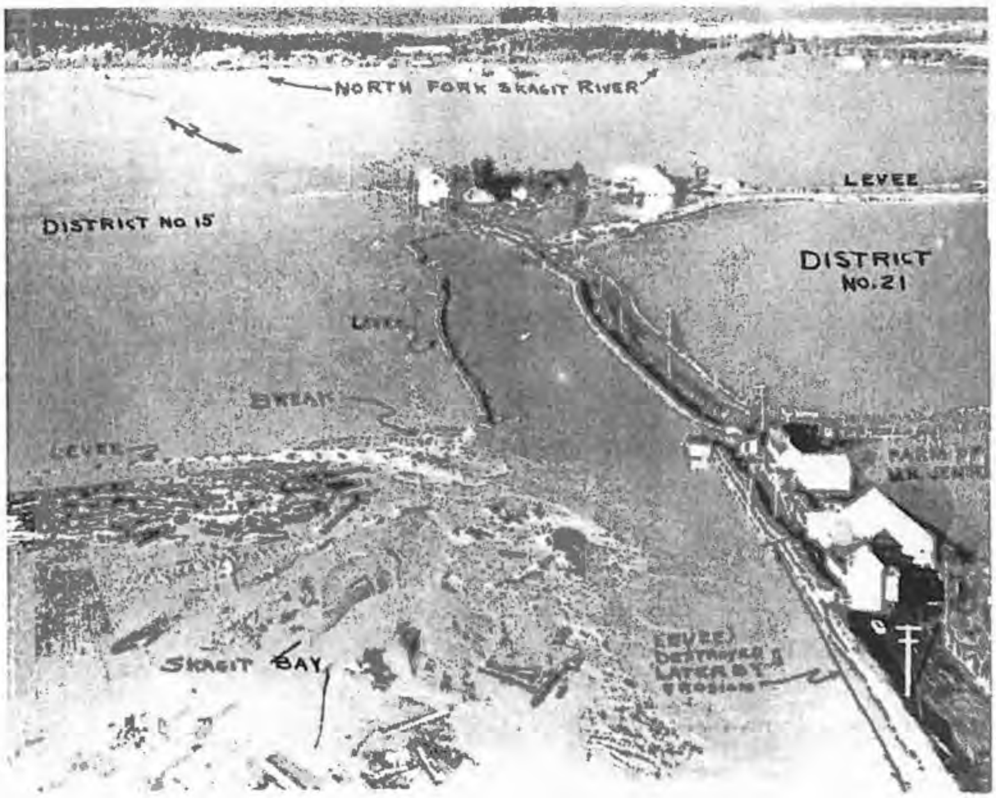
Skagit River Basin
Skagit River Flood Risk Management Study

DRAFT REPORT

HYDRAULIC TECHNICAL DOCUMENTATION



US Army Corps
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HYDRAULICS TECHNICAL DOCUMENTATION
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Appendix A Water Surface Profiles with Levee Failure Points for No Breach Scenario
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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 information documenting hydraulic modeling input to the economic evaluations and analyses will be provided in the project economics reports.

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 is 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 135 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 24) 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 alluvial fan and delta with an east-west width of about 11 miles and a north-south width of about 19 miles. The alluvial fan 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.

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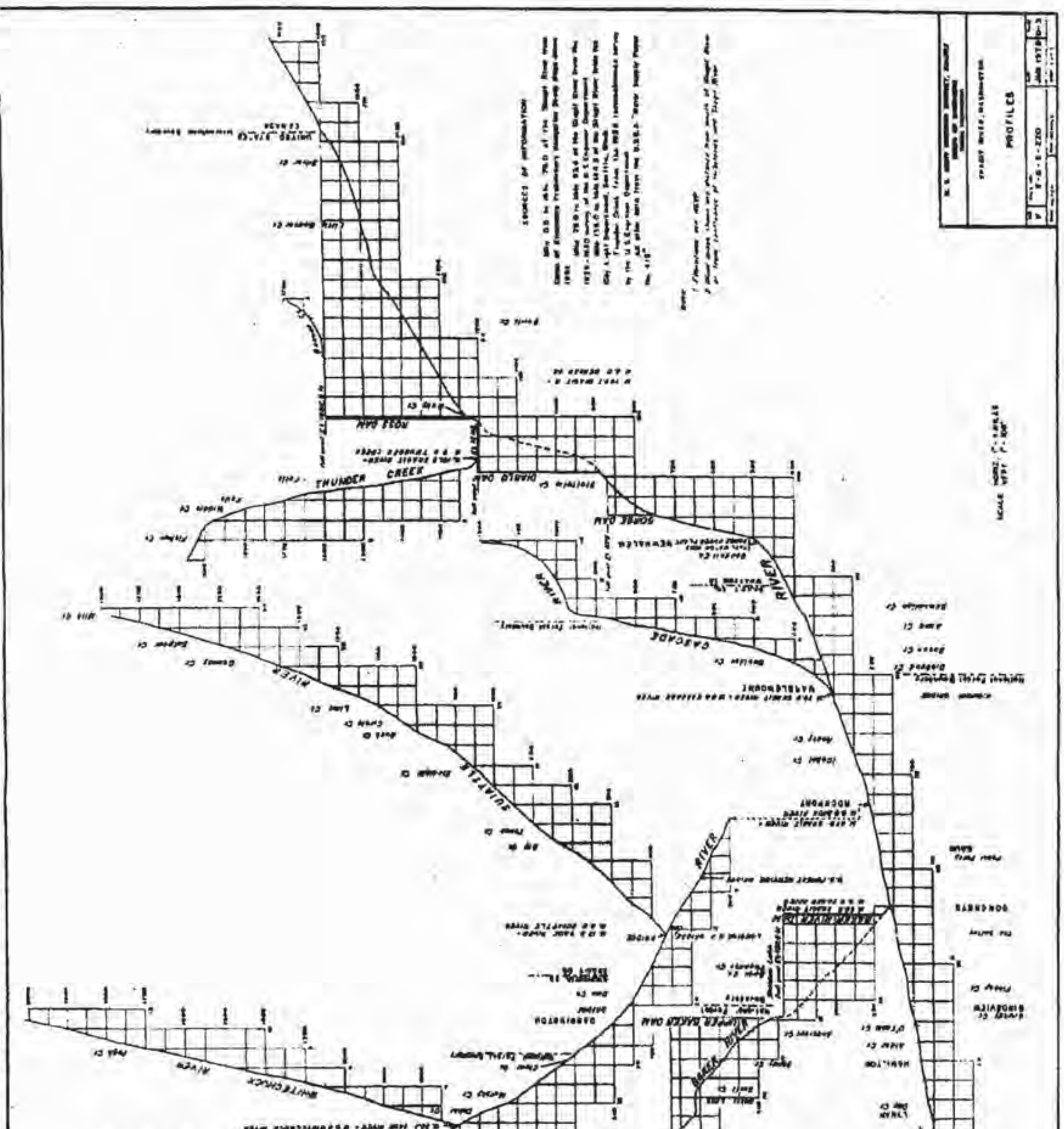
Figure 2. Skagit River and Major Tributary Stream Profiles



