SKAGIT RIVER FLOOD DAMAGE REDUCTION FEASIBILITY STUDY

HYDRAULICS TECHNICAL DOCUMENTATION

I. BACKGROUND

A. 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 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 damages downstream of Sedro Woolley. A secondary purpose is to investigate measures to restore ecosystem functions and processes in the project area to benefit fish and wildlife.

The authorization for the Skagit River Flood Control 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.

B. PURPOSE OF DOCUMENTATION

This report documents the work conducted for the Skagit River Flood Control Feasibility Study to develop hydraulic computer models, establish existing hydraulic conditions and floodplains, and use the hydraulic tools developed to evaluate two concept plans. The main product components of this effort include:

- Description of the hydraulic analysis methodology
- Development of the models (UNET and FLO-2D) for the Skagit River Basin
- Illustration of existing conditions based on model results
- Conclusions drawn from this effort
- Translation of results into data input to economic evaluations

C. STUDY AREA

The study area encompasses the Skagit River basin from Concrete, Washington (River Mile (RM) 55.35) to Skagit Bay. The Skagit River basin has a drainage area of 3,115 square miles of which 2,737 square miles is above Concrete, Washington. The study area is illustrated in Plate 1. The damage reaches that are evaluated start just downstream of Sedro Woolley (RM 22.4) and extend down to the mouth at Skagit Bay.

D. SKAGIT RIVER BASIN

The Skagit River Basin is located in northwest Washington State and has a total drainage area of 3,115 square miles. The Skagit River originates in a narrow, steep mountain canyon in Canada and flows west and south into Washington State where it continues for another 135 miles down to Skagit Bay. The source starts at 8,000 feet and drops to 1,600 at the US border. The hydraulic model starts at Concrete, WA, which is at RM 55.35. The average bed slope from Concrete to the mouth is 0.045%. From Concrete to Sedro Woolley (RM 22.4), the river flows in a valley 1 to 3 miles wide. The valley walls in this section are steeply rising timbered hills. Below Sedro Woolley, the valley descends to nearly sea level and widens to a flat, fertile outwash plain that joins the Samish Valley to the north and then extends west through Mount Vernon to La Conner and south to the Stillaguamish River. Between Mount Vernon and Sedro Woolley, a large area is being used as storage, primarily in the Nookachamps Creek Basin along the left overbank of the Skagit River. For very high river flows at Mount Vernon (over 146,000 cfs), a portion of the Skagit River in this reach can overflow along the right bank and escape out of the system through Burlington to the Samish River and Samish Bay. The Skagit River continues through a broad outwash plain in the lower reach nearest the river mouth and divides between two principal tributaries, the North Fork and the South Fork, which are 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 the usual range of river discharges.

II. HYDRAULIC ANALYSIS METHODOLOGY

A. Model Extent

The damage reaches that are evaluated start just downstream of Sedro Woolley (RM 22.4) and extend down to the mouth at Skagit Bay. This section describes the hydraulic analysis methodology, including the development of the UNET and FLO-2D hydraulic models, the modeling approach, levee failure methodology, and the development of floodplains. These models will be used to identify current, baseline conditions and analyze the effects of various alternatives and measures.

B. Study Approach

For this study, two computer hydraulic models, UNET and FLO-2D, are utilized to represent the hydraulics in the Skagit River Basin. 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 basin.

C. Floods Studied

For the hydraulic analysis, floods with 10-, 25-, 50-, 75-, 100-, 250-, and 500-year return frequency are explicitly modeled. Flows with less than a 10-year return period were not studied because they typically remain in the banks even in early failure scenarios and do not cause serious economic impacts on a system-wide basis. For information on how the hydrographs are developed for input into the models see the Hydrology Appendix (Appendix 1).

D. Description of Hydraulic Models

Computer-based hydraulic models, such as UNET 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 are simulated using UNET, and the FLO-2D model is used to simulate extensive flows in overbank and floodplain areas where they occur. The UNET model is interfaced to 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 UNET and FLO-2D as well as locations of cross sections and levees can be seen in Plate 1.

