lower Baker Dam completed
1929	Diablo Dam completed
1940	Ross Dam 1st step construction completed
1946	Ross Dam 2nd step construction completed
1949	Ross Dam 3rd step construction completed
1950	2nd Gorge Dam completed
1954	120,000 acre-ft of flood storage required in Ross Reservoir by FERC license
1956	16,000 acre-ft flood storage required in Upper Baker Reservoir by FERC license
1959	Upper Baker Dam Completed
1961	High Gorge Dam completed
1977	An additional 58,000 acre-ft flood storage in Upper Baker Reservoir authorized by Congress

4.4.2.4.1 Reservoir Flood Operation

Flood control regulation at Ross is coordinated with flood control storage regulation at Puget Sound Energy's (PSE) Upper Baker plant. Ross is located approximately 40 miles and an 8-10 hours hydraulic travel time upstream from Concrete, and Upper Baker is located 9.3 miles and 1-3 hours hydraulic travel time upstream from Concrete. The Seattle District of the Army Corps of Engineers' Reservoir Control Center (RCC) regulates both projects concurrently to coordinate their regulated discharges and optimize their combined flood control storage. There is no authorized flood control storage at Diablo, Gorge, or Lower Baker Dams. During flood control events, the RCC, SCL, and PSE must monitor the operation of Diablo, Gorge, and Lower Baker to assure that (1) regulated discharges from Ross and Upper Baker are routed through the lower dams as expeditiously as possible, (2) adequate gate operation staff are available for necessary gate operations at all plants, and (3) no drafting of the three lower plants (Diablo, Gorge, or Lower Baker) will occur without first coordinating with the RCC. This third provision means that these lower 3 dams cannot release more than the outflows seen at the larger upstream dam plus the instantaneous local inflow coming into the project from local tributaries flowing into the dams between the upper dam and the lower dam.

Some pertinent information regarding the system and the regulation analysis include:

- Travel time between Ross and Concrete is considered to be nine hours
- Travel time between Upper Baker and Concrete is considered to be 1.5 hours
- Maximum outlet capacity at Lower Baker Dam is 41,000 cfs. If inflows exceed this value with a full pool the project would be overtopped.
- The ideal maximum flow at Newhalem, downstream of Gorge Dam, is 30,000 cfs.
- The ideal maximum release from Ross Dam is 25,000 cfs.
- Minimum outflow at Upper Baker is 5000 cfs.
- Minimum outflow at Ross is generally 5000 cfs but can be 0 cfs.

The “ideal” maximum flow at Newhalem and “ideal” maximum release from Ross Dam are flows above which damage may start to be experienced. Attempts are made to not exceed these “ideal” maximum flows, but they are not constraints on project operations.

4.4.2.4.2 Flood Regulation

The Water Control Manual (WCM) for each project has specific guidelines as to how each project is to be regulated during a flood. The WCM states that eight hours before the Northwest River Forecast Center forecasts the natural (unregulated) flow at Concrete to be 90,000 cfs, flow out of both Ross and Upper Baker will be set to their respective minimums. Typically, in an effort to preserve storage at Upper Baker, inflows would be passed until about two hours before the natural flow at Concrete is forecast to reach 90,000 cfs. These minimum outflows will be maintained until such time that the regulated flow at Concrete peaks or higher outflows are required by the Special Gate Regulation Schedule (SGRS). When the regulated flow at Concrete has peaked, Upper Baker can be ramped up to evacuate storage and Ross should be ramped up to pass inflow. This ramp up should not increase the flow at Concrete to a level greater than that at which it has already peaked. Care is needed when evacuating Upper Baker to ensure that the increased outflow from Ross does not push Concrete back above its peak or cause a secondary peak. When the flow at Concrete recedes to 90,000 cfs, evacuation of Ross can commence.

4.4.2.4.3 Flood Regulation Simulations

Reservoir regulation simulations were performed to estimate releases from Ross and Upper Baker for the 5-, 10-, 25-, 50-, 75-, 100-, 250-, and 500-year inflow events. The 2-year event was not regulated since it does not reach the 90,000 cfs flow on the Skagit River near Concrete which triggers flood control regulation. Estimation of the inflow hydrographs for these events is described in Sections 4.4.2.1 and 4.4.2.2 of this document.

Simulations were performed using an Excel spreadsheet constructed to route flows through the Ross and Upper Baker reservoirs at an hourly time step according to the flood control regulations described in the project Water Control Manuals. Each of the eight flood events from the 5-year event to the 500-year event was regulated using the spreadsheet model based on an "average case" or "most likely" regulation scheme as follows:

> Upper Baker outflow is reduced to a minimum of 5,000 cfs about three hours before the estimated natural flow at Concrete reaches 90,000 cfs. At Upper Baker, for large events, or events early in the flood control season, where outflow is dictated by the Spillway Gate Regulation Schedule, the Spillway Gate Regulation Schedule is followed until the flow at Concrete peaks. Inflows are then passed for about three to four hours after the Concrete peak has passed, and then only increased by an amount that does not increase the Concrete flow beyond that which occurred three hours after the Concrete peak. When possible, the 5,000 cfs minimum outflow is held for three to four hours after the Concrete peak. Where possible, consideration is given to keeping outflow to a level that allows Lower Baker to operate within its 41,000 cfs outlet capacity or as close to it as is deemed reasonable. Ross outflow is reduced to a minimum of 5,000 cfs eight hours before the estimated Concrete natural flow reaches 90,000 cfs and not ramped up to pass inflow until three to four hours after Concrete has peaked. In addition, the ideal maximum flow of 30,000 cfs at Newhalem is considered, and a reasonable attempt is made not to exceed this flow, or at least limit the amount/duration by which a flow of 30,000 cfs is exceeded.

Some variation from the "average" regulation scheme would be expected, particularly with regard to evacuation of flood control storage in situations where another significant flood is forecast.

A key consideration in the simulation of flood control regulation is the pool elevation (or, equivalently, amount of storage available) at the start of the simulation. The seasonal variation of flood control storage required at Upper Baker and Ross reservoirs is shown in Tables 10 and 15 respectively. The full amount of flood control storage is not required at Upper Baker until November 15 and at Ross until December 1. Large floods have, however, occurred early in the flood control season before the full amount of flood control storage is required under current operating policies. The most recent early season floods include the October 2003 floods described in Section 2.4.9.6, and the flood of November 6-7, 2006

Analyses were conducted of the impact of seasonal variation in flood control storage on regulated flood flows on the Skagit River near Concrete (USGS gage 12194000). The analyses (described in Appendix G) examined the flood control performance of Upper Baker and Ross reservoirs, with seasonally varying flood control storage, at two-week intervals from the start of the flood control season on October 1 through December 1, when the full amount of flood control storage is available at both Upper Baker and Ross. The impact of the seasonal variation of flood storage on regulated flows for the 5-

through 500-year events was then determined by weighting the regulated flow hydrographs for the Skagit River near Concrete on the basis of the historical frequency of occurrence of annual maximum winter flows within each two-week window through the flood control season. The analysis described in Appendix G concluded that allowance for the seasonal variation of flood control storage through use of weighted event hydrographs would increase regulated peak flow quantiles for the Skagit River near Concrete by about 5% for 50-year events and larger. Smaller events showed a smaller increase.

The weighted regulated event hydrographs for the Skagit River near Concrete were subsequently used as input to the lower basin hydraulic models used to characterize flood risk (see the Hydraulic Technical Documentation for hydraulic model details). The unregulated and weighted regulated hydrographs for the Skagit River near Concrete are provided in Appendix E.

4.4.2.5 Regulated Frequency Curve for Skagit River near Concrete

A combination of observed regulated peak flow events and hypothetical data from the reservoir regulation simulations (combination of the two methods mentioned in Sections 4.4.1 and 4.4.2.4.3) are used to calculate a regulated peak flow frequency curve at Concrete. The simulated data are used to draw the upper end of the frequency curve, while the observed data is used to define the lower end. A “best fit” line of the observed data is not used because regulated peak flow data do not fit any statistical distribution such as the Log Pearson type III (used to fit unregulated peak flow data). Frequency curves are provided in Appendix D.

The regulated frequency curve for peak annual flow at Concrete shows discontinuities or slope changes at regulated flows of about 62,000 and 90,000 cfs. These flows correspond to regulation “trigger points”. The 62,000 cfs discontinuity represents the “shutting down” of Ross and Upper Baker Reservoir discharges to minimum flows due to a forecast of 90,000 cfs at Concrete. The flattening of the plotting positions at 90,000 cfs represents regulation attempts to limit river flows to this value. The regulated curve does not merge back into the unregulated frequency curve at high exceedance frequencies. This is due to continued peak flow reductions as project releases follow the gate regulation schedules per the Water Control Manuals.

4.4.3 Confidence Limits for The Regulated Frequency Curve at Concrete

Confidence limits for the Skagit River at Concrete regulated frequency curve were developed using the HEC-FDA computer program (flood damage analysis program). The confidence limits are derived using the “ordered statistics” approach outlined in the USACE engineering technical letter 1110-2-537 (Uncertainty, A Guide to Dealing with Uncertainty in Quantitative Risk and Policy Analysis.)

5.0 Lower Skagit River Basin from Concrete, WA to Mouths of the North and South Forks of the Skagit River

The majority of damages in the Skagit River floodplain are found from Sedro-Woolley to the mouths of the North and South Forks of the Skagit River. It is necessary, therefore, to translate the regulated Skagit River near Concrete flows downstream to this reach. This requires routing these flows using a hydraulic model (see Hydraulic Technical Documentation for more information on the model) and adding in the local tributary flows that enter in along this reach.

From Concrete to the mouths of the North and South Forks, the Lower Skagit River Basin has 368 square miles of additional drainage area. This lower basin analysis focuses on producing local flows from Concrete to Sedro-Woolley and for Nookachamps Creek.

The lower basin analysis also includes estimation of flows for the Samish River. While the Samish River is not a tributary to the Skagit per se, during large floods, a portion of the spill from the right bank of the Skagit between Sedro-Woolley and Burlington flows north and co-mingles with flows from the Samish before discharging to Samish Bay. The drainage area of the Samish River at its mouth is about 106 square miles.

The hydrology investigation does not compute discharges along the mainstem Skagit River below Concrete due to unknown routing effects. The river below Concrete spreads out into a wider and shallower flood plain. The Skagit River water surface elevation becomes much more sensitive to channel characteristics with and without levees, changing floodplain widths, bridge crossings, and back-water caused by slower velocities as the gradient reduces near the mouth. A hydraulic model is used to calculate the time-varying discharges and stages along the Skagit River instead of a hydrologic model. The hydraulic model takes the weighted regulated discharges at Concrete, adds tributary flow along the lower Skagit River and calculates information that is used to construct discharge frequency curves for the damage reaches downstream of Sedro-Woolley.

5.1 Local Flow from Concrete to Sedro-Woolley

There are 13 creeks that flow into the Skagit River between Concrete and Sedro-Woolley. These drainages are Finney Creek, Presentin Creek, Grandy Creek, Mill Creek, Boyd Creek, O'Toole Creek, Alder Creek, Cumberland Creek, Jones Creek, Day Creek, Sorenson Creek, Gilligan Creek, and Hansen Creek. This local flow enters the Skagit River from River Mile 54.1 to River Mile 24.2 and has a drainage area of 278 square miles.

Streamgage information on tributaries in the lower Skagit River basin is limited. The significant tributary gages in the lower Skagit River basin are Alder Creek near Hamilton which existed from 1944-79 and has a drainage area of 10.7 square miles, Day Creek near Lyman which existed from 1944-61 and has a drainage area of 34.2 square miles, Day Creek near Hamilton which existed from 1962-69 and has a drainage area of 32.3 square miles, East Fork Nookachamps Creek near Clear Lake which existed from 1944-1950, 1962-1963 and 2001-present and has a drainage area of 20.5 square miles, Finney Creek near Concrete which existed from 1943-8 and has a drainage area of 51.6 square miles, Hansen Creek near Sedro-Woolley which existed from 1943-5 and has a drainage area of 9.66 square miles, and Samish River near Burlington which existed from 1943-71, and 1997–present, and has a drainage area of 87.8 square miles. The two Day Creek gages can be merged together with a small adjustment for drainage area to make a continuous record from 1944 to 1969.

It would be ideal to perform regressions with the lower basins to the unregulated Skagit River near Concrete flows to be consistent with how the upper basin flows are developed. However the lower basin flows do not correlate well with the unregulated flows calculated at Concrete particularly for the higher flows that are being developed. This occurs for several reasons. From 1955-75, the mainstem Skagit River did not experience very large floods. This leaves the 1949 and 1951 floods as the only large floods that some of these gages represent. As is detailed in Section 2.4.9, the 1949 flood had very little precipitation in the lower basin whereas the 1951 flood had a significant contribution from the lower basin. This variation is not unusual and can be seen in the most recent October 2003 event versus the November 9-12, 1990 event. In 2003, the storm hung up on the mountains and continued to rain long after the lower valley had dried out. The event was also preceded by a very dry summer that helped the ground to absorb more in areas that did not receive as much precipitation. The 1990 event was preceded by a very wet month and had a significant low elevation snowpack that added a lot to the lower basin local flows.

The fact that there is not a consistent pattern between the flows seen in the lower basin to the flows seen in the upper basin is not a problem if there is enough data because an average condition can be derived. The concern with the limited data that is present for the lower basin is that it can be skewed to one or two specific conditions. This is what may occur if regressions are done with the data that has only the 1949 and 1951 peak flows. It is for this reason that a correlation with a longer period of record was looked for. There are two gaged basins that drain a nearby area and have a long period of record. These two gages are the North Fork Stillaguamish River near Arlington that has been recording from 1928 to present and drains an area just over the southern ridge of the Skagit River from the Sauk to Sedro-Woolley and the South Fork Nooksack River near Wickersham that has been recording from 1934 to present and drains an area just over the northern ridge of the Skagit River from roughly River Mile 45 to Sedro-Woolley.