1.5 Study History

Hydraulic model development and hydraulic analyses for the Skagit River Flood Risk Management Feasibility Study was conducted by the Seattle District USACE in parallel with similar work by the District for preparation of an updated Flood Insurance Study 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. 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 the present 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 Nookachamps 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 2008 FLO-2D model from the FIS and modification of the FIS model to better meet the computational demands of 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 flood plain features not incorporated into the FIS FLO-2D model.

1.6 Datum

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

1.7 River Stationing

River stationing for the study is understood to have originated from the hydraulic models created for the 1984 Flood Insurance Study, based on distances between surveyed cross-sections measured from topographic maps. However, geo-referencing of the HEC-RAS model, undertaken for the work reported here, found an apparent under-reporting of channel lengths downstream from Sedro-Woolley. For the purposes of this study, the distributary point of the North and South Forks is set at RM 9.48 consistent with previous models. Measuring along the thalweg of the channel in the geo-referenced HEC-RAS model from this point to the SR-9 bridge below Sedro-Woolley gives a distance of 14.02 miles compared with 12.82 miles in previous models. The reason for this inconsistency is not known. To avoid recomputing the river stationing for the river system, river miles in the main body of the report refer to the stationing as used in 2004. However water surface profile plots of the system downstream from Sedro-Woolley provided in the report appendices show distances as determined from the geo-referenced 2011 HEC-RAS model.

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 22.8) 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 4.0.0 and FLO-2D Version 2009, are utilized to represent hydraulic conditions. The steps taken to develop these models will be explained. In addition, detailed 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. The modeling effort is further constrained by limited or incomplete historical hydrologic data. Another possible 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 floods use the "average" case for reservoir regulation unless otherwise stated. For information on how the hydrographs are developed for input into the models, see the Hydrology Technical Documentation.

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 management 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 flood plain flows within FLO-2D.

2.4.1 HEC-RAS Model Development

The computer model HEC-RAS Version 4.0.0, 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 March 2008 HEC-RAS User's Manual.

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 saltwater.

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

<u>Cross Sections</u>

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 of the US Army Corps of Engineers' (USACE) Survey Branch. Floodplain geometry for the 1984 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 1984 study 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 1984 study.

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.

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. 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 will however be considered in the 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.

Storage Areas

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 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 and field investigations.

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.

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 used for the modeling, with class defined by flood magnitude. A "medium" class was used for the 2 through 10-year event, and a "large" class for the 25 through 500-year events. The debris dimensions are based on a review of flood photographs and personal observations from multiple floods. In particular, the BNSF bridge debris loading condition is based on conditions observed during the November 29, 1995 flood. It is assumed, based on past experience, that in-flood debris removal efforts, particularly for large floods, are ineffective. High, average and low debris blockages were estimated for the two bridges in order to estimate the sensitivity of stage to debris blockage. Debris loading dimensions for the Great Northern and BNSF Railroad Bridges are listed in Tables 3 and 4 below. Other bridges were assumed to be free from debris.

Table 3. Debris Blockage Parameters by Flood Class for Great Northern Railroad Bridge

Flood Class	Debris Load	Total Width of Blockage (ft)	Depth of Blockage (ft)	Center Station of Blockage(s) (ft)
Medium	Low	200	5	2762
	Average	200	10	2762
	High	250	15	2762, 2944
Large	Low	200	10	2762
	Average	250	15	2762, 2944
	High	350	15	2762, 2944

Table 4. Debris Blockage Parameters by Flood Class for BNSF Bridge

Flood Class	Debris Load	Width of Blockage (ft)	Depth of Blockage (ft)	Center Station of Blockage (ft)
Medium	Low	200	5	376
	Average	300	10	376
	High	500	20	492
Large	Low	300	10	376
	Average	500	20	492
	High	700	20	502

Levees

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 2010 survey by Woolpert, Inc. 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 and downtown Mount Vernon, where there are no formal levees but where flows can overtop, were extracted from 2009 aerial photogrammetry flown for the Cities of Burlington and Mount Vernon respectively.

Information about the integrity of the levees in the Skagit River system was obtained from geotechnical engineers from the Seattle District of the US Army Corps of Engineers, 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.

Diversion/Impoundment Structures

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.

c. Boundary Conditions

The four primary types of boundary conditions in HEC-RAS are interior, upstream, downstream, and internal. Interior boundary conditions define reach connections and ensure continuity of flow. 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). 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.

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.

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 for description on 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.

Channel Roughness

Stage uncertainty due to uncertainty in channel roughness was determined by varying Manning's "n" values by +/- 20% from the calibrated model values and running the model for the nine hypothetical floods.

Bridge Debris

Stage uncertainty due to bridge debris loading was determined by varying the blockage parameters as shown in Table 3 and 4 and running the model for the nine floods.

Overall Stage Uncertainty
Overall stage uncertainty 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 Economics Report for discussion of index points).

Levee Breach Methodology
A levee breach methodology is 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. under contract to the Seattle District. The locations for the borings were selected in consultation with local dyking 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, reviewed by the Seattle District, shows overtopping as being the most likely probable failure mode at this location. In this case, the PNP and PFP elevations are both taken as the levee crest elevation.

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. To determine the PFPs and PNPs at any location within a specific reach, it was then assumed that the distance from levee crest to PFP or PNP was the same as at the representative boring location for that reach. The PFPs, LFPs and PNPs determined in this way for each lateral structure (i.e. levee segment) within the HEC-RAS model are listed in Table 5

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 the LFP for a given lateral structure. 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 a 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 are determined through consultation with USACE geotechnical engineers. These breach widths are modeled to reach a maximum width of 300 feet 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 flood plain inundation due to levee failure is discussed in the hydraulic appendix to the economics report.