1. UNET Model Development

The computer model, UNET, developed by Dr. Robert Barkau, is designed to simulate unsteady flow through a full network of open channels, weirs, bypasses, and storage areas. For this study, use of the UNET model is limited primarily to the riverine channels. The July 1996 UNET Version 3.1 is used for this study. For more information about the capabilities of this model, refer to the April 2001 UNET User's Manual.

a. Purpose of Model

The purpose for using UNET in the Feasibility Study is to provide a means for understanding and representing the channel hydraulics in the Skagit River system. The UNET models are constructed to allow modeling of flood flow conditions. The UNET models are used to determine river stage, velocity, and depth as well as breakout flows from overbank areas.

b. Procedures and Process

Cross sectional data from the upstream boundary to the downstream boundary 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 Engineer's (USACE) Survey Branch. Floodplain geometry was obtained via aerial photogrammetry, while channel cross sections were field surveyed. All of the 52 1975 cross sections from Concrete to Sedro Woolley (RM 55.35 to RM 22.4) are used for this study. All of the cross sections from Sedro Woolley to Skagit Bay were resurveyed in 1999 by Skagit County. Some of these cross sections only included the underwater portions of the cross section so some parts of the 1975 cross sections are used in this reach to provide more detail.

Supplemental bridge data was field surveyed in 1998 by USACE - Seattle District's 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 measurement, photographs, USGS topographic maps, and profile point data. Bridge low and high chords are modeled along with bridge piers. For the second railroad bridge at RM 17.56, a significant amount of debris stacks up at the bridge. The debris loading condition used at the RR Bridge assumes that there is debris stacked up on the bridge 20 feet high for 90 percent of the width of the channel. This is the condition that was observed in the November 29, 1995 flood. This loading condition is selected as the existing condition because it would be difficult to remove this debris during extreme high flow events. A lower level debris condition was also looked at (debris that had a height of 5.5 feet high for 30 percent of the channel) and mainly results in more flooding occurring in Mount Vernon versus Burlington.

In the damage reach (Sedro Woolley (RM 22.4) to Skagit Bay), an analysis of 25 of the cross sections was completed by WEST Consultants, Inc. to determine the level of aggradation in the channel 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.

Overbank and channel distances between cross sections were assigned by scaling the linear channel and overbank distances between sections on a topographic map. Overbank distances were adjusted according to the presumed flow path. Due to the relatively confined nature of the floodplain from Concrete to Sedro Woolley and the somewhat steep channel gradient, no UNET-defined off-stream storage areas are used for that reach.

Resistance factors are estimated based on engineering judgment from field assessment of the channel and overbanks of the reach and from interpretation of topographic maps. Channel resistance factors of 0.035 are typical, while overbank resistance factors of 0.06 to 0.12 are assigned based on judgment dependant primarily on land use, land cover, topography, and historic and expected depth of flooding. These values are very similar to the values used for the 1984 Flood Insurance Study (FEMA, 1984).

c. Boundary Conditions

The four primary types of boundary conditions in UNET 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 this model, an upstream hydrograph is developed for the Skagit River at Concrete (for methodology, refer to the Hydrology write-up (Appendix 1)).

Downstream boundary conditions are required at the downstream end of all river systems not connected to another reach or river. Downstream boundary conditions consist of stage hydrographs and represent tailwater conditions such as tidal or estuary influences. The downstream boundary condition for both the North and South Forks of the Skagit River is a tidal hydrograph, which has a primary peak at the Mean Higher High Water (4.62 feet NGVD 29), a secondary peak at the Mean High Water (3.72 feet NGVD 29), and a low at the Mean Low Water. This downstream boundary only strongly influences the stages in the immediate vicinity of the boundary (lower couple miles). Various runs were done initially to ensure that this is true. Because of this, timing of the peak in the tidal hydrograph is fairly arbitrary. Internal boundary conditions are coded in UNET to represent levee failures or storage interactions, spillways or weir overflow/diversion structures, bridge or culvert hydraulics, or pumped diversions. Lateral inflow is distributed evenly from Concrete down to Burlington (RM 55.35 to RM 20.0) (see Hydrology write-up (Appendix 1) for methodology).

d. Basic Assumptions and Limitations

It is important to note some of the basic capabilities, assumptions, and limitations inherent with the UNET models. UNET 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. The spacing of cross sections in the UNET models also limits the application of these models to problems requiring finer detail.