In performing 1-day regressions with the lower Skagit River basin flows to these two basins, it is clear that the North Fork Stillaguamish correlates quite well with these Skagit River tributary flows. The North Fork Stillaguamish River runs parallel to the Skagit

River in a direction from East to West while the South Fork Nooksack River runs in more of a U-shaped pattern from South to North. It is likely that this similarity makes the North Fork Stillaguamish River correlate a lot better with the lower basin (Alder, Day, Finney, EF Nookachamps) flows in the Skagit than the South Fork Nooksack River does. Using the North Fork Stillaguamish River adds five Skagit River flows that are larger than the 1949 and 1951 events at Concrete and another five events that are within 15% of these events. This greatly improves the confidence of the definition of the upper flows in the regression relationship between the Skagit River near Concrete and coincident flows on the North Fork Stillaguamish.

The general approach adopted for estimation of coincident lower basin tributary flows between Concrete and Sedro-Woolley was thus a two-step regression. Firstly, a regression relationship was developed between 1-day unregulated peak flows for the Skagit River near Concrete and 1-day peak flows for coincident floods on the North Fork Stillaguamish River. Secondly, regression relationships were developed between 1-day peak flows for the North Fork Stillaguamish River and 1-day peak flows from coincident floods on the lower basin tributaries. The 1-day unregulated flow quantiles for the Skagit River near Concrete derived from frequency analysis (see Section 4.2) were then used as input to the regression relationships to determine first the coincident 1-day peak flow for the corresponding return period for the North Fork Stillaguamish, which flow was then used to determine the coincident 1-day peak flow for the lower basin tributary. Note that due to timing differences, 1-day peak flows in coincident floods sometimes occur on different observation days. Timing differences between flood events on the Skagit River near Concrete and coincident lower basin floods are discussed in Section 5.4.

Because of the limited data sets of some of the lower basins, it was felt necessary to use multiple winter flood events per year to better define the relationship between the flows seen on the North Fork Stillaguamish River compared to the lower Skagit River basin flow. For the regression that determines the relationship between the North Fork Stillaguamish River and the unregulated Skagit River near Concrete flows, all separable floods greater than 30,000 cfs near Concrete are used for the entire period of concurrent record (1943-2007). For the regressions that determine the relationship between the lower Skagit River tributary flows and North Fork Stillaguamish River flows, all separable floods greater than 5,000 cfs on the North Fork Stillaguamish are used for the entire periods of concurrent record.

It is then necessary to determine which of the lower Skagit River tributary flows best represent the flows seen in the entire reach from Concrete to Sedro-Woolley. On the right bank, the only gages that are present are on Alder and Hansen Creeks. Alder Creek's longer record gives greater confidence in the data set. Most of the tributaries along this right bank are similarly oriented in the North to South direction and all have similar sized drainage areas (less than 20 square miles). The limited data set for Hansen Creek shows a slightly higher runoff per square mile but not significantly or consistently enough to justify using a different runoff per square mile runoff ratio for the rest of the basin. Therefore, the entire right bank runoff (69.8 square miles) is estimated from the regression with Alder Creek.

The left bank is a little more complicated. Day Creek has the best record and also is in the middle of the Concrete to Sedro-Woolley reach. In looking at Finney Creek upstream and the East Fork of the Nookachamps downstream as well as the flows from the right bank, Day Creek has a significantly higher runoff per square mile than its counterparts. This is likely due to an orographic effect from the fact that it is surrounded by the Cultus Mountains on the west and Coal Mountain on the east. Finney Creek at the very upstream part of this lower reach and the East Fork Nookachamps Creek on the very downstream part of this lower reach, however, do have very similar runoff per square mile ratios. Because the majority of the tributaries coming in from the left bank enter in the upper half of this lower reach, Finney Creek is used to determine the runoff from the left bank with the exception of Day Creek (174 square miles). Given the short record available, Finney Creek flows were estimated by regression against Day Creek, which flows were in turn estimated by regression against the North Fork Stillaguamish.

All regression relationships are shown in Appendix F and 1-day flows are listed in Table 20 below.

The HEC-RAS hydraulic model uses a single inflow hydrograph uniformly distributed from Concrete to Sedro-Woolley. The bottom line of Table 20 represents the total inflow to the HEC-RAS hydraulic model for the Concrete to Sedro-Woolley reach.

TABLE 20 – CONCRETE TO SEDRO-WOOLLEY 1-DAY COINCIDENT FLOWS (IN CFS) DERIVED FROM REGRESSION EQUATIONS

Location	2-year	5-year	10-year	25-year	50-year	75-year	100-year	250-year	500-year
Unregulated Skagit River near Concrete	68000	105000	134000	174000	207000	227000	242000	294000	336000
North Fork Stillaguamish River near Arlington	15450	20120	23780	28830	33000	35520	37410	43980	49280
Day Creek	2270	2890	3380	4050	4610	4940	5190	6070	6780
Finney Creek	1880	2320	2670	3150	3540	3780	3960	4590	5090
Alder Creek	210	280	330	410	470	510	540	640	720
Left Bank Flows without Day Creek	6350	7840	9010	10620	11950	12760	13370	15460	17160
Right Bank Flows without Alder Creek	1140	1530	1830	2250	2600	2810	2970	3520	3960
Total Concrete to Sedro-Woolley Local	9950	12530	14580	17390	19630	21110	22040	25710	28620

It is recognized that there is considerable uncertainty in the estimates of coincident lower basin flows due to both the paucity of data and the poor regression relationships. Nor is it clear that the two-step regression relationships described above increase the reliability of estimates compared with a direct regression between flood flows for the Skagit River near Concrete and coincident lower basin flows. However we note that, on average, the lower basin tributary inflows peak roughly 17 hours before the peak flow on the Skagit River near Concrete (see Section 5.4). Peak flows for the Skagit River at Sedro-Woolley are thus insensitive to uncertainty in the lower basin tributary inflows.

5.2 Nookachamps Creek

Nookachamps Creek flows northwest into the Skagit River on the left bank at River Mile 18.8, downstream from Sedro-Woolley. Nookachamps Creek flows mostly northwest from Lake McMurray on the west fork and Cultus Mountain on the east fork. It has a total drainage area of 71.6 square miles.

A gage on the East Fork Nookachamps Creek near Clear Lake was operated by the USGS (USGS gage 12200000) from 1944-1950 and 1962-1963 and by Washington State Department of Ecology (WSDOE gage 03G100) from 2001-present. The drainage area at this gage site is 20.5 square miles. A gage on the west fork, Nookachamps Creek at Baker Heights (USGS gage 12199600), was operated for water years 2007-2008 and has a drainage area of 25.5 square miles. Analysis of flows on Nookachamps Creek is complicated by the different characteristics of the east fork and west fork and by the short record available (2 years only) on the west fork.

The east fork drains Cultus Mountain. Slopes are moderately steep and response to rainfall is rapid. Furthermore, storm rainfall amounts on Cultus Mountain are expected to be significantly higher than over the west fork due to orographic effects. Sub-basin average 100-yr 24-hour rainfall amounts estimated by the Oregon Climate Service are about 7 inches above the east fork gage and about 4.5 inches above the west fork gage.

In contrast to the east fork, the west fork is a low gradient stream, with peak flows significantly attenuated by floodplain storage and by routing through a number of lakes (notably Lake McMurray and Big Lake).

Estimates of coincident 1-day peak flows for the East Fork Nookachamps Creek were first derived by regression against 1-day unregulated annual peak flows for the Skagit River near Concrete. The relationship between peak flows on the Skagit River and coincident flows on the East Fork Nookachamps Creeks is poor. The regression relationship is shown in Appendix F and the estimated 1-day coincident flows for the East Fork Nookachamps Creek at the gage site are listed in Table 21 below. These flows were then adjusted for the total drainage area of Nookchamps Creek of 71.6 square miles as follows.

Comparison of the short period of concurrent daily flow record from the east fork and west fork for high flow events with combined daily peak discharges greater than 400 cfs shows that the 1-day peak discharge for the combined flow (combined drainage area of

46 square miles) is on average about 40% greater than the corresponding 1-day peak discharge from the east fork gage alone (drainage area 20.5 square miles). The coincident 1-day flows for the east fork gage site were thus multiplied by 1.4 to estimate coincident flows for the combined gaged area of the basin. These flows were then multiplied by the ratio of total drainage area to gaged area (71.6/46 = 1.56).

The resulting estimates of coincident 1-day peak flows for Nookachamps Creek are listed in Table 21 below.

5.3 Samish River

The Samish River flows generally southwest onto the Skagit River floodplain just north of Burlington and then flows west and northwest to discharge into Samish Bay near Edison. The drainage area of the Samish River where it crosses Interstate-5 at the edge of the Skagit floodplain is approximately 94 square miles. The drainage area at the mouth at Samish Bay is reported as 106 square miles. The Samish River basin upstream from I-5 is in mixed agricultural and forest land-use with some areas of low density residential development. Downstream from I-5, the basin is almost entirely agricultural.

Streamflow data for Samish River are available from Samish River near Burlington (USGS gage 12201500). Daily data are available from 1943-1971 and 1997-present. Annual instantaneous peak flows are available for water years 1944-1984 and 1997-present. The drainage area at the gage site is 87.8 square miles.

The Samish River has a longer gage record than other lower basin streams and includes data concurrent with the 1949, 1951, 2003 and 2006 Skagit River floods. Consequently, coincident flows for the Samish River were derived directly by regression of unregulated winter 1-day peak flows for the Skagit River near Concrete against coincident 1-day peak flows on the Samish (as with development of upper basin flows), using available data through water year 2007. The resulting flows were then adjusted for a drainage area of 106 square miles. The relationship between peak flows on the Skagit River and coincident flows on the Samish is poor. The regression relationship is shown in Appendix F, and the estimated 1-day coincident flows for the Samish River are listed in Table 21 below.

TABLE 21 – NOOKACHAMPS AND SAMISH RIVER 1-DAY COINCIDENT FLOWS (IN CFS) DERIVED FROM REGRESSION EQUATIONS

Location	2-year	5-year	10-year	25-year	50-year	75-year	100-year	250-year	500-year
Unregulated Skagit River near Concrete	68000	105000	134000	174000	207000	227000	242000	294000	336000
North Fork Stillaguamish River near Arlington	15450	20120	23780	28830	33000	35520	37410	43980	49280
East Fork Nookachamps Creek at gage	400	620	790	1030	1220	1340	1430	1730	1980
Total Nookachamps Creek	880	1350	1730	2240	2670	2930	3120	3790	4330
Samish River	1170	1810	2310	3000	3570	3920	4180	5080	5800

5.4 Development of Hypothetical Hydrographs for Lower Basin

The regressions provided 1-day peak flows for each of the lower basin inputs. Regressions are then developed for each of the lower basins to relate their winter peak 1-day flows to their coincident instantaneous peak and 3-day flows (see Appendix F). The lower basin hypothetical hydrographs are then shaped to match these peak, 1-day, and 3-day flows using the October 2003 North Fork Stillaguamish River hydrograph as a guide. The one exception to this approach was for Nookachamps Creek where, due to lack of data, the 1-day to instantaneous peak and 1-day to 3-day flow relationships for the Samish River were applied. The Samish River (gaged area of 87.8 square miles) has a similar basin area to Nookachamps Creek (total drainage area of 71.6 square miles) and similar land use and physiographic features.

5.5 Timing of Lower Basin Flows

The timing for when local discharges from the Nookachamps Creek and Concrete to Sedro-Woolley combine with discharges on the Skagit River can vary considerably. In the 2003 event, the North Fork Stillaguamish River peaked 6 hours before the Skagit River near Concrete. In 1995, it peaked 19 hours before Skagit River near Concrete. The Upper Basin local flow that has the same relative size of drainage basins and proximity to the mainstem Skagit is the Marblemount to Concrete local. From the Upper Basin analysis, it was determined that this local inflow peaks roughly 15 hours before the Skagit River near Concrete does on average. To be consistent with this upper basin timing and assuming that the lower local inflows would peak slightly earlier as it takes some time for the precipitation to travel from the lower basin to the upper basin, a peak timing of 17 hours before the Skagit River near Concrete peaks is used for the lower basin local inflows.

6.0 Hydrologic Results

There are several general locations where it is important to know what the derived flows are for specific events. These locations are Concrete, Sedro-Woolley, and Mount Vernon. Concrete is important because it represents the upstream location where most of the hydrology was developed. Sedro-Woolley's flows are of note because they represent the flows that enter the lower basin before the Nookachamps basin storage is accounted for. The Mount Vernon flows show how much water can make it through the narrowed levee reach. The flows listed in the tables below are derived from "infinite" levee hydraulic model runs which assume that no water can escape from the river channel due to spill, levee overtopping, or levee failure. This information is only for the purposes of understanding the amount of flow that needs to be accounted for in this lower basin. More detailed information on flows and stages for specific levee failure runs can be found in the Hydraulic Technical Documentation. Note that for consistency, all flows reported in Table 22, including the unregulated flows at Concrete, are from routing of the synthetic hydrographs (see Section 4.4.2) as opposed to results of frequency analyses.