Table 5. 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	44.97	44.97	44.97	44.97
						20.89	DD12	47.39	47.39	47.39	47.39
						19.99	DD12	47.31	47.31	47.31	47.31
						19.47	DD12	46.91	46.91	46.91	46.91
						18.56	DD12	43.91	43.91	43.91	43.91
						17.89	DD12	45.18	45.18	45.18	45.18
17.529	DD17-1L	46.52	45.82	43.77	41.72	17.519	DD12	42.87	42.87	42.87	42.87
17.045	DD17-1L	44.8	44.1	42.05	40	17.049	DD17-1L	44.39	43.69	41.64	39.59
16.779	DD17-1L	44.71	44.01	41.96	39.91	16.777	DD17-1L	44.19	43.49	41.44	39.39
16.599	DD17-2L	44.07	43.77	42.82	41.87	16.58	DD17-2L	44.01	43.71	42.76	41.81
16.29	DD17-2L	43.44	43.14	42.19	41.24	16.28	DD17-2L	43.39	43.09	42.14	41.19
15.899	DD17-2L	42.99	42.69	41.74	40.79	15.88	DD17-2L	41.66	41.36	40.41	39.46
15.08	DD17-2L	40.26	39.96	39.01	38.06	15.09	DD17-2L	40.63	40.33	39.38	38.43
14.58	DD17-2L	40.67	40.37	39.42	38.47	14.59	DD17-2L	40.46	40.16	39.21	38.26
13.98	DD17-3L	39.9	39.3	37.7	36.1	13.99	DD1-1R	40.85	40.45	39.25	38.05
13.78	DD17-3L	38.8	38.2	36.6	35	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

Table 5. (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	26.89	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	23.3	22.4	20.9	19.4	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	24.8	24.7	24.05	23.4	778	DD22-1R	25.8	25.7	25.05	24.4
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	15.9	15.8	15.15	14.5						
339	DD3-1L	17.37	17.27	16.62	15.97						
249	DD3-1L	16.19	16.09	15.44	14.79						

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 account for sediment movement, scour, or deposition. The models assume no exchange with groundwater. The model is intended to adequately reproduce levee breaks and 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 hydraulics for this study in all areas downstream from SR-9 (RM 22.3) except the lower Nookchamps 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 2009.06 FLO-2D Reference Manual.

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 give 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.

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 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. In addition to updating elevations, 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.

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 have 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 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 with 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. The 1990 event had a levee failure during the event that would affect the calibration and the 1975 flood would be significantly affected by the channel changes that are shown in Table 1.

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 6. A more detailed discussion of the observed data for the 2006 event is included in the validation section of this chapter.

Table 6. 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 7.

Table 7. 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 Skagit Bay	0.030-0.038	0.04-0.012

3.2.1 Calibration: October 2003 Event

The model calibration simulates within 0.5-ft, most of the high water marks for the 2003 event from Sedro-Woolley downstream. Table 8 and Figure 3 show the high water marks for the event and the model's simulated maximum water surface profile.

The high water marks upstream of Sedro-Woolley are typically more than a foot above the model simulated water surface for 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 match high water marks in this reach.

The lowest high water marks on the North and South Forks are around 3-ft 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 Mt. 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 Mt. Vernon matched observed flows (Figure 4).

Table 8. October 21, 2003 Simulated vs. Observed High Water Marks

Source	River	Location	High Water Mark	Simulated	Difference
		(River Mile)	(feet NAVD88)	(feet NAVD88)	(feet)
USGS Gage	Mainstem Skagit	78.7	323.0	322.7	-0.3
Skagit County	Mainstem Skagit	59.65	199.3	200.9	1.6
USGS Gage	Mainstem Skagit	54.1	176.0	172.3	-3.7
Skagit County	Mainstem Skagit	49.75	153.8	151.6	-2.1
Skagit County	Mainstem Skagit	40.18	104.4	103.3	-1.1
Skagit County	Mainstem Skagit	29.90	67.3	65.8	-1.5
Skagit County	Mainstem Skagit	22.78	48.9	47.9	-1.1
USGS Gage	Mainstem Skagit	22.3	45.6	45.8	0.2
Skagit County	Mainstem Skagit	21.6	44.5	45.1	0.6
Skagit County	Mainstem Skagit	19.48	43.5	43.5	0.0
Skagit County	Mainstem Skagit	17.07	40.4	41.2	0.8
USGS Gage	Mainstem Skagit	17.04	40.0	41.0	1.1
Skagit County	Mainstem Skagit	15.89	39.0	38.8	-0.2
Skagit County	Mainstem Skagit	13.03	34.0	34.0	0.0
Skagit County	Mainstem Skagit	12.18	32.0	31.7	-0.3
Skagit County	North Fork Skagit	8.09	25.0	24.7	-0.3
Skagit County	North Fork Skagit	4.42	15.5	12.6	-2.9
Skagit County	South Fork Skagit	5.8	19.7	19.6	-0.1
Skagit County	South Fork Skagit	3.52	14.1	11.1	-3.0
USGS Gage	Sauk River	5.40	288.7	289.3	0.6

Figure 3. HEC-RAS Simulated Water Surface Profile and Observed High Water Marks for October 2003 Event.