2. FLO-2D Model Development

In general, FLO-2D was used to model overbank hydraulics for this study. Out-of-bank flows were generated in UNET and passed to the corresponding grid elements in FLO-2D to delineate the floodplain. The February 2002, Version 2002.2.5 is being used to conduct this effort. More information about FLO-2D can be found in the March 2002 FLO-2D User's Manual.

a. Purpose of Model

FLO-2D is used in this study to model overbank flows, which are comprised of flows that travel out of stream channels and across the topography of the floodplain. FLO-2D has the capability of modeling both one-dimensional channel flow and two-dimensional overbank flow. In the Skagit River system, FLO-2D is run in overbank areas only, exclusive of the channels. The Skagit River from Concrete to Sedro Woolley is fairly contained so UNET is exclusively used to model that reach. Additionally, the left bank (looking downstream) from Sedro Woolley to Mount Vernon (RM 22.4 to RM 13.1) can be accurately represented by defining the area as a storage area in UNET so no FLO-2D was utilized for this section. FLO-2D is necessary to define the routing of overbank flows in the rest of the basin (right bank from RM 22.4 to the mouth, left bank from RM 12.96 to the mouth, and Fir Island) (see Plate 1).

b. Procedures and Process

Assembling topographic data is the first task in developing the FLO-2D model for the Skagit River Basin. The entire floodplain for the lower Skagit Valley was aerial surveyed in 1999. This information is used to develop new topographic maps of the lower floodplain. A FLO-2D grid of the floodplain has been developed using the information from the aerial flight. This mapping was done to an accuracy that meets ASPRS standards for Class 2 accuracy for 2-foot contours, which means that the topographic feature points are +/- 1.33 feet and the spot or Digital Terrain Model (DTM) elevation points are +/- 0.67 feet. The floodplain model uses a grid system to route the overbank flows. For this study a 400-by-400 foot grid is utilized. This grid size is chosen to provide the necessary detail on the floodplain without burdening the model computationally with excess grids.

The average elevation for each grid cell is coded into the model along with information on the location and size of all structures in the floodplain. All features in the floodplain are noted on the new maps including houses, structures, and roads. Elevated roads are input so that the height of the roads could direct flow. The roads are modeled as levees that direct flow. Sea dikes are modeled the same way. Structures are included in the floodplain model by reducing the flow surface that each grid element can use. Post-processing of the output in conjunction with basin topographic data is performed to generate and define floodplains.

Because the channel portion of the FLO-2D model is not used, the floodplain needed to be broken into three distinct FLO-2D models. The first covers the right bank of the Skagit River as you look downstream, which starts at RM 22.4 and extends to the mouth of the North Fork Skagit River. The location of the upstream boundary is at the upstream end of the damage reach. This portion of the floodplain is modeled with 15,442 grids encompassing

56,720 acres. The second covers the left bank of the Skagit River, which starts at RM 12.96 and extends to the mouth of the South Fork Skagit River. The location of the upstream boundary of this model is at the Division Street Bridge. Flow from upstream of this area is blocked by I-5 and is modeled upstream as a storage area in UNET. This model contains 2,715 grids covering 9,970 acres. The third covers Fir Island from where the Skagit River splits into its North and South Forks down to their mouths. This represents the combined floodplain from the left bank of the North Fork to the right bank of the South Fork. This floodplain is modeled with 2,244 grids covering 8,240 acres.

FLO-2D allows flow to be routed through the model with either a diffusive wave approximation or with the fully dynamic wave routing technique. Guidelines on which technique should be used can be found in EM 1110-2-1417, Flood Runoff Analysis (USACE, 1994). Summarizing the criteria, a rapidly rising hydrograph being delivered to a flat-sloped area such as that found on the Skagit floodplain after a levee failure is best routed using a fully dynamic wave technique.

To model using the fully dynamic wave method requires defining the wave celerity and the minimum and maximum numerical wave stability factor. The values chosen for these factors are the ones that represent the average of the recommended starting point range in the FLO-2D manual (FLO Engineering, 2002). These values gave results that did not exhibit any stability problems and did not cause excessive run times so these values were not adjusted. These values can be seen in Table 2.

Table 2 – FLO-2D Dynamic Wave Coefficients

Minimum Stability Coefficient	Maximum Stability Coefficient	Weighting Factor
1.0	10.0	0.2

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 and inflow hydrographs are provided from the UNET model where the river gets past the levees. Outflow boundary nodes are indicated in the input data along with the general direction of the outflow (among the eight possible directions). The downstream boundaries for the Right Bank model are the Swinomish Channel and Padilla Bay, for the Fir Island model is Skagit Bay, and for the Mount Vernon model is just North of Stanwood. Tailwater conditions for the outflow nodes are based on normal depth, with the slope computed from adjacent node elevations.