TABLE 22 – PEAK FLOWS (CFS) AT CONCRETE, SEDRO-WOOLLEY, AND MOUNT VERNON

Recurrence	Unregulated Concrete	Regulated Concrete[1]	Unregulated Sedro-Woolley	Regulated Sedro-Woolley[1]	Unregulated Mount Vernon	Regulated Mount Vernon[1]
2-year	77,300	77,300	80,500	80,500	76,400	76,900
5-year	120,500	101,100	125,600	105,200	110,500	92,900
10-year	153,300	127,700	159,400	133,000	142,600	119,000
25-year	201,200	165,300	211,700	169,800	169,900	149,800
50-year	229,300	189,100	235,000	197,400	210,200	167,600
75-year	255,500	211,400	261,200	220,000	220,800	192,300
100-year	272,400	225,900	280,100	235,700	236,400	206,500
250-year	325,400	279,700	320,100	289,400	278,100	244,700
500-year	363,600	324,400	356,900	325,400	320,900	282,600

Notes:
1. Regulated data from weighted regulated hydrographs (see Section 4.4.2.4.3)

In addition, it is useful to see the flows derived from frequency analyses for key sub-basins. These values, provided in Tables 23 to 25, are different than the flows in Table 18 for several reasons. The first is because the flows derived in Table 18 are the coincident flows in these basins when the Skagit River near Concrete peaks, which may not correspond to the same frequency for the sub-basin. For example, if the Skagit River near Concrete is having a 100-year event, the contribution from a specific sub-basin could be a 50-year event or a 200-year event. The second complication in comparing these flows is that the analysis for the Skagit River near Concrete uses the historical flows derived by Stewart (as adjusted by the USGS in 2007), but the other gages do not use this information. This factor does not affect the results of this study as the correlations relied on in Table 18 do account for these historical flows. With these caveats, the table below shows the flows derived from frequency analyses for the most critical sub-basins; Upper Baker Dam inflow, Ross Dam inflow, and Sauk River near Sauk. Also shown for purposes of comparison are the regulated flows for the Skagit River near Concrete and Skagit River near Mount Vernon derived from routing of synthetic hydrographs with "infinite" levees.

TABLE 23 – INSTANTANEOUS PEAK FLOWS (CFS) FOR CRITICAL SUB-BASINS

Recurrence	Regulated Concrete[1]	Regulated Mount Vernon[1]	Upper Baker Dam Inflow[2]	Ross Dam Inflow[2]	Sauk River near Sauk[2]
2-year	77,300	76,900	17,200	20,100	30,500
5-year	101,100	92,900	22,300	28,000	47,900
10-year	127,700	119,000	25,800	33,100	61,300
25-year	165,300	149,800	30,400	39,600	80,200
50-year	189,100	167,600	34,000	44,300	95,900
75-year	211,400	192,300	36,200	47,100	106,000
100-year	225,900	206,500	37,700	49,000	113,000
250-year	279,700	244,700	42,900	55,200	138,000
500-year	324,400	282,600	47,000	59,900	159,000

Notes:
1. Quantiles from routing of synthetic weighted regulated hydrographs.
2. Quantiles are for computed probability using annual (full year) data through water year 2004.

TABLE 24 – 1-DAY PEAK FLOWS (CFS) FOR CRITICAL SUB-BASINS

Recurrence	Regulated Concrete[1]	Regulated Mount Vernon[1]	Upper Baker Dam Inflow[2]	Ross Dam Inflow[2]	Sauk River near Sauk[2]
2-year	68,300	72,200	11,400	9,340	22,300
5-year	88,200	89,300	16,400	16,700	35,000
10-year	113,700	115,700	19,900	22,700	44,100
25-year	143,700	143,700	24,300	31,800	56,200
50-year	172,000	165,800	27,800	39,600	65,700
75-year	191,200	182,300	29,800	44,600	71,300
100-year	206,600	200,100	31,200	48,300	75,400
250-year	254,600	236,700	34,800	61,400	88,700
500-year	297,100	273,700	39,700	72,600	99,200

Notes:
1. Quantiles from routing of synthetic weighted regulated hydrographs.
2. Quantiles are for computed probability using winter data through water year 2004.

TABLE 25 – 3-DAY PEAK FLOWS (CFS) FOR CRITICAL SUB-BASINS

Recurrence	Regulated Concrete[1]	Regulated Mount Vernon[1]	Upper Baker Dam Inflow[2]	Ross Dam Inflow[2]	Sauk River near Sauk[2]
2-year	51,800	58,700	8,360	7,680	16,400
5-year	75,300	81,500	11,600	13,200	25,200
10-year	95,500	102,900	13,700	17,600	31,500
25-year	115,900	125,000	16,400	23,800	40,100
50-year	135,900	145,800	18,400	29,000	46,700
75-year	148,600	158,300	19,500	32,200	50,800
100-year	161,400	171,300	20,300	34,500	53,700
250-year	198,200	206,500	22,900	42,600	63,400
500-year	230,000	238,600	24,800	49,300	71,100

Notes:
1. Quantiles from routing of synthetic weighted regulated hydrographs.
2. Quantiles are for computed probability using winter data through water year 2004.

6.1 Comparison with Previous Study Results

The results of the hydrologic analyses presented in this 2013 report differ from the results presented in the 2004 and 2011 draft Hydrology Technical Documentation. A comparison of estimated peak flows from the 2004 draft Hydrology Technical Documentation, the 2008 draft Flood Insurance Study, the 2011 draft Hydrology Technical Documentation, and the present work is provided in Table 26. As in Tables 22 through 25 above, the flows provided for Sedro-Woolley and Mount Vernon represent the “infinite” levee condition.

The principal factors which contributed to changes in peak discharge from the 2004 draft report to the 2011 draft report were as follows:

1. An approximately 5% reduction, by the USGS, in the estimated magnitude of the historic floods of water years 1898, 1910, 1918 and 1922.
2. Increased record length for the Skagit River near Concrete, reflecting both recent data from water years 2005 through 2007, and incorporation of data for the period 1925 through 1943 which had not previously been available.
3. A change from estimation of flood quantiles with expected probability adjustment in the 2004 report to use of computed probability flood quantiles in the 2011 (and 2013) reports, consistent with requirements for risk-based analysis and design of flood risk management projects.

The above three changes are also reflected in the peak discharge estimates reported in the 2008 draft Flood Insurance Study.

There are several other changes which accounted for differences between the 2011 work and the draft FIS study, and which also contributed to the differences between the 2011 and the 2004 drafts. These included:

4. Adjustment to some upper basin hydrographs to improve the consistency in hydrographs for different return periods and provide improved nesting of those hydrographs.
5. Modification to the spreadsheet program used to route floods through Upper Baker and Ross reservoirs to improve model representation of spillway gate regulation curves.
6. Reanalysis and reduction in Nookachamps Creek coincident flows, incorporating data either not used or not available for the earlier work.
7. Rather extensive changes to and recalibration of the HEC-RAS model representation of the Skagit River from Sedro-Woolley to Mount Vernon. Routing of flows from Sedro-Woolley to Mount Vernon is affected by several factors including floodplain storage in the Nookachamps Creek basin and assumptions regarding debris load on the Burlington Northern Railway bridge in Mount Vernon. The HEC-RAS model, its calibration, and hydraulic model results are described in detail in the Hydraulic Technical Documentation.

The principal factors which contribute to differences in estimated peak discharges between the 2011 draft Hydrology Technical Documentation and the current 2013 report are as follows:

8. Use of weighted regulated hydrographs in the current work to account for seasonal variation in flood control storage at Upper Baker and Ross reservoirs.
9. Corrections and refinements to the HEC-RAS model representation of the Burlington Northern Railway bridge, including changes to debris load assumptions. These changes affect floodplain storage in the Nookachamps Creek basin and were found to have a significant impact on unregulated flows at Mount Vernon for the 25-year event and larger as the bridge goes into pressure flow and forces water into the Nookachamps storage area at a lower discharge than previously estimated.
10. Other refinements to the HEC-RAS model, including corrections and refinements to the model representation of the Division Street bridge in Mount Vernon and the Highway 9 and former Great Northern Railway bridges immediately downstream from Sedro-Woolley.

TABLE 26 – COMPARISON OF PEAK FLOWS (CFS) AT CONCRETE, SEDRO-WOOLLEY, AND MOUNT VERNON GAGES

Recurrence Interval and Data Source	Unregulated Concrete	Regulated Concrete	Unregulated Sedro-Woolley	Regulated Sedro-Woolley	Unregulated Mount Vernon	Regulated Mount Vernon
2-yr 2004 GI	72,900	72,900	78,100	78,100	75,700	75,700
2-yr 2011 GI	77,300	77,300	80,500	80,500	76,500	76,500
2-yr 2013 GI	77,300	77,300	80,500	80,500	76,900	76,900
5-yr 2004 GI	119,400	93,900	124,300	99,400	116,500	97,300
5-yr 2011 GI	120,500	100,700	126,000	105,000	110,700	92,400
5-yr 2013 GI	120,500	100,100	125,600	105,200	110,500	92,900
10-yr 2004 GI	156,000	120,400	160,600	125,100	142,700	117,400
10-yr 2008 FIS	159,000	116,300	156,920	123,610		
10-yr 2011 GI	153,300	125,500	159,800	130,400	142,800	117,700
10-yr 2013 GI	153,300	127,700	159,400	133,000	142,600	119,000
25-yr 2004 GI	205,300	158,000	210,300	163,400	199,400	146,000
25-yr 2011 GI	201,200	159,300	203,700	162,600	192,900	143,400
25-yr 2013 GI	201,200	165,300	211,700	169,800	169,900	149,800
50-yr 2004 GI	248,100	192,100	252,000	198,500	233,700	190,900
50-yr 2008 FIS	241,000	180,260	233,290	183,780		
50-yr 2011 GI	229,300	180,300	234,800	186,100	219,100	167,700
50-yr 2013 GI	229,300	189,100	235,000	197,400	210,200	167,600
75-yr 2004 GI	248,100	192,100	252,000	198,500	233,700	190,900
75-yr 2011 GI	255,500	200,700	259,400	205,800	237,400	196,400
75-yr 2013 GI	255,500	211,400	261,200	220,000	220,800	192,300
100-yr 2004 GI	297,100	235,400	298,600	242,000	273,900	230,100
100-yr 2008 FIS	278,000	209,490	277,220	215,270		
100-yr 2011 GI	272,400	214,200	275,500	220,100	250,300	207,300
100-yr 2013 GI	272,400	225,900	280,100	235,700	236,400	206,500
250-yr 2004 GI	372,200	320,200	368,100	319,800	334,000	289,800
250-yr 2011 GI	325,400	267,400	323,500	271,800	288,000	246,300
250-yr 2013 GI	325,400	279,700	320,100	289,400	278,100	244,700
500-yr 2004 GI	437,000	386,900	429,900	380,800	396,700	346,400
500-yr 2008 FIS	373,000	316,530	371,670	322,900		
500-yr 2011 GI	363,600	313,300	353,100	314,200	317,800	280,100
500-yr 2013 GI	363,600	324,400	356,900	325,400	320,900	282,600

2004 GI: 2004 Draft Hydrology Technical Documentation
2011 GI: 2011 Draft Hydrology Technical Documentation
2013 GI: 2013 Hydrology Technical Documentation
2008 FIS: 2008 Draft Flood Insurance Study

7.0 Limits of Downstream Flood Protection

Levees in the lower valley are the only flood control structures in the basin except for the Ross and Upper Baker flood storage projects. Sixteen diking districts in the lower valley provide primary levee protection, protecting 45,000 acres of land. These levees vary in level of protection with hydraulic capacities ranging from about 80,000 cfs to 150,000 cfs. Individual owners have constructed private levees that protect an additional l,000 acres. Between Concrete and Sedro-Woolley, low levees protect several rural areas. Most of the levees were constructed years ago by farmers and local people attempting to protect their property. Many of these older levees have been raised and strengthened in recent years, but sub-standard foundation materials make them vulnerable to failure during major floods due to seepage and erosion conditions. Table 27 is taken from the Water Control Manuals for both Ross and Upper Baker Dams to show the flow levels that create problems in the lower basin.

TABLE 27 - FLOOD CONDITIONS RELATED TO THE GAGE
SKAGIT RIVER NEAR MOUNT VERNON, WASHINGTON

Stage (Ft.)	Discharge		Character of Flooding
	(cfs)		
25.0	53,200	1.	Beginning of backwater in Nookachamps Creek area with flooding of low-lying farmlands --no damage
28.0	67,850	1.	Zero damage
30.3	82,260	1.	Beginning of flooding in town of Hamilton
		2.	South End of Francis Road is overtopped and closed to traffic which is the road to Sedro-Woolley via Clear Lake. Those living in this lower area on Francis Road no longer have an escape route.
		3.	Beginning of overland flow to levee east of Burlington on Fairhaven Street, on north side of river between Sedro-Woolley and Burlington.
32.7	100,300	1	Major damage discharge in the vicinity of Mount Vernon
33.8	110,000	1.	Levee freeboard as follows: Levee east of Burlington on Fairhaven Street -3 to 4 feet.
		2.	Levee failures may occur when river remains above this stage more than 24 hours, with flood conditions varying as levees fail or are overtopped throughout the valley
		3.	In view of the inadequate cross-section of practically all Skagit River dikes, the following action should be taken by the Corps at this time <u>if a 2-foot rise is indicated in the next 24 hours:</u> **Be prepared to evacuate flood fighting crews from areas below Mount Vernon.**
36.60	141,500	1.	Flooding expected in many districts. Dikes on either right or left bank from Hwy. 99 bridge downstream to Mt. Vernon may be breached
38.1	160,000	1.	Emergency raising of Burlington and Mount Vernon levees necessary to prevent flooding

U.S. Army Corps
of Engineers
Seattle District

Skagit River Flood Damage Reduction Feasibility Study

SKAGIT RIVER BASIN

SEDIMENT BUDGET AND FLUVIAL GEOMORPHOLOGY

June 2008

SKAGIT RIVER BASIN

SEDIMENT BUDGET AND FLUVIAL GEOMORPHOLOGY

Table of Contents

1.0 INTRODUCTION

This report describes the Skagit River's baseline sediment budget and fluvial geomorphology. These are important factors that have shaped the existing stream system and will influence the impacts of any flood damage reduction measures that may be implemented. Other landscape shaping factors, such as geology and climate, are summarized to give a background for the fluvial geomorphology. The methods and analysis followed in this investigation generally satisfy the criteria for a Stage 1 Sediment Impact Assessment as established in EM 1110-2-4000 *Sedimentation Investigations of Rivers and Reservoirs* (USACE 1995).