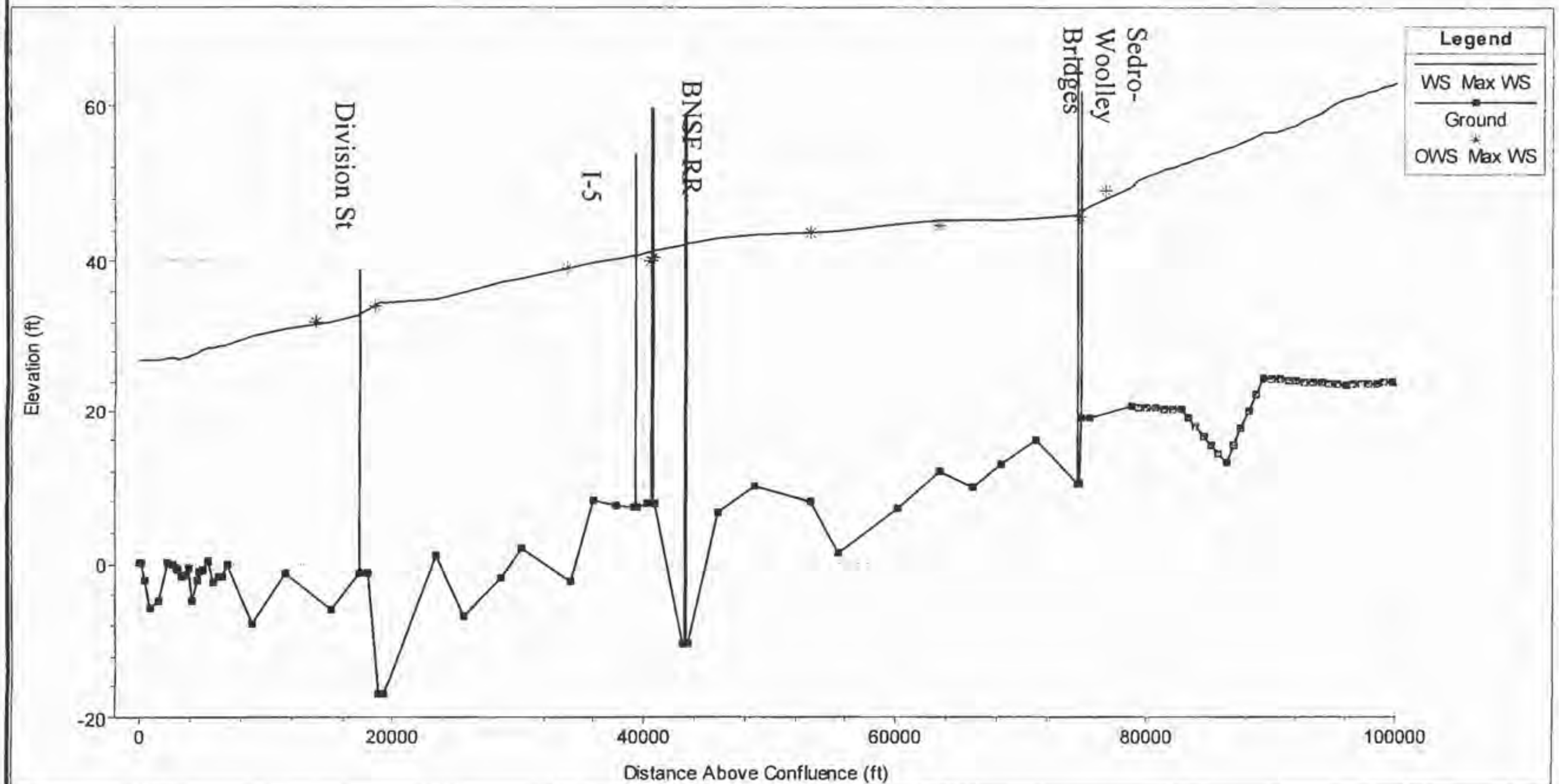
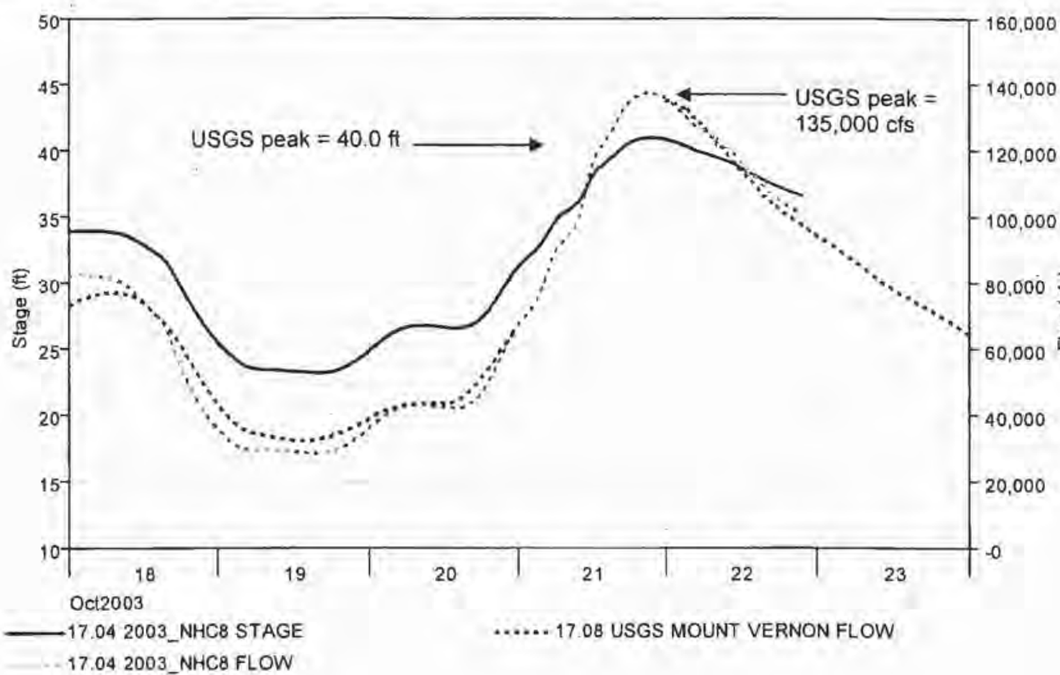


Figure 4. Comparison of USGS Gage Record to HEC-RAS Simulated Stage and Discharge Hydrographs at Mount Vernon for October 2003 Event.



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. 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 Mt. Vernon flows reasonably well (Figure 6).

Observed and simulated high water marks are shown in Table 9 and Figure 5. The model results closely approximate the observed high water marks downstream of Sedro-Woolley (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. Due to this there is less confidence given to how the USACE high water marks were collected for this event. 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.

Table 9. November 29, 1995 Simulated vs. Observed High Water Marks.