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 model can route channel and overland flow using the fully dynamic wave or the diffusive wave approximation to the momentum equation. The fully dynamic wave is used for this study due to the rapid rise of the inflow hydrographs due to the levee failures.

The simulations performed represent a fixed bed analysis so erosion and sedimentation in the floodplain are not modeled. Culverts under roads are also not modeled. The reason that culverts are not modeled for overland flow 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. Structures Affecting Flow

a. Levees

A map showing the extent of levees in the Skagit River system is provided in Plate 1. 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. Levees are modeled as they were when the cross-sections were surveyed in 1999 so do not reflect any improvements that have been made since then. The levee breaches that enter storage areas for input into the Right Bank, Mount Vernon and Fir Island FLO-2D grids are assumed to be going into storage areas of infinite volume because it is assumed that the storage area in these overbank areas is so large that there is not a backwater impact from the storage area. The minimum elevation of the storage areas is defined as the ground surface behind the levee.

b. Bridges

The bridges in the Skagit River Basin modeled in this study are shown in Table 3. Some bridges within the basin are not included in the modeling effort because they do not significantly affect the hydraulics of the system. Information regarding bridge geometry, size, and other parameters included in the UNET model are obtained from bridge as-built drawings and field investigations.

Table 3 – Modeled Bridges on the Skagit River

Bridge Name	Skagit River Mile	Modeling Approach
Burlington Northern RR 1	22.4	Normal
State Route 9	22.3	Normal
Burlington Northern RR 2	17.56	Special
Riverside Drive	17.08	Normal
I-5	16.8	Normal
Division St.	12.95	Normal

There are two methods for modeling bridge hydraulics in UNET: the 'normal' and 'special' procedures. The normal bridge procedure simply subtracts the area of the embankments and bridge structure from the total cross sectional area. The decrease in cross sectional area and the increase in wetted perimeter combine to reduce conveyance through the bridge. The cross section of the bridge structure and the embankments is specified in UNET for the normal bridge method by the BT card. The normal method is most commonly used for perched bridges, where embankments are low and generally submerged, or where information about the bridge is not readily available.

The second method, the special bridge procedure, utilizes a family of free and submerged rating curves to simulate bridge hydraulics. The rating curves for the special bridge method consider the three types of hydraulic conditions that could occur at the bridge: free or low flow (when flow is below the bridge deck and only constricted by the piers), pressure flow (when the bridge deck is submerged and the bridge acts as a pressurized conduit or orifice), and weir flow (when flow is overtopping the bridge deck). The free and submerged rating curves are computed for the bridge-weir system for a range of headwater and tailwater elevations.

Only the Burlington Northern Railroad Bridge 2 has a condition where there is pressure flow so it is the only bridge where the special bridge approach is necessary. Additionally, this bridge is the main bridge that debris hangs up on. In 1995, roughly 20 vertical feet of debris hung up on the bridge for roughly 90% of the width of the channel. Similar debris loadings have been seen on this bridge during other large events. It is for this reason that this level of debris is modeled as the existing condition. UNET does not have specific debris cards so the debris is modeled by expanding the size of the piers and lowering the bridge deck to reduce the area available for conveyance (see Appendix C for debris loading run comparisons).

c. Diversion/Impoundment Structures

No diversions or impoundment structures are modeled from Concrete to the Mouth. This is because there is none that significantly impact the flood flows of the Skagit River.

4. Levee Breach Methodology

A levee breach methodology was devised to determine when simulated flows would cause levees to fail and a floodplain would be formed. To determine when a levee would fail and at what recurrence interval the levees would fail a Probable Failure Point/Probable Non-Failure Point (PFP/PNP) analysis of the levee system was conducted. To determine the points on the levee system that the levee would fail, geotechnical engineers from the Corps of Engineers completed an inventory of the levee system. The inventory determined for each reach of the river system where the PNP/PFP elevations would occur. The definition of the PFP is that when the water surface elevation (WSEL) in the river reached that level the levee would fail 85% of the time. The PNP level is where the levee would only fail 15% of the time.

For the average condition, likely failure points (LFP) (50% probability of levee failure) are developed for all of the levees along the Skagit River, which is taken as the halfway point between the PFP and PNP. The UNET model makes its determination of when the levee fails from the water surface elevation halfway between cross sections. A list of the LFPs used in the model is shown in Table 4.