1.1 General

Authority for the Skagit River, Washington, Flood Damage Reduction Feasibility Study is derived from Section 209 of the Flood Control Act of 1962 (Public Law 87-874). Section 209 authorized a comprehensive study of Puget Sound and Adjacent Waters, including tributaries such as the Skagit River, in the interest of flood control, navigation, and other water uses and related land resources. The current feasibility study was initiated in 1997 as an interim study under this statutory authority. Skagit County is the local sponsor of the feasibility study. The purpose of the study is to formulate and recommend a comprehensive flood hazard management plan for the Skagit River floodplain that will reduce flood damages in Skagit County with the focus on the floodplain downstream of Sedro-Woolley. A secondary purpose is to investigate measures to restore ecosystem functions in the project area to benefit fish and wildlife.

In order to identify potential ecosystem restoration actions and to comply with the impact assessment requirements of the National Environmental Policy Act (NEPA) and the Washington State Environmental Policy Act (SEPA) the Skagit River Flood Damage Reduction Feasibility Study must describe the affected environment, including both physical and biological resources. This report addresses those requirements by describing the lower Skagit River's sediment budget and geomorphology. Understanding the sediment budget and fluvial geomorphic processes are important to the formulation of project alternatives and defining potential environmental impacts.

1.2 Purpose of Report

This is the second flood damage reduction study report to address geomorphology and sediment processes in the Skagit River. The Phase I report by Pentec Environmental (2002) described the geomorphology of the river channels downstream of River Mile (RM) 30. The main purpose of this report is to describe the basin-wide sediment budget and the geomorphology of the river and delta channels, and the nearshore areas. This will provide a baseline to evaluate potential sediment budget and geomorphic impacts of alternative flood damage reduction and environmental restoration measures. The main components of this effort include:

- Annual basin sediment yield estimate
- River and delta channel geomorphology
- Nearshore geomorphology

1.3 Study Area

The study area for sediment budget estimates takes in the uncontrolled portions of the Skagit River basin, downstream of Gorge and Lower Baker dams (see Figure 1). The geomorphic analysis is focused on the mainstem Skagit River, the North and South Fork channels, and the Puget Sound nearshore.

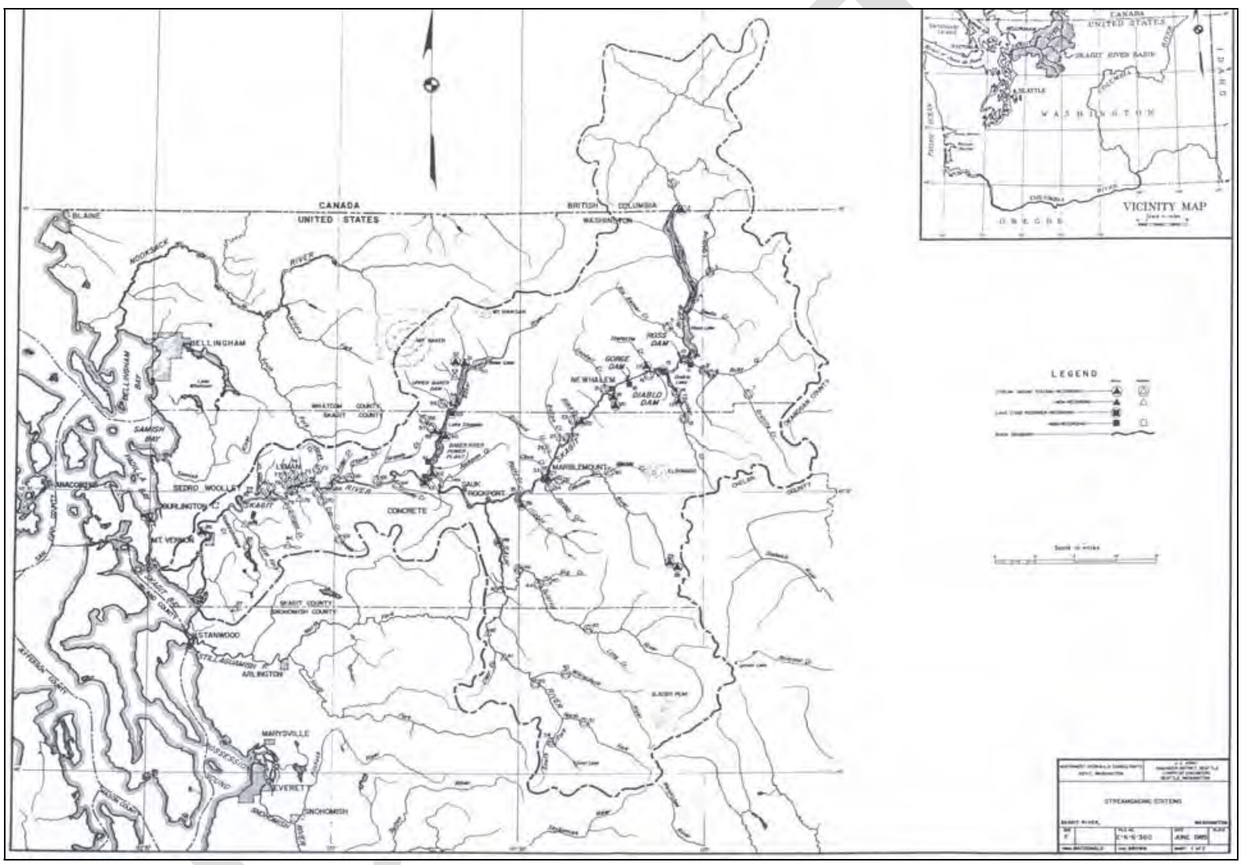


Figure 1 – Skagit River Basin Map

2.0 GENERAL BASIN CHARACTERISTICS

The Skagit River basin is located in the northwest corner of the State of Washington (see Figure 1). The northern end of the basin extends 28 miles into Canada. The Skagit River drainage area is 3,115 square miles, with slightly over half the area controlled by reservoirs. The basin extends about 110 miles in the north-south direction and about 90 miles in the east-west direction between the crest of the Cascade Range and Puget Sound.

2.1 Topography

A major portion of the Skagit River basin lies on the western slopes of the Cascade Range. Most of the eastern basin is mountainous, with 22 peaks higher than 8000 ft. Many of those peaks are topped by glaciers. The two most prominent topographical features in the basin are Mount Baker at an elevation of l0,778 feet on the western boundary of the Baker River basin, and Glacier Peak at an elevation of l0,568 ft in the Sauk River basin. The upper reaches of nearly all tributaries are situated in steep-walled mountain valleys. The middle and lower reaches of the tributaries are covered by timber.

Upstream of the Cascade River at RM 78, the Skagit River flows through a narrow, steep-walled canyon. From the Cascade River down to Sedro-Woolley (RM 23) the Skagit River flows in a l-mile to 3-mile wide valley. In this reach, the valley walls are moderately steep, timbered hillsides with few developments. Downstream of Sedro-Woolley, the river flows through the cities of Burlington and Mount Vernon, and then divides into North and South forks before discharging into Puget Sound. In this reach, the floodplain widens to a flat, fertile outwash plain that adjoins the Samish valley to the north and the Stillaguamish valley to the south.

2.2 Geology

The eastern mountainous region of the upper Skagit Basin consists of ancient metamorphic rocks, largely phyllites, slates, shales, schists, and gneisses together with intrusive granitic rocks and later andesitic lavas and pyroclastic deposits associated with Mount Baker and Glacier Peak. The valleys are generally steep sided and frequently flat floored. Alpine glaciers have contributed to the steepness of the valley sides and to the depth of the valley bottoms. Over ten thousand years ago the upper Skagit Valley and the peaks were severely glaciated, removing not only the soil, but much of the loose rock. Glaciation exerted a powerful influence on the geomorphology of the Skagit River basin. Drainage patterns in the basin have many peculiar features, including long interconnected valleys, breached hydrologic divides, bisected valleys, and low-elevation mid-valley divides occupied by lakes and wetlands. The Skagit basin was likely much smaller prior to Quaternary glaciation. Geological evidence suggests overflow of proglacial lakes breached the North Cascades crest at Skagit Gorge and caused the lower Skagit River to capture upper Skagit valley (Riedel et al. 2007).

Many river channels created during the glacial melt have continued to aggrade, and as a result of that glacial action, the bedrock bottoms of most canyons are covered with glacial alluvium.

These deposits are a heterogeneous mixture of sand and gravel together with variable quantities of silt and clay depending on the mode of deposition. Some of these deposits are highly susceptible to land sliding when saturated. The floodplain of the Skagit River below Concrete is composed of sands and gravels that diminish to sands, silts, and some clays further downstream. Below Hamilton, fine-grained floodplain sediments predominate.

Two volcanoes, Glacier Peak and Mt. Baker, are located in the upper watershed. Previous eruptions of Glacier Peak have generated lahars that traveled through the Skagit River to Puget Sound. Mt. Baker eruptions have deposited pyroclastic and lahar material in the Baker River watershed, but have not deposited substantial volumes material in the Skagit River floodplain (Gardner, et al, 1995). Future large eruptions could form thick fills of lahars and pyroclastic-flow deposits in the upper valleys near the volcano. Lahars from Glacier Peak could reach the delta, or there could be induced flooding due to temporary damming of watercourses in the upper watershed. Subsequent incision of volcanic deposits could fill riverbeds farther downstream with sediment for many years after the eruption, thereby affecting the capacity of stream channels and locally increasing flood heights (Waitt, et al, 1995).

2.3 Watershed Description

Headwaters of the contemporary Skagit River basin originate in a network of narrow, precipitous mountain canyons in Canada and flows south into the United States and then west for over 100 miles to Skagit Bay. The Skagit River falls rapidly from near 8,000 ft at its source in Canada to 1,600 ft at the head of Ross Reservoir at RM 128. Ross Reservoir and the associated Diablo (RM 101) and Gorge (RM 97) reservoirs reduce flood discharges, store spring snowmelt runoff, and trap sediment from 1,125 sq mi of the headwaters of the Skagit River.

From Gorge Dam to Newhalem (RM 94) the Skagit River plunges 250 ft in less than 3 miles. Downstream of Newhalem the river's slope flattens substantially to approximately 8 feet per mile between Newhalem and Concrete (RM 56). Numerous tributaries enter the Skagit River in this reach. Many of those tributaries are relatively small, consisting of steep heavily forested basins with drainage areas of less than 20 sq mi that discharge directly into the Skagit River. However, there are three large drainage basins; the free-flowing Cascade and Sauk Rivers and the regulated Baker River.

The Cascade River has a drainage area of 185 square miles and enters the Skagit River at RM 78.1, just upstream of the town of Marblemount. The Cascade River runs for 29 river miles north and west from South Cascade Glacier on Sentinel Peak to the Skagit River. The basin ranges in elevation from 300 to 8,300 feet. The Cascade River is classified as a Wild and Scenic River. The basin is mostly forested and the river opens from a roughly 400-foot wide canyon at RM 3.3 to a 2800-foot wide floodplain at its mouth. The Cascade River is the second largest contributor to the sediment to the Skagit River.

The Sauk River is the largest tributary to the Skagit River and flows into it on the left bank at RM 67.2. The Sauk River is also designated a Wild and Scenic River. The Sauk River originates near Monte Cristo Peak and flows generally north for over 50 miles. The Sauk River has a drainage area of 732 miles, which is over 25% of the total drainage area of the Skagit River at their confluence. It is also approximately 50% of the total uncontrolled sediment contributing area in the basin. There are two large tributaries that flow into the Sauk River from Glacier Peak. The largest is the Suiattle River (346 square mile drainage area), which flows in from the east at River Mile 13.2 and is over 40 miles in length. The White Chuck River (86.2 square mile drainage area) flows in from the east at River Mile 31.9. The elevations in the basin range from 10,541 feet to 210 feet at the mouth. The high elevation headwater areas have sparse vegetation and several peaks are glaciated. The middle and lower watershed is forested. The lower reaches of the rivers have braided and meandering channels with unstable banks. The Sauk River watershed is the largest contributor to the sediment to the Skagit River.

The Baker River enters the Skagit River from the north at RM 56.5, at the town of Concrete. The Baker River has a drainage area of 298 sq mi. The basin has several high peaks including Mount Baker, Mount Shuksan, Whatcom Peak, and Bacon Peak. The runoff from 297 sq mi drains into Lake Shannon or Baker Lake. The temporary storage of flood discharges in those lakes greatly reduces flood peaks and the sediment yield from the Baker River.

From Concrete (RM 56) to Sedro-Woolley (RM 23) a few small tributaries enter the Skagit River from both banks. Those tributaries originate in the forested, lower elevation foothills of the Cascade Mountains. Potentially larger tributary flows from Mount Baker are intercepted by the South Fork of the Nooksack River. The valley floor has somewhat irregular topography and is typically a half-mile to a mile wide. Most of the valley floor is utilized for agriculture.

Downstream from Sedro-Woolley (RM 23), the Skagit River crosses a broad outwash plain before discharging into Skagit Bay in Puget Sound. The floodplain stretches north-south about l9 miles, from Samish Bay on the north, to Camano Island on the south. The floodplain is a rich agricultural area. The cities of Burlington, Mount Vernon and La Conner are located on this floodplain. Nookachamps Creek is the only significant tributary in this reach. Immediately downstream from Mount Vernon, the river divides into two distributaries, the North and South forks. These two distributaries carry about 60 percent and 40 percent of the normal flows of the Skagit River, respectively.

2.4 Climate

The major factors influencing the climate of the Skagit River basin are terrain, proximity of the Pacific Ocean, and the position and intensity of the semi-permanent high and low pressure centers over the north Pacific. The basin lies about 100 miles inland from the moisture supply of the Pacific Ocean. Westerly air currents from the ocean prevail in these latitudes bringing the region considerable moisture, cool summers, and comparatively mild winters.