Source	Location	High Water Mark	Simulated	Difference
	(River Mile)	(feet NAVD88)	(feet NAVD88)	(feet)
USGS Gage	78.7	322.6	322.5	-0.1
USACE	54.12	175.4	174.7	-0.6
USGS Gage	54.10	175.3	171.8	-3.5
USACE	52.90	166.6	165.8	-0.8
USACE	46.97	142.5	136.2	-6.3
USACE	40.03	107.1	102.5	-4.6
USACE	32.93	75.7	74.4	-1.2
USACE	30.30	65.1	65.7	0.6
USACE	24.70	54.6	54.2	-0.3
USACE	22.40	50.0	46.5	-3.5
Leonard Halverson	22.30	45.7	46.2	0.5
Leonard Halverson	21.93	45.1	45.6	0.5
Leonard Halverson	21.60	45.2	45.6	0.4
Leonard Halverson	18.57	43.8	43.9	0.1
Leonard Halverson	17.90	44.6	43.5	-1.1
Leonard Halverson	17.53	43.0	42.1	-0.9
Leonard Halverson	17.08	41.0	41.5	0.5
USGS Gage	17.04	41.1	41.2	0.0

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.

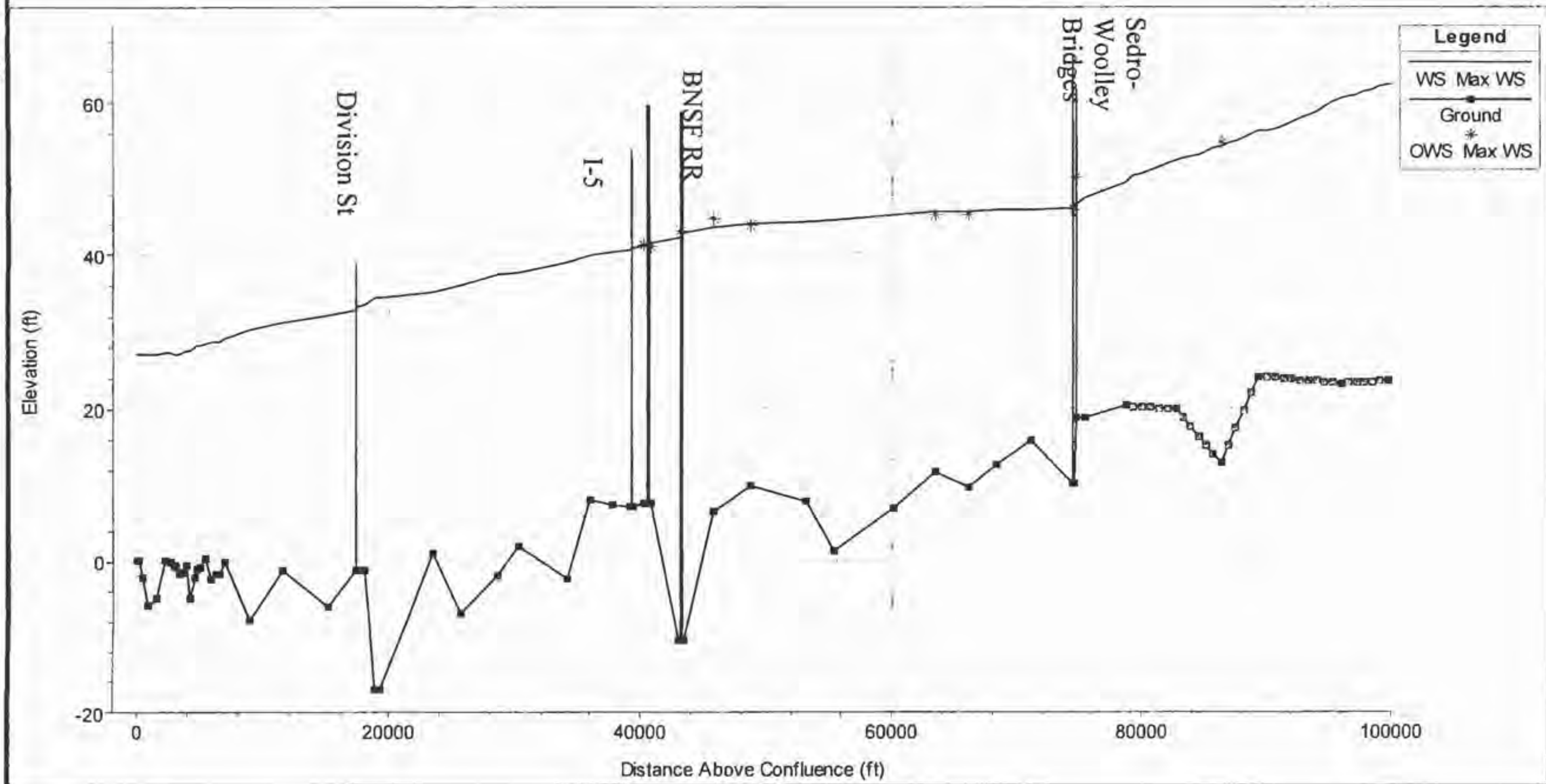
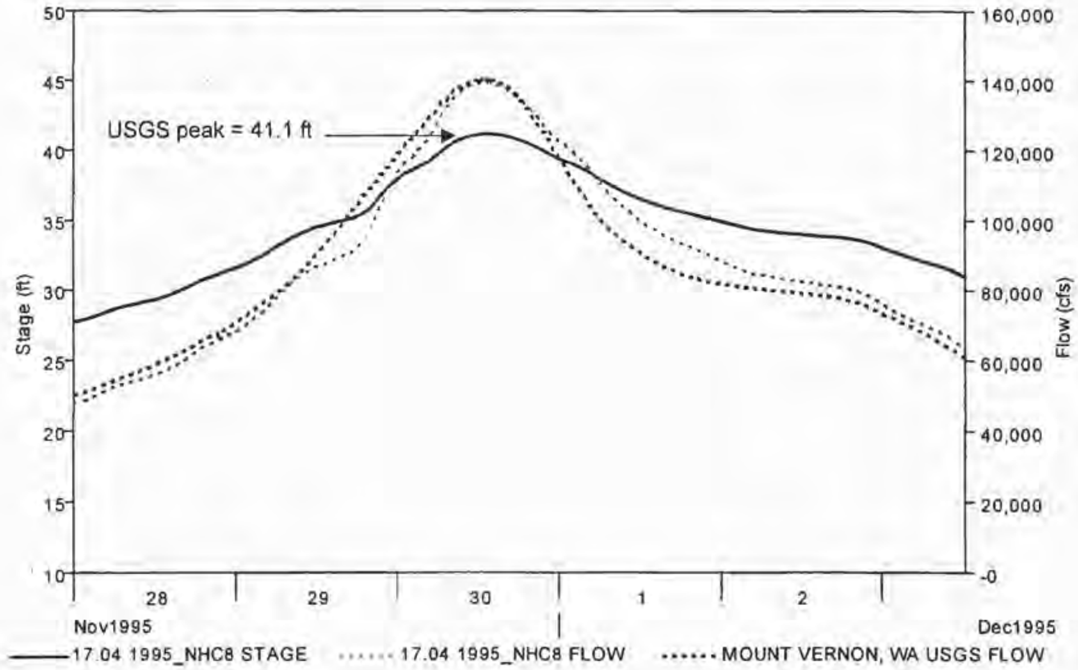


Figure 6. Comparison of USGS Gage Record to HEC-RAS Simulated Stage and Discharge Hydrographs at Mount Vernon for November 1995 Event.