Levee failure occurs in UNET for the average condition when the water surface elevation reached the LFP for a given levee. Levee failure is simulated by UNET as a levee breach. This failure method was adopted for UNET because levees tend to fail before they overtop, and flood-fight efforts and intentional breaching often prevent catastrophic failures of long sections of levee. Flow through a levee breach is then routed into floodplain storage areas by UNET. Levee failures for this study use the UNET embankment failure (EF) card.

The detailed embankment failure method, identified by the EF card, simulates an enlarging breach corresponding to either a piping or embankment failure. The breach starts when a failure elevation is exceeded, and is assumed to enlarge at a linear rate. Flow through a piping breach is given by an orifice equation, with failure occurring when the pipe breaks through the top of the levee. Flow through an overtopping breach is given by a weir equation. Levee break widths were also determined through consultation with geotechnical engineers. These break widths are modeled to be approximately 350-450 feet wide and would occur almost instantaneously. The levee failures are assumed to fail down to the existing ground level. The one exception is at the Burlington Northern Railroad embankment (RM 22.2 to 20.9) that is modeled as a levee where it is modeled as going to a storage area without the levee reducing its elevation when overtopped. This is because the ground behind the embankment is roughly the same height and is also backed up by Highway 20.

To represent the full range of possible flooding scenarios for each reach, a 5 and 95% methodology of levee failure needed to be developed. This was done by performing two additional runs where one of them has all of the levees failing at the Probable Non-failure Point and one where none of the levees fail until they overtop. These scenarios are used to define the upper and lower flow ranges for each reach for input into the economic analysis.

III. MODEL CALIBRATION

A. Sources of Data

There is limited data to calibrate the models with. High-water marks for the December 1975 flood and the November/December 1995 flood are used to calibrate the in-channel UNET model. The only overbank flooding information that is available is the data on the Fir Island levee break in November 1990.

B. Datum

The datum in use for both the FLO-2D and UNET Models and their output is the Washington State Plane Coordinate System, 1983/91 North American Datum, and Vertical Datum NGVD 29. This is different than the datum used for both the Mount Vernon and Concrete gages. The Mount Vernon gage datum is sea level and Concrete's gage datum is 130 feet above sea level. Sea level is 5.38 feet NGVD 29.

C. UNET Calibration and Validation

The roughness values of the channel for the UNET model of the Skagit River Basin is calibrated to the 1975 flood. The reason that this flood is chosen for calibration is because it is the flood with the largest set of high water marks (53 that cover from Concrete down to the mouths of both the North and South Forks of the Skagit) and it is close to a bankfull flow condition (130,000 at Mount Vernon) without anomalies that would make it atypical (i.e. no levee breaks).

For this calibration, the cross sections that were obtained in 1975 for the 1984 Flood Insurance Study were used. These cross sections are used for the calibration as opposed to the newer cross sections surveyed in 1999 because the channel has seen some aggradation at a number of cross sections and do not best represent the in-channel conditions of the 1975 flood. There is some complexity to calibrating to this flood because the upstream hydrograph has a double hump shape which gets flattened out by the time it moves down to Mount Vernon. This particularly makes the local inflow estimates difficult to time and predict. In the hydrologic analysis, the local inflow for a 10-year event is roughly 10% of the entire coincident hydrograph of Concrete. It is for this reason that a local inflow of 10% of the Concrete hydrograph is input from Concrete to Sedro Woolley (for more discussion on local inflow, see Hydrology write-up (Appendix 1)).

A comparison of the observed and modeled 1975 event at Mount Vernon can be seen in Figure 2. A comparison of the high water marks observed throughout the modeled reach and model results can be seen in Table 5. Because of the uncertainty of the local inflow and the precise locations of some of the high water marks, the focus of the calibration exercise was to ensure that those selected represent a reasonable approximation of the roughness in the channel and so was not refined to ensure exact matching of the high water marks.

The UNET roughness ranges are listed in Table 6 (for complete list by cross section, see Table 9).