Annual precipitation varies markedly throughout the basin due to elevation and topography. Mean annual precipitation is 40 inches or less near the mouth of the Skagit River and in the portion of the basin in Canada that lies in topographic rain shadows. An average annual precipitation of l80 inches or more falls on the higher elevations of the Cascade Range in the southern end of the basin and over the higher slopes of Mount Baker. The annual precipitation over the basin above the town of Mount Vernon averages 92 inches with approximately 75 percent of this amount falling during the 6-month period, October-March.

Snowfall in the Skagit River basin is dependent upon elevation and proximity to the moisture supply of the ocean. The mean annual snowfall at stations in the basin varies from 6 inches at Anacortes to 525 inches at Mount Baker Lodge.

Major storm activity occurs during the winter when the basin is subject to rather frequent ocean storms that can bring heavy rain or snow to the mountains. The type and timing of precipitation in the mountains influences the basin's sediment production. Rain on bare ground is expected to produce more sediment erosion than typical rain-on-snow events and regional snowmelt events, especially when intense rains fall on saturated ground. However, there is insufficient sediment data to attempt to determine the most significant sediment producing storm conditions in the Skagit Basin.

2.5 Hydrology

This report summarizes the Skagit River hydrology. A more detailed explanation of the hydrology is given in the Skagit River Hydrology Report that is also part of this Skagit River Flood Damage Reduction Study.

The Skagit River basin is subject to rain and snowmelt runoff during the fall and winter, and snowmelt runoff during the spring. Spring snowmelt runoff is caused predominantly by melting of the winter snowpack and is characterized by a relatively slow rise and long duration. Some minor contribution to the rate and peak of the snowmelt is occasionally provided by warm spring rains, but the spring rain-on-snow impact is usually not significant. Highest mean monthly snowmelt discharges are usually reached in June. The Skagit River and all of its major tributaries usually have low flows during August and September after the high-elevation snowpack has melted and the baseflow has receded. Glacial melt continues to contribute to the baseflows during this period.

With the advent of heavy precipitation in the fall and winter, the Skagit River experiences a significant flow increase. Floods and the highest daily and highest instantaneous peak discharge of the year usually occur during this period. Heavy rainfall and warm winds during typical 1-3 day winter storms cause streamflows to rise rapidly. Streamflows also recede rapidly after the storms have moved eastward through the region, although base flows and basin soil moistures usually remain high for several days. Several minor rises usually occur each winter, while major floods are more intermittent. Winter rain-type floods usually occur in November or December but may occur as early as October or as late as February.

Annual runoff varies throughout the Skagit basin. The average annual runoff at the following streamgage stations reflects that variation; Skagit River at the Newhalem, 51.1 inches, Sauk River Near Sauk, 83.0 inches, Baker River at Upper Baker, 131.0 inches, Baker River at Concrete, 121.8 inches, and Skagit River near Mount Vernon, 73.2 inches. The watershed above Ross Dam, located in the rain shadow of western mountains that shield the basin from winter storms, has an annual runoff of only 45.6 inches.

3.0 SEDIMENT BUDGET

A sediment budget is an important component for understanding the sedimentation processes that help shape a river system and how potential flood control measures could impact the river environment (USACE 1995). An ideal sediment budget accounts for the major sources of sediment and identifies the location, timing, and material size distribution of the sediment moving through a river system. In the Skagit River basin there is insufficient data to construct a highly detailed, conclusive, sediment budget; but a preliminary sediment budget can be developed from existing information to help further the understanding of the geomorphic processes of the Skagit River system.

3.1 Methodology

The annual sediment yield for the Skagit River Basin was estimated by separate geomorphic and hydraulic methods. The geomorphic method estimates the annual sediment yield from upland areas based on sub-basin characteristics and provides information about the sources of sediment in the river system. The hydraulic method utilizes measured sediment transport data to calculate the annual sediment discharge in the river, and provides information about the sediment size and timing of the sediment yield.

The geomorphic method estimates the annual basin sediment budget based on geology, land use and erosion processes present in the basin (Swanson, et al, 1982). The first step in this method is to identify the dominant erosion processes. Field measurements or aerial photography are then used to measure the sediment produced by a sampling of those erosion processes, usually measured over a period of years. The measured erosion rates are then applied to the entire basin to estimate the average annual sediment budget.

The hydraulic method combines a sediment load curve (a numerical relationship between sediment discharge and water discharge) with observed water discharge data to calculate an annual sediment transport. This method requires streamflow and sediment transport measurements at one or more locations in the river system. The importance of high flow events on overall sediment yield can be determined from this method.

3.2 Basin Sediment Budgets

This analysis applies the geomorphic method to Skagit River sub-basins to estimate average annual sediment budgets for each sub-basin and the entire watershed. A comprehensive assessment and inventory of sediment sources and yield does not exist for the Skagit River basin. Paulson (1997) developed annual sediment budgets for 9 Skagit River sub-basins. Paulson examined three erosion processes; mass wasting, surface erosion of roads and soil creep. Of those, mass wasting was found to be the dominant erosion process. Paulson then investigated the failure mechanisms, and geologic and land use influences on mass wasting. Three mass failure mechanisms, shallow-rapid landslides, debris flows and earth slumps were identified. It was determined that debris flows and earth slumps delivered over 75 percent of the mobilized material to the streams. Paulson's results consistent with other studies conducted on forested, west coast watersheds that found mass wasting to be a dominant source of sediment in steep watersheds (Benda and Dunne 1997; Montgomery et al. 1998).

An average annual sediment budget was developed for this study, based on Paulson's work (1997). There is not adequate information available for the remainder of the basin to perform the detailed analysis of geology and land use that Paulson performed in the 10 sub-basins. Therefore sediment yields were extrapolated to other sub-basins based on proximity to one of Paulson's sub-basins. Table 1 lists the average annual sub-basin sediment yields estimated for this analysis.

Table 1. Skagit River Basin Average Annual Sediment Budget by Sub-basin derived from results obtained by Paulson (1997).

Sub-basin	Sediment Contributing Drainages in Mi^2	Non-Contributing Drainages in Mi^2	Annual Sediment Yield Yds^3/Mi^2	Annual Basin Sediment Yield in Yds^3
Upstream of Gorge Dam		1159		
Skagit u/s of Cascade River	228		280	63,840
Cascade River	185		160	29,600
Jackman Creek	24		3800*	91,200
North side tributaries	40		280	11,200
Illabot Creek	42		160*	6,720
Sauk River	732		400	292,800
Baker River		297		
Finney Creek	52		800*	41,600
South side tributaries	90		260	23,400
East Fork Nookachamps Creek	36		260	9,360
Lower Valley Floor		160		
North side tributaries	70		460	32,200
Basin Totals	1,499	1,616		601,920

* Basin yield taken directly from Paulson, 1997.

Nichols (personnel communications, 2006) recommended sediment yields would be significantly higher from glaciated areas. It was decided to add those source areas as a separate item in the sediment budget. Table 2 shows the glaciated drainage areas, estimated from maps and aerial photographs, which would contribute to the sediment yield from the Skagit Basin. Nichols (2006) estimated the glaciated areas would produce 2,600 tons/sq mi/yr or around 1,900 cu yds/sq mi/yr. Thus the 56 sq mi of glaciated area listed in Table 2 could add about 100,000 cu yds/yr of sediment.

The average annual sediment budget for the Skagit River Basin, based on key geomorphic processes that are shaping the watershed, is estimated to be between 600,000 and 700,000 cu yds/yr. Nearly half the sediment is produced by the Sauk River sub-basin, the largest free-flowing tributary to the Skagit River. The high sediment yield from Jackman Creek demonstrates the uncertainty of this type of analysis, as about 50 percent of the sub-basin yield came from one large mass failure (Paulson, 1997). That one large failure also highlights the limitation of focusing on average annual sediment values, as the equivalent of several years of sediment can be delivered in a single large event.

Table 2. Glaciated drainage areas that contribute to the Skagit River Basin sediment budget.

Sub-basin	Glaciated Area in Sq Mi
White Chuck River (Sauk)	6
Suiattle River (Sauk)	15
Cascade River	23
Newhalem Creek	2
Ladder Creek	1
Goodell Creek	7
Bacon Creek	2
Basin Total	56

3.3 Fluvial Sediment Budget

In this analysis the average annual sediment yield for the Skagit River was calculated from streamflow and suspended sediment data collected by the USGS at their Mount Vernon gaging station and by Pentec (2002). Suspended sediment data collected from 1971 to 1993, 2001, and 2006 were used to define a sediment load curve, the numerical relationship between water discharge and the sediment transport rate. The sediment load curve was then combined with daily discharges from 1940 through 2004 to calculate daily and annual fluvial sediment transport. As is explained below, this method contains some uncertainty in the sediment transport parameters because of the limited amount of available sediment data.

Prior to this study, Skagit River suspended sediment data had been collected between 1971 and 1993 by the USGS. A review of that data found there was no information available for discharges above 50,000 cfs. This was considered a significant limitation given that the highest 1 percent of the daily discharge record all exceeded 50,000 cfs, ranging up to a maximum of 142,000 cfs, and flood peaks can exceed 200,000 cfs. As a result, the Corps collaborated with the USGS to collect suspended sediment data from a large storm in November 2006. The five samples collected in November 2006 were from discharges ranging from 63,000 cfs to nearly 120,000 cfs. The complete suspended sediment data set and the best fit power function suspended sediment load curve are shown on Figure 2.

The impact of the November 2006 data on the suspended sediment load curve can be seen in Figure 2 by comparing the USACE 2008 curve to the Collins 1998 curve. The Collins 1998 curve is based on the 1971-1993 USGS data (Collins, 1998). The UASCE 2008 curve is about three times higher than the Collins curve for discharges over 20,000 cfs, however, the Collins curve is about three times higher for discharges less than 15,000 cfs. The differences between these two curves highlight the uncertainty commonly found in sediment load curves. The uncertainty comes from the scatter in the 1971-1993 data for discharges less than 50,000 cfs and from the limited amount of data available for discharges over 50,000 cfs. Several years of intensive suspended sediment sampling, especially during high discharges, would be necessary to significantly reduce the uncertainty in the sediment load curve. Because of the limitations of the 1971-1993 data, Collins (1998) recommended his curve was suitable for showing general patterns of sediment yields, but not for accurately calculating yields. The USACE 2008 curve is slightly better, because it incorporates the November 2006 high discharge data, but still has a high degree of uncertainty, and should also be used consistent with Collins' recommendation.

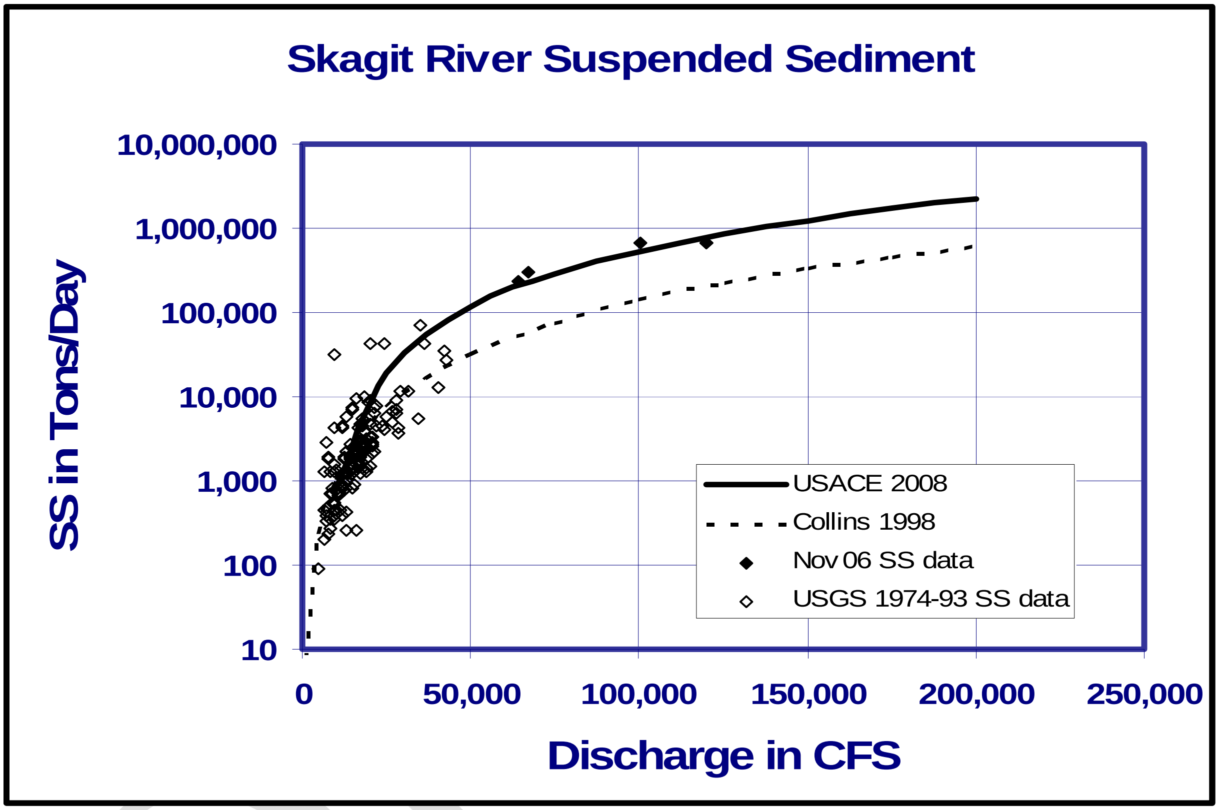


Figure 2. Suspended sediment load curves for the Skagit River at Mount Vernon. The difference between the two suspended sediment load curves is mainly due to the incorporation of the November 2006 data into the USACE 2008 sediment load curve analysis.