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.

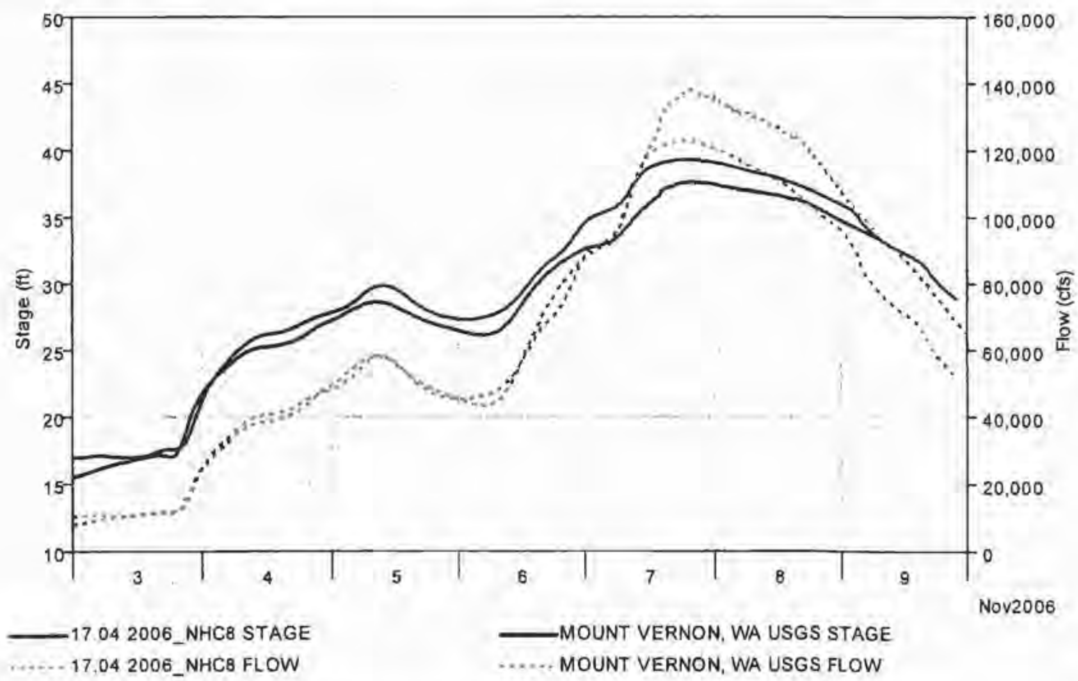
Initial HEC-RAS simulation results were poor when the local inflows were scaled so that Mount Vernon gage simulated flows matched the published peak USGS flow of 138,000 cfs, as was done for the 2003 and 1995 flood model runs. The USGS measured a discharge of 125,000 cfs on the rising limb of the November 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 6). Simulation results presented here for the 2006 flood assume a peak flow at Mount Vernon of 123,000 cfs.

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 120,000cfs, 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.