Table 6 – UNET Roughness Ranges

	Main Channel	Left Overbank (looking downstream)	Right Overbank (looking downstream)
Mainstem Skagit above RM 17.5	0.030-0.043	0.06-0.12	0.06-0.12
Mainstem Skagit below RM 17.5	0.033-0.035	0.036-0.040	0.036-0.040
North Fork Skagit	0.035	0.035	0.035
South Fork Skagit	0.020-0.034	0.020-0.034	0.020-0.034

Validation

There are two other events for which high water marks exist. These are the November 1990 and November/December 1995 event. The November/December 1995 event was used to validate the model over the November 1990 event because the November 1990 event had a levee failure at Fir Island, which makes it very difficult to mimic the event without detailed information on the when, how wide, and exactly where the levee failure occurred. The 1995 event is modeled using 1999 cross sections from Sedro Woolley to Skagit Bay and the roughness values determined from the 1975 run calibration. The cross sections used from Concrete to Sedro Woolley are the 1975 cross sections. The simulated versus observed high water mark results can be seen in Table 7. The model appears to generally replicate the high water marks (although there are only eight HWM observations), with the notable exception of the high water marks between RM 40 and 50, which are higher than the computed elevation at these locations. These two water marks are considered fair and were captured in and near the town of Hamilton. The discrepancy in this area may be due to a split flow condition on the right overbank and therefore model accuracy might be improved by incorporating a split flow algorithm in the vicinity of these high water marks. It is also possible that lateral inflow contributions may be more heavily weighted in this area. Because these high water marks are all upstream of Sedro Woolley, there could also be some impacts due to channel aggradation like that found in the channel cross sections evaluated downstream by WEST. Since the affected area was upstream of the damage reaches and the problem does not appear to perpetuate itself into the damage reaches, the effort was not made to match these high water marks. A comparison of simulated versus observed hydrographs at Mount Vernon provides a better evaluation of how the model is performing in the damage reaches (see Figure 3).

Table 7 – November/December 1995 Simulated vs. Observed High Water Marks.

Location	High Water Mark	Simulated	Difference

(River Mile)	(Ft NGVD 1929)	(Ft NGVD 1929)	(Ft)
54.12	171.6*	169.99	-1.61
52.9	162.8*	163.26	0.46
46.97	138.7*	134.52	-4.18
40.03	103.3*	98.23	-5.07
32.93	71.9*	70.13	-1.77
30.3	61.3*	61.81	0.51
24.7	50.8*	50.65	-0.15
22.45	46.18*	42.32	-3.86
22.3	41.9**	42.32	0.42
21.95	41.3**	42.21	0.91
21.7	41.4**	42.11	0.71
18.57	40**	41.35	1.35
17.9	40.8**	41.06	0.26
17.4	39.2**	39.39	0.19
17.1	37.2**	37.56	0.36

*- From USACE

** - From Leonard Halverson via Larry Kunzler

D. FLO-2D Calibration

The only data on floodplain flows comes from the levee breach at Fir Island in 1990. Calibrating to the high water marks for this floodplain flow, however, would not be representative of the roughness values that can be seen with varying floodplain depths. This event put a large amount of water in the Fir Island area and the high-water marks are more influenced by a backwater situation opposed to floodplain roughness. Most of the hypothetical floodplain flows are affected by the floodplain roughness. It is for this reason that 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 8.

Table 8 – 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-	0-	Smooth	0	Negligible-	0 -	Low	0.01	0.01-	0.011[2]-?

	Lawn	0.02			Appreciable	0.03			0.06	

[1]From USACE (1993) EM 1110-2-1416

[2]From Engman (1986)

IV. UNET/FLO-2D Model Results and Output

UNET and FLO-2D were jointly used to model the hydraulic conditions in the Skagit River Basin. The results of the modeling simulations of existing conditions are discussed in this section. Flow, stage, and frequency relationships calculated by the model are reported at the index points.

A. Index Points

For each damage area (Right Bank upstream of RM 15.1, Right Bank downstream of RM 15.1, Left Bank in the Nookachamps area, Left Bank in the vicinity of the Big Bend upstream of Mount Vernon, Left Bank from Mount Vernon downstream, and Fir Island), an index point needs to be chosen to represent the ranges of flow and stages for the reach. For each of these damage areas, there is a cross section that best represents the typical ranges of flow and stages found in the reach. This was often chosen as the point just before a large levee failure occurs on that bank. These points are chosen as the index points as they relate closest to the damages caused for those reaches. For the Right Bank upstream of RM 15.1 and for the Left Bank in the Nookachamps area, the index point chosen is at RM 21.6. For the Right Bank downstream of RM 15.1, it is RM 14.0. For the Left Bank in the Big Bend area it is RM 16.82. For the Left Bank from Mount Vernon downstream is at RM 11.7. For Fir Island it is on the South Fork of the Skagit River at RM 5.25.