The USACE 2008 suspended sediment load curve was combined with the daily discharges from the USGS gage at Mount Vernon to compute daily and annual suspended sediment yields for the period 1940-2004. This method resulted in an average annual suspended sediment yield of 3.8 million tons (mt)/yr, or 2.8 million cubic yards (mcy)/yr. This is approximately four times the annual yield estimated by the basin sediment budget method. It is also more than double the 1.7 mt/yr estimated by Collins (1998).

The yearly suspended sediment yield results are shown on Figure 3. There is over an order of magnitude difference between the highest and lowest annual sediment yields. The highest annual yield was in WY 1991, 10.4 mt, and the lowest in WY 2001, 0.4 mt. Those two years are also the highest and lowest recorded runoff years during the period of record.

The calculated daily sediment yields were analyzed to determine how the average annual sediment delivery was distributed over the range of discharges. The results of that analysis are summarized in Table 3. The importance of floods to the sediment budget is evidenced by the highest 1 percent of the daily water discharges producing 21 percent of the average annual suspended sediment yield. It is also notable that the nine days in the period of record with discharges over 100,000 cfs are estimated to have produced a total of over 5 mcy of suspended sediment, about 3 percent of the period of record total. The highest single day was November 25, 1990, with calculated sediment yield of over 0.75 mcy.

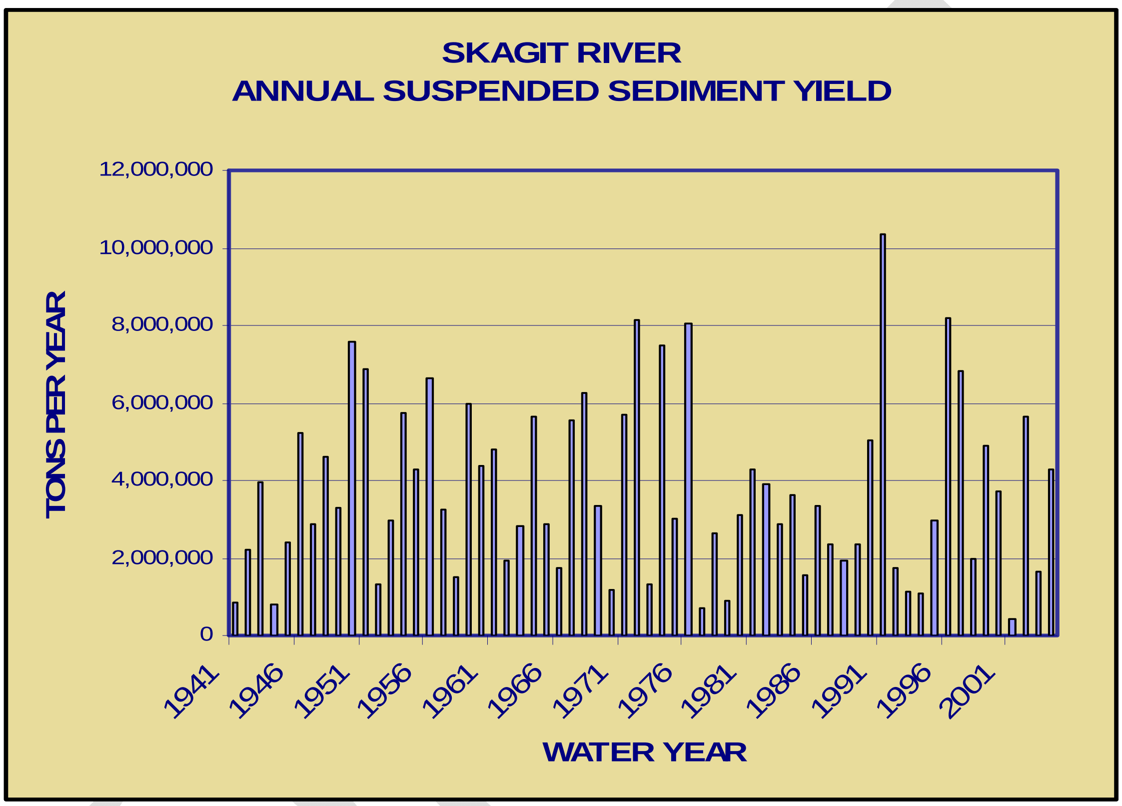


Figure 3. Calculated annual Skagit River suspended sediment discharges at Mount Vernon.

Table 3. Skagit River suspended sediment yield relative to magnitude and duration of water discharge.

Percent of Time Discharge is Exceed	Minimum Discharge in CFS	Suspended Sediment Yield		Percent of Average Annual Sediment Yield
		Tons/Year	MCY/Year	
1	50,600	780,000	0.6	21
10	27,200	2,400,000	1.8	63
50	14,500	3,700,000	2.7	98

The grain size distribution for the suspended sediment also has a large amount of scatter in the data, but shows a general trend of becoming finer as the discharge increases. With reference to Table 3, the sand percentage for the discharges between 14,500 and 27,200 cfs is estimated to be 70 percent, with an observed range of 3-90percent; for discharges between 27,200 and 50,600 cfs, the sand portion is estimated to be 50 percent, with a range of 40-77 percent; and for discharges exceeding 50,700 cfs, the sand portion is estimated to be 40 percent, with a range of 30-55 percent. Combining those percentages with the suspended sediment yields in Table 3 results in an average annual sand discharge at Mount Vernon of approximately 1.4 mcy/yr, which is half of the total annual suspended sediment yield. The remaining 50 percent of the suspended sediment yield at this location is silt and clay.

Bedload is a significant, but unexplored process in the Skagit River. Bedload has not been measured and its contribution to the annual sediment budget of the river is unknown. Bedload is also an important geomorphic process, as it is capable of moving the gravel and cobbles found in the riverbed.

3.4 Sediment Budget Conclusions

The average annual sediment yield estimates have a wide range; from 0.6-0.7 mcy/yr (0.8-0.9 mt/yr) for the basin geomorphic sediment budget, up to 2.8 mcy/yr (3.8 mt/yr) for the fluvial hydraulic sediment budget. Both methods have a substantial amount of uncertainty in their estimates. The basin sediment budget currently does not account for all erosion processes active in the basin, nor have all the sub-basins been examined. The uncertainty in the fluvial sediment budget comes from relying on data with a fair amount of scatter and few high discharge measurements. The two methods take very different approaches and produce very different sediment budgets, however, both methods identified the importance of intense, short-term events (mass failures and floods) to sediment production.

The importance of intense, short-term events suggests that some of the difference in yields estimated by the two methods may be on account of the October 2003 storm. That storm was an unusually large storm that washed out roads and bridges in the upper Skagit River basin. The full extent of sediment producing disturbances, such as landslides, debris flows and bank erosion, caused by the storm has not been quantified. The November 2006 storm was the first of comparable magnitude following the October 2003 storm. It is possible that sediment sources created in 2003, and therefore not accounted for in the basin sediment budget, could have contributed to the suspended sediment measured in November 2006. This could have raised the 2006 suspended sediment concentrations to levels higher than pre-2003 levels. If this is the case, sediment yields can be expected to decline toward pre-2003 levels in a few years as the new sediment sources are depleted. Continued suspended sediment monitoring during large storms would be required to identify any long-term trends in sediment yields.

The estimated 0.8-3.8 million tons/yr sediment yield equates to 530-2,500 tons/ sq mi/yr from the 1,500 sq mi of the Skagit basin that is not regulated by dams. This Skagit River range is consistent with the regional range of 830-2,500 tons/sq mi/yr of sediment from glacially-fed rivers compiled by R2 Resources (2004) for Puget Sound Energy.

A comprehensive, long-term monitoring program of watershed erosion processes and suspended sediment measurements would be required to refine the Skagit River sediment budgets and reduce the level of uncertainty. Such a comprehensive sediment analysis is not considered necessary to evaluate the potential impacts of the measures under consideration in this Flood Damage Reduction Feasibility Study. No actions are being contemplated within the sediment source areas of the upper watershed and only the peaks of the flood hydrographs might be diverted or stored by flood control measures under consideration.

4.0 FLUVIAL GEOMORPHOLOGY

The Skagit River can be divided into five geomorphic reaches. In the upper basin the Skagit River occupies the narrow, steep-walled canyon upstream of the Cascade River. The middle river extends from the confluence of the Cascade River downstream to Sedro-Woolley. As the valley floor widens through this reach and the channel becomes more sinuous and complex. The lower river runs from Sedro-Woolley to the estuary. The lower river is confined to a single channel with hardened banklines. Downstream of Mount Vernon, the river splits into two distributary estuary channels, before discharging into Skagit Bay on Puget Sound.

4.1 Upper River

The upper reach covers the channel upstream of the Cascade River (RM 78). The channel form in this reach is controlled by the steep North Cascade Mountain geology. Most of the channel upstream of Gorge Dam (RM 97) is submerged by reservoirs. From Gorge Dam downstream to the Cascade River (RM 78), the river flows freely through a narrow bedrock confined channel in a series of rapids and deep pools. The Skagit River has a slope of 10 ft/mi in this lower reach. The riverbed is composed of bedrock, boulders, cobbles and gravel.

4.2 Middle River

The middle reach extends from the Cascade River downstream to near Burlington (approximately RM 19). This is the most active stretch of the river, with complex channel forms and only intermittent bank protection. The lower part of this reach was described by Pentec (2002) in the Phase 1 geomorphology report for this Skagit River Flood Damage Reduction Feasibility Study.

In this reach the river flows on a mountain valley floor that gradually widens in the downstream direction. The Cascade and Sauk rivers contribute large sediment loads to this reach of the Skagit River. The riverbed in the Cascade-Baker river reach is composed of boulders, cobbles, and gravel. The stream gradient falls from over 6 ft/mi upstream of Concrete to about 2 ft/mi upstream of Sedro-Woolley (approximately RM 23) and then steepens again to around 5 ft/mi at the downstream end of the reach. The bed becomes finer downstream and is mostly gravel with some sand near Sedro-Woolley. The floodplain soils tend to be sand, silt and clay.

The channel begins to meander and becomes more complex downstream of the Sauk River. Side channels become more frequent as the valley widens and the slope flattens between Hamilton and Sedro-Woolley. There are numerous side channels, oxbows and overbank erosion scars created during large floods of the past. Some meanders have been cutoff. Bank protection is intermittent throughout the entire reach, generally occurring along Highway 20 or adjacent to riverside communities.

Pentec (2002) mapped the 1894 and 1998 river channels downstream of Hamilton and identified a highly active channel migration zone approximately 2 miles wide between Hamilton and Sedro-Woolley. Between RM 24 and RM 19 the river occupies an active channel 1,000-1,600 feet wide, but there is no active meander zone. The riverbanks consist of alluvial materials and are generally 20-30 ft from top of bank to the submerged toe of the slope. Downstream of RM 22, the river had a meander zone approximately 2 miles wide in 1894, but there is currently little channel migration as most of the banks are now protected by revetments.

Large woody debris (LWD) is common in the middle river reach (Pentec, 2002). LWD exists along the shoreline, both in water and as recruitable trees. Concentrations of LWD can be found at the upstream end of islands, such as those at RM's 35 and 58, or the entrance to side channels, such as at RM 64.

Changes in bed elevations in between RM 19.4 and 22.4 were analyzed by WEST (2000) by comparing 1975 and 1999 channel cross-sections. Those results, listed in Table 4, show a bed elevation rise of 2 ft or more at 5 of 6 cross-sections. Those increases must be viewed with caution as most of those cross-sections are located in a river reach that has a wide channel and an unstable alignment that make cross-section comparisons difficult. Those channel conditions are however consistent with what would be expected in a depositional river channel, which is what the cross-sections indicate and what USACE reported in 1978.

Table 4. Skagit River Bed Elevation Changes for Selected Cross-sections Surveyed in 1975 and 1999 (WEST, 2000).

Reach	River Station (miles)	Change in Thalweg (feet)	Average Change in Bed (feet)
Skagit R.	19.4	2.8	2.4
Skagit R.	20	-0.7	2.7
Skagit R.*	20.9	4.2	4.0
Skagit R.*	21.6	-1.1	1.9
Skagit R.*	21.9	-1.6	2.4
Skagit R.*	22.4	-6	-2.8

* Cross sections are questionable, they do not appear to be surveyed at the same locations.

4.3 Lower River

The lower river runs from RM 19, slightly upstream of Burlington, downstream to RM 8, where the river splits into the North and South Forks. Within this reach the river occupies a single channel, typically 600-700 ft wide with 20-30 ft high banks. This reach has been extensively modified with levees, bank protection, and dredging over the past 100 years or more. Levees line both sides of the river, with minimal setback distances. The banks are continuously armored with riprap. No eroding banks were observed within this reach and the river occupies essentially the same location as 100 years ago (Pentec, 2002).

There is a limited amount of LWD in this reach. Most of the LWD that exists are individual pieces scattered along the riverbed. LWD does collect at bridge piers, especially during floods. Flood fight efforts usually remove the LWD from the bridge piers. There are a few small, isolated sources of LWD along the banks.

In this reach, the riverbed material changes from gravel to sand. Upstream of RM 17, the riverbed is a mixture of gravels and coarse sand. Downstream of RM 17, the bed generally consists of medium and coarse sands, with very little gravel or fine (silt or clay) material.

Bed elevation changes in this reach can be analyzed spatially and temporally. The cross-section analysis discussed in the Middle River reach was also done for this reach. That analysis gives an indication of the erosion/deposition trends through the reach between 1975 and 1999. There are also cross-section measurements taken by the USGS at the Mount Vernon stream gage. Those cross-sections were surveyed more frequently and offer a means of analyzing short-term bed changes at the gage at RM 15.8.

To evaluate bed elevation changes, the Corps had WEST Consultants (2000) compared Skagit River cross-sections surveyed in 1975 and 1999. The results of that comparison are summarized in Table 5. The WEST findings showed that the majority of the locations in the lower river reach have aggraded, and only two have degraded. There was wide variation in the amount of aggradation, with increases ranging from 0.1 to 3.7 ft. The average increase in overall bed elevation was 1.4 ft for the 25 year time period.