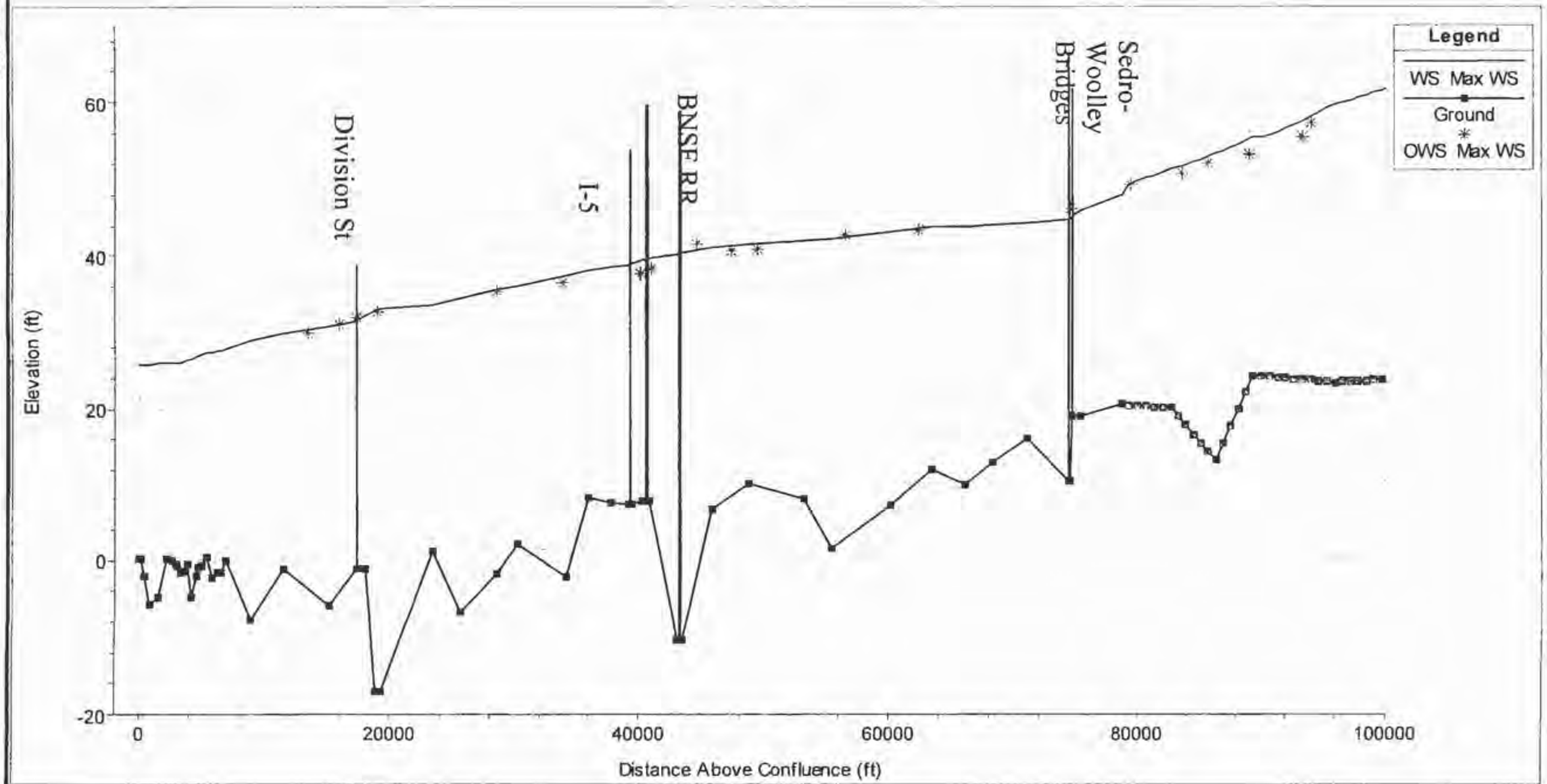


In general, the HEC-RAS simulation produced similar water surface elevations to the observed high water marks for the 2006 event, as shown in Table 10 and on the water surface profile of Figure 8. The primary exceptions to this are for the reach extending from the USGS Mount Vernon gage to upstream of 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 10. November 7, 2006 Simulated vs. Observed High Water Marks.

Source	River	Location	High Water Mark	Simulated	Difference
		(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	2.1
Skagit County	Mainstem Skagit	24.59	51.9	52.7	0.9
Skagit County	Mainstem Skagit	24.23	50.6	51.6	1.0
Skagit County	Mainstem Skagit	23.38	49.2	49.1	-0.1
USGS Gage	Mainstem Skagit	22.30	46.0	44.8	-1.2
Skagit County	Mainstem Skagit	22.29	43.6	44.8	1.2
Skagit County	Mainstem Skagit	21.40	43.2	43.5	0.2
Skagit County	Mainstem Skagit	20.9	42.6	43.0	0.4
Skagit County	Mainstem Skagit	18.77	40.9	41.5	0.6
Skagit County	Mainstem Skagit	18.31	40.5	41.2	0.8
Skagit County	Mainstem Skagit	17.79	41.5	40.7	-0.8
Skagit County	Mainstem Skagit	17.12	38.4	39.6	1.3
USGS Gage	Mainstem Skagit	17.04	37.6	39.2	1.6
Skagit County	Mainstem Skagit	15.85	36.5	37.2	0.6
Skagit County	Mainstem Skagit	14.80	35.4	35.6	0.2
Skagit County	Mainstem Skagit	14.59	35.4	35.6	0.2
Skagit County	Mainstem Skagit	13.05	32.5	32.9	0.4
Skagit County	Mainstem Skagit	12.96	32.0	31.8	-0.2
Skagit County	Mainstem Skagit	12.65	31.1	31.0	-0.1
Skagit County	Mainstem Skagit	12.09	30.0	30.4	0.4
Skagit County	South Fork Skagit	4.59	16.5	16.6	0.1

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 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.

Figure 9. Comparison of USGS Gage Record to HEC-RAS Simulated Stage Hydrograph at the Nookachamps Creek Swan Road Crossing for November 2006 Event.

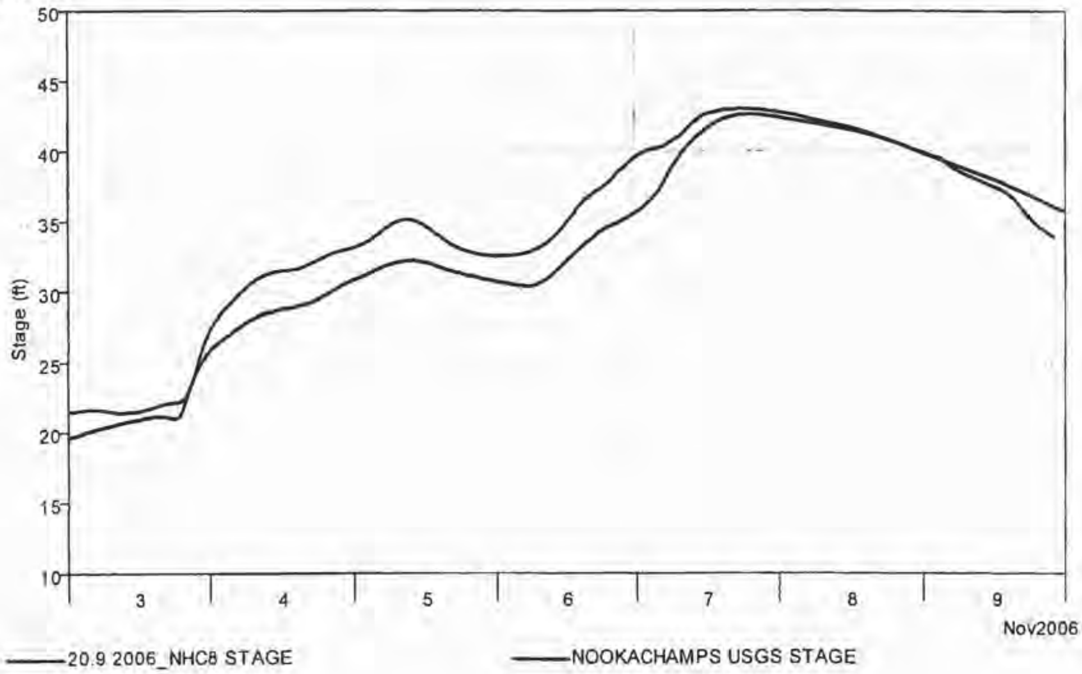
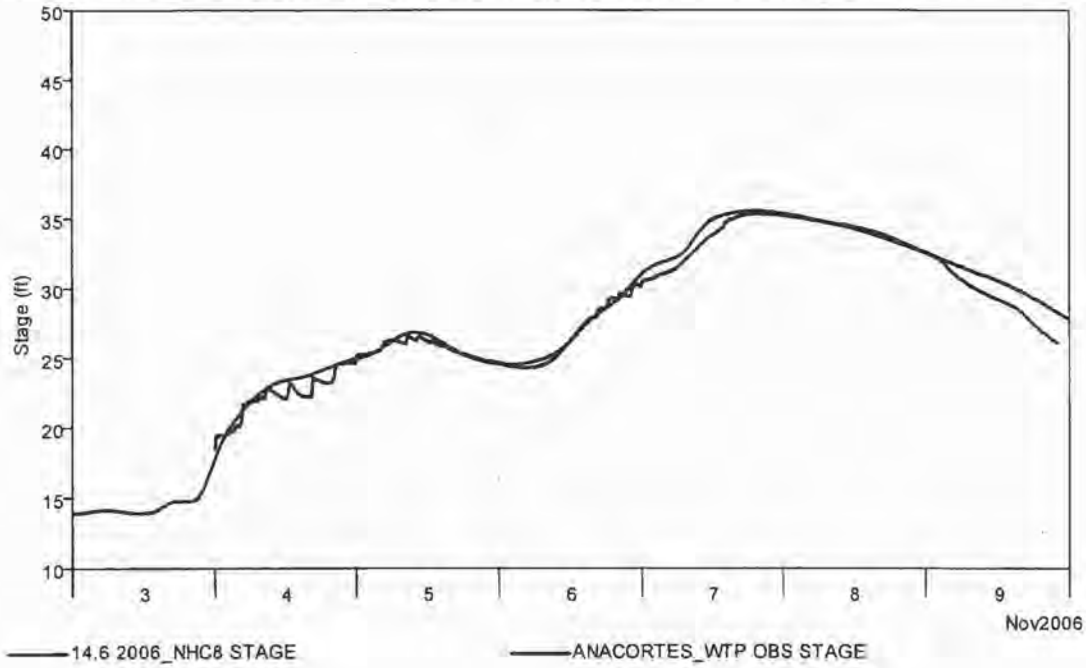