B. Flow/Stage/Frequency Relationships

The UNET and FLO-2D models calculated stage, flow, and velocity. Rating tables relating frequency, discharge, and stage at the index points for varying roughnesses are developed by using the output from the LFP run as the average condition. The variability of the stage is then accounted for by adding and subtracting the accepted uncertainty of roughness as found in Figure 5-4 of EM 1110-2-1619. The uncertainty in roughness comes from the unpredictability of bed forms, debris or other obstructions, sediment transport, channel scour and deposition, and changes in channel shape during or as a result of flood events. The variations in roughness values are shown in Table 9. The rating tables are shown in Table 10.

The variability of flow at each index reach is defined by the 5, 50 and 95% scenarios described in the Levee Breach Methodology section in section II D 4. This variability is shown in Table 11. For both Tables 10 and 11, certain cases exist where the average condition does not fall in between the high and low scenarios. This is in areas where the variability is very small and slight adjustments in how the river flows can put the average condition outside of these boundaries. In these cases, the average condition flow will act as both the average and one of the extremes.

C. Floodplain Delineations

Floodplains are developed for the 10-, 25-, 50-, 75-, 100-, 250-, and 500-year floods. The floodplains are shown on the floodplain maps in Appendix 2. <u>It is important to note that these are not FEMA floodplain maps, nor are they intended to replace or supersede existing FEMA maps.</u> Their main purpose is to develop an average annual damage estimate for existing conditions.

D. Qualification of Base Condition Results

The base condition results are plotted as n-year floodplains, but it must be emphasized that they are not FEMA floodplains, nor are they intended to replace or supersede existing FEMA maps. The intended use of the models and model output data is to evaluate the performance of the current and modified flood damage reduction features under a range of hydrologic conditions.

V. The Use of Hydraulic Results for Developing Project Outputs

A. Risk-Based Analysis

Risk involves exposure to a chance of injury or loss. Corps policy has long acknowledged risk and uncertainty in predicting floods and their impacts and to plan accordingly. Historically, planning relied on analysis of the expected long-term performance of flood management measures, on application of safety factors and freeboard, on designing for worst-case scenarios, and on other indirect solutions to compensate for uncertainty. These indirect approaches were necessary because of the lack of technical knowledge of the complex interaction of uncertainties in estimating hydrologic, hydraulic and economic factors because of the complexities of the mathematics required doing otherwise. However, with advances in statistical hydrology and the availability of analysis tools, it is now possible to describe the uncertainty in the choice of hydrologic, hydraulic, and economic functions, to describe the uncertainty in the parameters of the functions, and to describe explicitly in results when the functions are used. Through this risk-based analysis (RBA), and with careful communication of the results, the public can be better informed about what to expect from flood management projects and thus can make better informed decisions. The RBA is integral to the Corps plan formulation process, which systematically reviews the characteristics of the problem to identify and evaluate promising candidate flood management measures or combinations of measures. The policies, methods and procedures for the RBA conducted in this effort are as detailed in ER 1105-2-101, “Risked-Based Analysis for Evaluation of Hydrology/Hydraulics, Geotechnical Stability, and Economics in Flood Damage Reduction Studies” and in EM 1110-2-1619, “Risk-Based Analysis for Flood Damage Reduction Studies”.

B. Overview of RBA in Flood Management Studies

The determination of expected annual damage (EAD) in a flood management study must take into account complex hydrologic, hydraulic, geotechnical and economic information. Specifically, EAD is determined by combining the discharge-frequency, stage-discharge, and stage-damage functions then integrating the resulting damage-frequency function. Uncertainties are present for each of these functions and are carried forth into the EAD computation. In addition, for the rivers being studied that have levees or alternatives that contain levee measures, geotechnical failure parameters become very critical to the analysis. Once levees have failed and water enters the floodplain, then stages in the floodplain become more critical to the EAD computation than stages in the river channel. Additionally, economic efficiency of a plan or alternative is not the sole criterion for flood-damage reduction plan selection. Performance indices that assist in making informed decisions could include expected annual exceedance, long-term risk, and conditional probability of non-exceedance. These engineering performance indices allow for plan-to-plan comparison of risk of failure based on either the full range of floods or a specific flood. These indices are described below.