Table 5. Skagit River Bed Elevation Changes for Selected Cross-sections Surveyed in 1975 and 1999 (WEST, 2000).

Reach	River Station (miles)	Change in Thalweg (feet)	Average Change in Bed (feet)
Skagit R.	10.1	10.6	3.7
Skagit R.	10.6	4.3	0.9
Skagit R.	11.2	2.4	0.6
Skagit R.	11.7	5.2	1.8
Skagit R.	12.4	-1.5	1.5
Skagit R.	12.9	3.9	1.0
Skagit R.	13.1	1.9	1.6
Skagit R.	13.8	-0.2	1.3
Skagit R.	14	-1.3	2.2
Skagit R.	15	-2.2	0.1
Skagit R.	15.1	3.3	2.3
Skagit R.	15.9	1.6	2.6
Skagit R.	16.2	2	0.2
Skagit R.	16.6	2.4	2.4
Skagit R.	16.8	2.1	2.2
Skagit R.	17	-1	-1.5
Skagit R.**	17.5	1.7	-6.0
Skagit R.	17.9	4.2	2.0
Skagit R.	18.5	3.2	1.2
Average***		2.3	1.4

** Cross-section is questionable, it do not appear to have been surveyed at the same locations.
*** Does not include the cross-section 17.5 that is questionable.

The 1975-1999 aggradation rates can be roughly compared to the Corps' 1978 aggradation estimate to identify depositional trends. The 1978 USACE report gave an average infill rate downstream of Sedro-Woolley of 33.4 cu yds/sq mi/yr between 1931 and 1978 (USACE, 1978). That rate equates to an estimated total of 2.5 mcy of deposition in the channels downstream of Sedro-Woolley over the 47 year period. Exact dimensions of those channels are not known, but using approximations based on recent surveys, the 2.5 mcy could have produce an average bed elevation increase of around 1 ft (0.02 ft/yr) for all the main channels downstream of Sedro-Woolley. The 1.4 ft (0.06 ft/yr) of average aggradation in the 25 years between 1975 and 1999 in this lower river reach is approximately three times the estimated rate derived from the Corps' 1978 information. Reasons for the apparent increase in aggradation rate are unknown. Several factors could have contributed to the increase, including inconsistencies in the river reaches, the termination of sand and gravel mining in the 1980s, the large floods in 1991, or increased sediment yields from upstream.

The USGS routinely surveys the riverbed when measuring Skagit River discharges at Mount Vernon (RM 15.8). Those surveys provide an opportunity to evaluate bed elevation changes occurring at an annual or shorter time frame at that location. For this analysis the USGS (Mastin, 2006) provided survey data and average bed elevations for selected surveys between 1960 and 2005. To complement the bed elevation analysis, the stage/discharge curves for a subset of those surveys were used to evaluate water surface elevation changes for 10,000 cfs. The average bed and water surface elevations are shown in Figure 4.

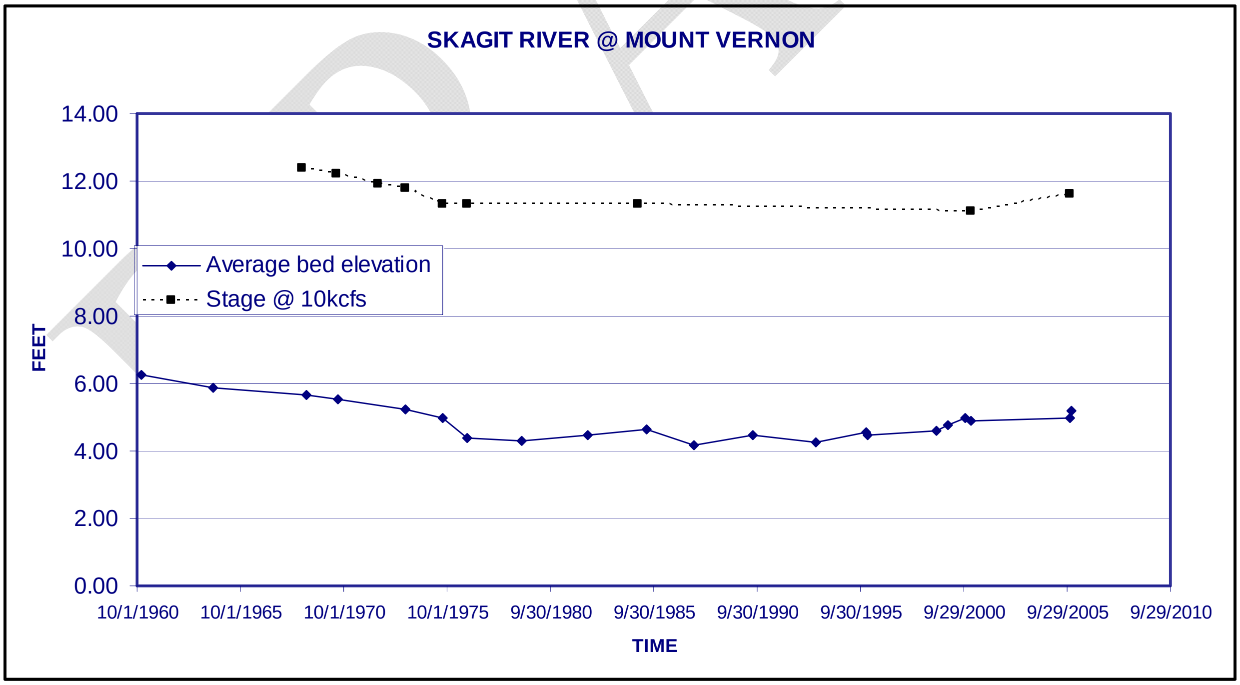


Figure 4. The average bed elevation and stage for 10,000 cfs are shown for the USGS streamgaging station on the Skagit River at Mount Vernon.

The record can be divided into three separate time periods based on the bed elevation change trends. There is a persistent decline in the average bed elevation from 1960 to 1976 that totals -1.9 ft. From 1976 to 1996 the bed elevations show small fluctuations that resulted in an overall rise of only 0.1 ft. The record does not show any unusual bed elevation change as a result of the high discharges and sediment yield in WY 1991. Then from 1996 to 2005 the average bed elevation rose 0.7 ft. This time period includes WY 2001, the lowest sediment yield year during the period of record. USACE (1978) reported a very similar -1.6 ft change in bed elevation at this location between 1959 and 1976. USACE also reported that the decline had been proceeded by an increase of 2.1 ft between 1940 and 1959.

The bed elevation changes and calculated sediment yields were visually compared to see if there was a relationship between bed changes and the magnitude or timing of the sediment yields. In Figure 5 the bed elevation changes at the USGS Mount Vernon gage are plotted along with the 3-year running average sediment yield for 1943-2005. A 3-year running average was used to smooth the sediment yield data and make it easier to identify temporal trends. There does not appear to be any relationship between the bed elevation changes and sediment yields.

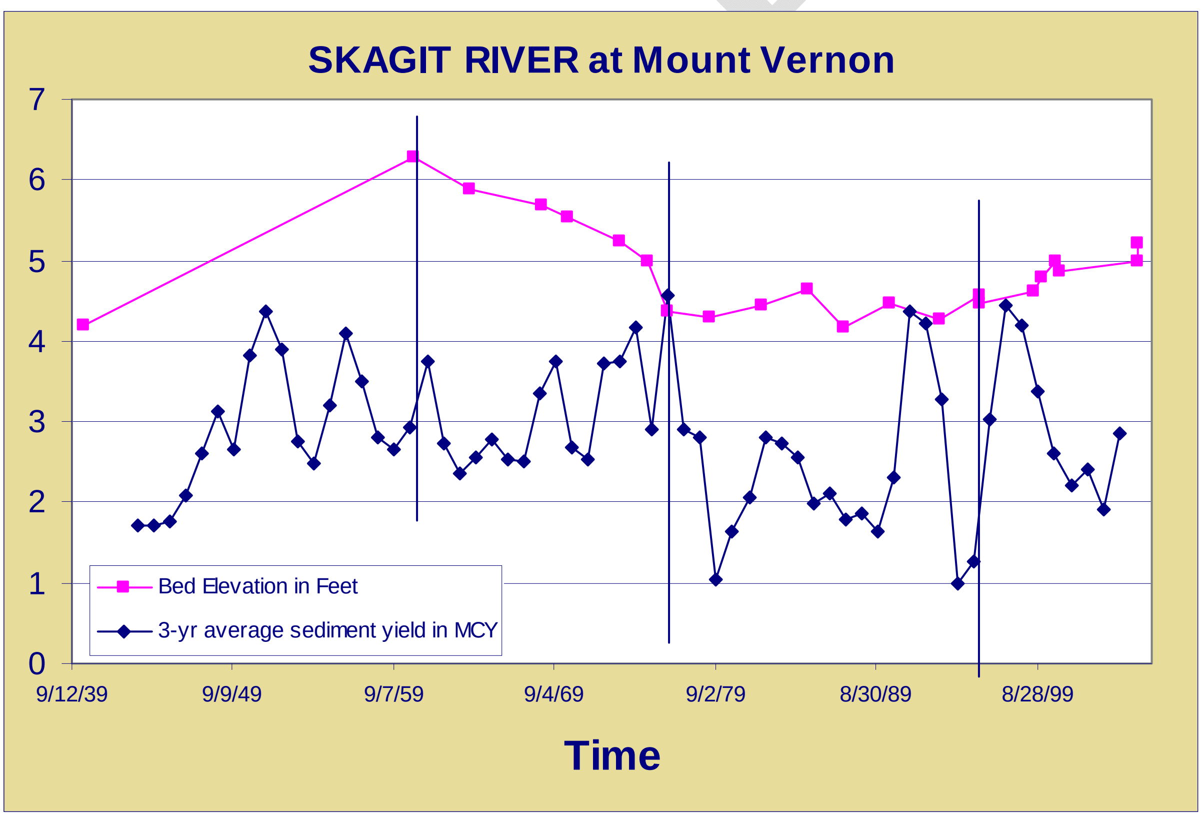


Figure 5. Bed elevations and sediment yield for the Skagit River at Mount Vernon.

Another interesting comparison is the bed elevation changes measured at the gaging station site by the two different surveys. During the 1975-1999 time period, the Corps' two cross-sections surveyed at the USGS station showed a -1.5 ft change, while the USGS data shows a change of -0.3 ft. While these differ in magnitude, they both indicate a decline in average bed elevation at this site. These declines are in contrast to the cross-section surveys overall average bed elevation change of +1.4 ft. However, the relatively consistent result at the gaging site does suggest the broader, overall depositional trend shown by the cross-sections is also reliable.

4.4 Estuary Channels

This reach includes the North and South Forks from their split with the main stem at RM 8, downstream to Skagit Bay in Puget Sound. The flows in these channels are influenced by both river discharges and tidal flows. The mean tide range of 12 ft in Skagit Bay generates large variations in the magnitude and direction of flows in the estuary channels (Philip Williams and Associates (PWA), and Skagit River System Cooperative (SRSC), 2004). The estuary channels and floodplains have been extensively altered by human activities (Collins, 1998).

4.4.1 North Fork. The North Fork carries about 60 percent of the Skagit River discharge. Upstream of about RM 2, the channel is confined by levees and high ground. Channel widths are typically 350-500 ft and the banks are around 15-20 ft high. Bed material samples identified a medium/coarse sand bed, with the D_{50} decreasing from 0.6 mm near RM 9 to 0.3 mm near the mouth (Pentec, 2002).

Bed elevation changes were measured by comparing North Fork cross-sections surveyed in 1975 and 1999 (WEST, 2000). The findings showed that the majority of the stations have aggraded, and only one had degraded. The results of that comparison are summarized in Tables 6. The North Fork had an average increase in overall bed elevations of 1.6 ft. This trend is consistent with that found in the South Fork and lower river channels.

Table 6. North Fork Skagit River Bed Elevation Changes for Selected Cross-sections Surveyed in 1975 and 1999 (WEST, 2000).

Reach	River Mile	Change in Thalweg (feet)	Average Change in Bed (feet)
NF Skagit R.	4.5	1.6	2.3
NF Skagit R.	4.75	4.2	2.8
NF Skagit R.	5.5	3	2.6
NF Skagit R.	6.2	10.4	1.1
NF Skagit R.	6.6	3	1.9
NF Skagit R.	7.2	0.5	0.8
NF Skagit R.**	7.33	3.9	2.9
NF Skagit R.	7.9	2.4	1.3
NF Skagit R.	8.1	2.5	1.1
NF Skagit R.	8.29	-3.7	-0.7
NF Skagit R.	8.85	2.2	2.3
Average***		2.6	1.6

** Cross sections are questionable, they do not appear to be surveyed at the same locations.
*** Does not include cross sections that are questionable.

Bankline vegetation along the North Fork generally consists of narrow bands of small trees and scrubs with isolated patches of larger trees, such as those near RM 4. There are no significant sources of LWD. LWD is scarce within most of the North Fork channel. The only significant accumulations of LWD occur at the upstream ends of the islands located near the mouth of the channel.

Two large channels, Dry Slough and Brown's Slough, used to branch off from the North Fork and flow south across Fir Island, but were cut off when the levees were built in the early 1900's. Both channels now have tide gates to control flows. Downstream of RM 2 several small channels divert flow into Skagit Bay before the North Fork enters the Bay at McGlinn Island.

4.4.2 South Fork. The South Fork has a more complex channel network than the North Fork. It occupies a single channel that varies from 400-900 feet wide downstream to RM 5.5 and then branches into three channels, Freshwater Slough, Steamboat Slough, and the main South Fork channel. Each of these channels branch into multiple channels as they approach Skagit Bay, creating a network of interconnected channels and islands. The South Fork channel complex carries approximately 40 percent of the total Skagit River flow.