Figure 10. Comparison of Gaged Stage Hydrograph at Anacortes Water Treatment Plant to HEC-RAS Simulated Stage Hydrograph for November 2006 Event.



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 11.

Table 11. 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 0.06	.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. More complete details of hydraulic modeling results, including delineation of floodplain inundation under various levee failure scenarios, are provided in the hydraulics appendix to the economics report. Discussion of the use of the hydraulic models in risk-based analysis is also deferred to the hydraulics appendix to the economics 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. Note that unlike the model calibration runs, these simulations assume no flood fighting activities

Simulations were performed for the average channel roughness and average bridge debris loads. Water surface profiles for the nine hypothetical floods are provided in Appendix A along with the existing condition levee probable failure and probable non-failure elevations. Discharges at selected locations in the system are provided in Table 12. Also shown in Table 12 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 a dependent on peak flow attenuation due to storage in the lower Nookachamps Creek basin, spill due to overtopping of Highway 20 at Sterling, and spill due to overtopping of the right bank Dike District 12 levees upstream from the BNSF bridge.

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

The assumed bridge debris loading at the BNSF bridge has a significant effect on system hydraulics. 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 small, average and large debris loads. Levees were assumed to overtop with no failures. The assumed debris loads are shown in Table 4. Simulations were also done for the no debris condition. Peak discharges at selected locations are shown in Table 12. Water surface profile plots for the small, average and large debris loads for the 25-, 50- and 100-year events are shown in Appendix B.

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 average debris loading.

Discharges at selected locations in the system under the infinite levee scenarios are listed in Table 12. 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. The infinite levee failure scenarios are also used to generate stage-discharge ratings at index points for future use in HEC-FDA.

Table 12: HEC-RAS Hydraulic Model Results

	XS	2 -yr	5 -yr	10 -yr	25- yr	50- yr	75 -yr	100- yr	250- yr	500- yr
No Breach Scenario - Discharge (cfs) under Average Debris Loads										
Sedro-Woolley	22.4	80,500	105,000	130,000	163,000	187,000	209,000	223,000	278,000	324,000
Sterling spill	21.59-22.269	0	0	0	14,000	30,300	42,500	50,300	76,800	99,000
Levee overtopping u/s BNSF bridge	17.89-20.89	0	0	0	1,500	6,300	12,800	18,600	42,000	62,800
Mount Vernon, Riverside Dr bridge	17.04	76,500	92,400	118,000	137,000	147,000	147,000	154,000	160,000	164,000
Low Debris Loads										
Sedro-Woolley	22.4	78,800	105,000	131,000	164,000	187,000	208,000	223,000	277,000	324,000
Sterling spill	21.59-22.269	0	0	0	2,600	17,300	29,900	39,500	69,200	91,900
Levee overtopping u/s BNSF bridge	17.89-20.89	0	0	0	0	900	3,600	5,900	23,400	41,200
Mount Vernon, Riverside Dr bridge	17.04	76,500	92,500	119,000	144,000	160,000	169,000	173,000	182,000	187,000
High Debris Loads										
Sedro-Woolley	22.4	80,700	105,000	130,000	163,000	187,000	208,000	223,000	275,000	322,000
Sterling spill	21.59-22.269	0	0	100	18,600	37,800	49,300	56,900	81,500	103,800
Levee overtopping u/s BNSF bridge	17.89-20.89	0	0	0	3,800	14,200	24,300	31,800	57,500	80,700
Mount Vernon, Riverside Dr bridge	17.04	72,200	91,800	115,000	128,000	132,000	134,000	135,000	138,000	140,000
No Debris Loads										
Sedro-Woolley	22.4	80,500	105,000	131,000	165,000	187,000	208,000	223,000	277,000	326,000
Sterling spill	21.59-22.269	0	0	0	1,300	15,000	27,500	37,100	67,700	91,200
Levee overtopping u/s BNSF bridge	17.89-20.89	0	0	0	0	400	2,600	4,700	19,200	36,900
Mount Vernon, Riverside Dr bridge	17.04	76,500	92,400	119,000	145,000	162,000	171,000	176,000	186,000	192,000
Infinite Levee Scenario - Discharge (cfs)										
Sedro-Woolley	22.4	80,500	105,000	130,000	163,000	186,000	206,000	220,000	272,000	314,000
Mount Vernon, Riverside Dr bridge	17.04	76,500	92,400	118,000	143,000	168,000	196,000	207,000	246,000	280,000

5.0 References

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