C. Flood Damage Reduction Analysis Model

The Corps primary model for performing flood damage reduction analysis is the Hydrologic Engineering Center's Flood Damage Reduction Analysis model (HEC-FDA, V1.2). The functions mentioned above are input into the model. HEC-FDA incorporates uncertainty for risk–based analysis using a Monte-Carlo simulation procedure. The two primary outputs from HEC-FDA include expected annual damage estimates and project performance statistics that are consistent with Corps guidance concerning the formulation of flood damage reduction plans. Plans can include structural as well as non-structural components.

D. Uncertainties Specific to the Skagit River Study

The Skagit River Flood Damage Reduction Feasibility Study, as with any other flood damage reduction study, has critical uncertainties associated with the hydrologic, hydraulic, geotechnical and economic data used to compute estimates of EAD and project performance statistics. The following discussion lists the important uncertainties for each of these disciplines and how they were (or were not) considered in this study.

Hydrologic-Uncertainty factors that may affect discharge are that the record lengths are often short or do not exist, precipitation-runoff computational methods are not precisely known, and the effectiveness of flow regulation is not precisely known.

Hydraulic-Uncertainty factors that may affect hydraulics are the modeling uncertainties.

Geotechnical - Conditional probability of failure curves described above help represent geotechnical uncertainty.

Economic-The @Risk program is being used in the Phase II economic analysis to develop stage-damage relationships with uncertainty. Damages are being estimated by impact area and by damage category. Economic variables with uncertainty used in the @Risk model include:

Structure Value
Content Value
Foundation Height
Depth-Damage Percentage

E. Expected Annual Damages

The benefits and costs of a flood reduction plan are to be expressed in average annual equivalents by performing appropriate discounting and annualizing methods. The expected value of annual damage is equivalent to integrating the annual damage-cumulative probability function. This function is developed by systematically combining the discharge-frequency, the stage-discharge and the stage-damage functions, including uncertainties. The Corps primary model for performing flood damage reduction analysis is the Hydrologic Engineering Center's Flood Damage Reduction Analysis model (HEC-FDA, V1.2). The functions mentioned above are input into the model. HEC-FDA incorporates uncertainty for risk –based analysis using a Monte-Carlo simulation procedure. Expected annual damages are computed for both without and with project conditions. Benefits are the difference between without and with project damages.

F. Expected Annual Exceedance Probability

The "expected annual exceedance probability" (AEP) is the probability of a project or alternative being exceeded in any one year. This performance parameter is derived by tracking the number of "failures" in the Monte Carlo sampling within HEC-FDA, divided by the number of samples. For example, if a levee has a 0.04 probability of being overtopped, it is said that in any given year it has a chance of failing 1 in 25.

G. Long-Term Risk

Long-term risk characterizes the probability of exceeding a plan or alternative in a specified period of time. This duration could be the proposed design life of the project, say 50 years or the duration of a home mortgage, 30 years. For example, within the 30-year life of a conventional home mortgage, the probability of overtopping may be 0.27 (or 27%). Such information is useful to help the public understand the risk of a given alternative and how it may apply directly to them.

H. Conditional Probability of Non-Exceedance

Conditional probability of non-exceedance is an index of the likelihood that an alternative will not be exceeded, given the occurrence of a specific hydrometeorological event. This index is similar to the AEP except the Monte Carlo sampling is performed at specific frequencies rather than sampling the entire range of frequencies. An example of the use of this index is, for the levee alternative X, the probability of containing the 0.01 or the 100-year event is 87%. This index is similar to the classic definition of "level of protection" (LOP). The LOP can be expressed as the average return period in years of the largest flood that can be contained by an alternative with a very high conditional non-exceedance probability, say 90% (see FEMA Certification below). Under this definition, the example levee alternative above does not meet the definition of a 100-year LOP.

I. FEMA Certification

The "Guidance on Levee Certification for the National Flood Insurance Program" dated 25 March 1997 was used to evaluate a levee alternative for FEMA certification. The guidance states that a levee is certifiable if the levee elevation meets FEMA criteria of 100-year flood elevation plus three feet of freeboard and achieves a conditional probability of non-exceedance of 90%. When the FEMA criteria results in a conditional probability of non-exceedance greater than 95%, the levee may be certified at the elevation corresponding to 95%.