Bed elevation changes along the single-channel reach of the South Fork were measured by comparing cross-sections surveyed in 1975 and 1999 (WEST, 2000). The results of that comparison are summarized in Table 7. The findings showed that all of the stations have aggraded. The South Fork had an average increase in overall bed elevations of 1.0 ft, with a range of 0.4 to 1.8 ft. This trend is consistent with that found in the North Fork and lower river channels. No bed elevation comparison was made downstream of RM 5.8.

Table 7. North Fork Skagit River Bed Elevation Changes for Selected Cross-sections Surveyed in 1975 and 1999 (WEST, 2000).

Reach	River Station (miles)	Change in Thalweg (feet)	Average Change in Bed (feet)
SF Skagit R.	5.8	0.3	1.8
SF Skagit R.	6.3	-0.4	0.9
SF Skagit R.**	6.95	4.4	0.1
SF Skagit R.	7.8	-0.5	0.5
SF Skagit R.	8.75	1.9	1.4
SF Skagit R.	9.25	-2.4	0.4
Average***		-0.2	1.0

** Cross sections are questionable, they do not appear to be surveyed at the same locations.
*** Does not include cross sections that are questionable.

The average bed material size in the South Fork is 0.56 mm, coarse sand. Similar to the North Fork, the D_{50} of the bed material decreases in the downstream direction, ranging from 0.8 mm near RM 8 to 0.3 mm near the mouth (Pentec, 2002).

The South Fork, while also constrained by levees, does not have continuous bank protection along its banks. There are expanses of riparian forests that provide a local supply of LWD to the channel. LWD is present through much of the main South Fork channel and the upstream end of Freshwater Slough. In several locations, LWD has been deposited on mid-channel bars. Pentec (2002) suggested the upstream growth of these bars is influenced by the accumulations of LWD.

4.5 Nearshore

The Skagit River nearshore covers an area approximately 8 miles long, north to south, and 2.5 to 5 miles wide, with shallow tidal flats extending nearly to Whidbey Island. This nearshore area could be divided into sub-areas based on any number of factors, such as fish or plant habitats, islands, or tidal channels. For this geomorphic analysis, it makes sense to follow Collins' (1998) lead and separate it into the North Fork Delta, South Fork Delta and Fir Island Delta. Each delta includes several habitat features, including shallow tidal flats, eelgrass, marshes, blind tidal channels, and distributary channels that are influenced by the presence or lack of river hydraulic and sediment processes. The deltas are separated from Whidbey Island by a deep channel along the eastern edge of the island.

4.5.1 North Fork Delta. The active North Fork Delta covers approximately 4,500 acres, generally west and south from the main channel mouth. The western edge of the delta is only 0.5 mile from Whidbey Island. The southern edge is not a distinct boundary, but is located in the vicinity of Craft Island, about 2 miles south of the main channel mouth. The northward expansion of the delta is cutoff by a rock jetty that is just over a mile long and runs between McGlinn and Goat islands. The jetty separates the North Fork Delta from Swinomish Channel and restricts sediment movement to the north.

Since the completion of the Skagit River levee systems, sediment discharges have been concentrated at the mouths of the North and South forks. Sand from the Skagit River is deposited throughout the 4,500 acres of the North Fork Delta, while silts and clays are transport beyond the delta. Sediment cores taken by the USGS on the delta have found several feet of recent sand deposits overlay older mud deposits (Grossman, 2008). Aerial photos available on Google Earth and Microsoft Live Search Maps, indicate the main North Fork channel delivers sediment to the northern part of the delta and the distributary channels supply sediment to the southern half of the North Fork Delta. Deposition appears to be greatest near the mouth of the main channel, as there are networks of small channels flowing west and south away from the mouth. The deposition has created new marsh habitat in the delta, replacing some of the marsh lost due to levees and agricultural development.

Marsh islands cover approximately 640 acres along the northeast edge of the delta. Collins (1998) indicated these marshes expanded slowly between 1889 and 1937, but then grow more rapidly between 1937 and 1991. The increased growth rate was attributed to an increase in the portion of Skagit River flow, and presumably sediment, carried by the North Fork channel. PWA and SRSC (2004) determined that these marshes had grown by 229 acres, an increase of over 50 percent, between 1954 and 2002. Pentec (2002) also concluded the islands were growing, based on observations of river sediments deposited around islands. The islands are separated by distributary channel from the North Fork Skagit River. The islands also contain a limited amount of blind tidal channels, dead-end channels that are formed by and convey tidal flows. Collins, found blind tidal channels made up only 4% of the marsh area. This small amount of the blind tidal channels is due to the small size of the islands (PWA and SRSC, 2004).

Sullivan Slough is a 3,000 acre marsh, located just north of the mouth of the river. Collins (1998) indicates the slough was once a major distributary channel of the Skagit River. The area was highly modified and the marsh area reduced between 1874 and 1940. Sediment from the Skagit River may have contributed to the aggradation during that period, but the current drainage pattern in Sullivan Slough indicates the area is now an independent blind slough complex.

Other interesting features of the delta include the presence of LWD around the islands and patches of eelgrass in the southern corner of the North Fork Delta (Grossman, 2005). The eelgrass is located far from the main channel mouth, where deposition would be the least. The USGS, in partnership with other agencies (including the SRSC), is studying the impact of sediment deposition on the eelgrass.

4.5.2 South Fork Delta. The active South Fork Delta is larger and more complex than the North Fork Delta. The South Fork Delta covers an area of around 13,000 acres in Skagit Bay. It is bordered on the south by the West Pass of the Stillaguamish River. The tidal flats extend 4 miles west, to approximately 1.2 miles from Whidbey Island. The northern boundary is not clearly defined, but generally runs west from the mouth of Freshwater Slough.

As with the North Fork Delta, sand from the Skagit River has been deposited over the South Fork Delta and the silts and clays transported away. Recent sand deposits overlay older mud deposits (Grossman, 2008). Deposition was most rapid between 1889 and 1937 when most of the marshes formed (Collins, 1998). Aerial photos available on Google Earth and Microsoft Live Search Maps, suggest the highest deposition is currently occurring at the mouth of Freshwater Slough, where an unstable, non-vegetated, low-tide island complex has developed.

Marsh islands cover approximately 2,000 acres, extending over a mile up along the main distributary channels. Collins (1998) indicated these marshes expanded most rapidly between 1889 and 1937. Growth continued after 1937, but at a much slower rate. PWA and SRSC (2004) determined that these marshes had grown by 518 acres between 1954 and 2002. This is nearly twice the rate of marsh growth as was observed in the North Fork Delta during the same time period. This higher growth rate was produced despite the South Fork carrying less water and sediment than the North Fork. The South Fork Delta islands are large and have well developed drainage networks. Collins (1998) estimated that blind tidal channels made up 7 percent of the marsh area in 1991. LWD is very sparse on the marshes, even along the shorelines.

Delta maps (Grossman, 2005) show fragmented patches of eelgrass occur within an approximately 3,000 acre area on the west side of the delta. Grossman concluded that the eelgrass fragmentation has been caused by sediment deposition following the concentration of Skagit River discharges in the North and South Fork channels.

4.5.3 Fir Island Delta. The Fir Island Delta is located between the North and South Fork deltas. The Fir Island Delta covers an area of around 5,000 acres in the center of Skagit Bay. The tidal flats extend 2.5 miles west, to about 0.75 miles from Whidbey Island. This area does not have any significant sources of freshwater or riverine sediment. The tidal flats are covered with sand that originates from the North or South forks.

There are approximately 500 acres of marsh in the Fir Island Delta. The marsh is located on a narrow, 4 mile stretch of the mainland shoreline. About 3 percent of the marsh is composed of blind tidal channels. There are levees along the landward side that have cut-off large portions of the marsh and the associated channel network (Collins, 1998). The marsh area has declined due to erosion and land subsidence; both processes are aggravated by the lack of sediment from the river. Collins (1998) estimated about 200 acres of erosion between 1937 and 1991, and PWA and SRSC (2004) measured 160 acres of marsh loss between 1954 and 2002. Most of the Fir Island shoreline contains accumulations of LWD.

Delta maps (Grossman, 2005) show the Fir Island Delta contains nearly 1,500 acres of continuous eelgrass habitat along the western edge of the delta. Apparently because of the lack of distinct channels and sediment deposition, this patch of eelgrass has not been fragmented as has happened to the eelgrass habitat in North and South deltas.

5.0 DATA GAPS

There are numerous data gaps that would have to be filled to thoroughly define the Skagit River's sediment budget and fluvial geomorphology. Some of the most important investigations required to fill those data gaps are:

- Inventory all significant erosion processes and sediment sources active in all sub-basins
- Identify the gradation of sediments produced in each sub-basin
- Monitor suspended sediment and bedload transport in the main stem and major tributaries
- Continue to re-survey channel cross-sections every 10 years or so
- Refine geomorphic analysis using a time series of aerial photographs
- Improve the understanding of relationships between subbasin sediment production and channel aggradation through watershed sediment yield modeling and sediment transport modeling.

These investigations would be very expensive and would require years to complete. Such a comprehensive investigation is not considered necessary to evaluate the potential impacts of the measures under consideration in this Flood Damage Reduction Feasibility Study. However, detailed studies are being conducted by other groups, such as the SRSC, USGS, and Skagit Watershed Council, to support habitat restoration efforts in the lower Skagit River, estuary, and nearshore. The results of those studies will be reviewed as they become available and the results considered in evaluating potential project impacts.

6.0 SEDIMENTATION CONCLUSIONS

Based on the results of the above sediment budget and fluvial geomorphology analyses, the Skagit River's sediment regime can be fairly well defined. There remains some uncertainty about precise annual values, but long-term trends are clear.

The Skagit River channel is fairly stable with the most migration occurring in the middle reach. Channel alignment in the upper basin is controlled by natural geology, while the lower river and estuary channels are controlled by levees and bank protection. The middle reach has only intermittent bank protection and the active migration zone is up to 2 miles wide. The estuary and nearshore islands are growing, but the Fir Island shoreline is eroding.

The average annual sediment yield at Mount Vernon is in the range of 0.6 to 2.8 mcy/yr. The major sources of sediment are the Cascade and Sauk rivers. Approximately half the basin does not contribute sediment because the sediment is stored in reservoirs. Large storms, those with daily discharges above 50,000 cfs, are a major factor in sediment production, causing upper basin land disturbances and producing an estimated 21 percent of the average annual sediment yield.

Upstream of RM 17, the Skagit riverbed is composed of gravel, cobble, and boulders. Downstream of RM 17 the riverbed and nearshore delta bottom are mainly sand. The 2.8 mcy/yr annual suspended sediment yield at Mount Vernon is composed of approximately 50 percent sand, 50 percent silt and clay. Most of the sand, and all the silt and clay are transported through the lower river and into Skagit Bay.

Since 1931, there has been a consistent long-term trend of sediment deposition in the channels downstream of Sedro-Woolley. This has resulted in an overall average bed elevation increase of approximately 2 1/4 ft since 1931. The bed upstream of RM 15.8 appears to be rising slightly faster than the overall average. Sand deposition has also been occurring in the estuary and on the delta. Islands and marsh habitat have been growing at the mouths of the North and South Forks.

7.0 REFERENCES

Benda, L., and T. Dunne. (1997). “Stochastic forcing of sediment supply to channel networks from landsliding and debris flow”. *Water Resour. Res*., 33, 2849–2863.

Collins, B., 1998, Preliminary Assessment of Historic Conditions of the Skagit River in the Fir Island Area: Implications for Salmonid Habitat Restoration, Prepared for Skagit System Cooperative, August 1998.

Gardner, C.A., K.M. Scott, C.D. Miller, B. Myers, W. Hildreth, and P.T. Pringle, 1995, Potential Volcanic Hazards from Future Activity of Mount Baker, Washington: U.S. Geological Survey Open-File Report 95-498.

Grossman, Eric, 2005, Deltaic Habitats in Puget Sound - Natural Verses Human-Related Changes, USGS Sound Waves, January 2005.

Grossman, Eric, 2008, Personnel communications, USGS, Burlington, Washington, January, 2008.

Mastin, M., 2006, Personnel communications, USFS geologist to Karl Eriksen, USACE Seattle District.

Montgomery, D.R., K. Sullivan and H.M. Greenberg. (1998). “Regional test of a model for shallow landsliding”. *Hydrol. Process*., 12, 943-955.

Nichols, R., 2006, Personnel communications, USFS geologist to Karl Eriksen, USACE Seattle District.

Paulson, K., 1997, Estimating Changes in Sediment Supply due to Forest Practices: A Sediment Budget Approach Applied to the Skagit River Basin in Northwestern Washington, Unpublished Masters Thesis, University of Washington, Seattle, Washington.

Pentec Environmental, 2002, geomorphic and Sediment Transport Study of Skagit River Flood Hazard Mitigation Project Skagit County, Washington, Phase 1 Interim Report, Prepared for USACE Seattle District, December 2002.

PWA and SRSC, 2004, An Assessment of Potential Habitat Restoration Pathways for Fir Island, WA, Prepared for the Skagit Watershed Council, Mount Vernon, Washington.

R2 Resource Consultants, 2004, Hydrology and Geomorphology of the Baker and Middle Skagit Rivers - Sediment Transport and Channel Response, prepared for Puget Sound Energy, February 2004.

Swanson, F. J., R. J. Janda, T. Dunne, and D. N. Swanston, 1982, Sediment Budgets and Routing in Forested Drainage Basins, U. S. Department of Agriculture, Pacific Northwest Forest and Range Experiment Station, General Technical Report PNW-141, Portland, Oregon.

USACE, 1978, Skagit River General Design Memorandum Levee Improvements, USACE Seattle District, Seattle, Washington.

Waitt, Richard B., Larry G. Mastin, and James E. Begét, 1995, Volcanic-Hazard Zonation for Glacier Peak Volcano, Washington: U.S. Geological Survey Open-File Report 95-499.

WEST Consultants, 2000, Skagit River Cross Section Comparison and Analysis, Memorandum prepared for USACE Seattle District, Seattle, Washington.