SKAGIT RIVER HYDROLOGY INDEPENDENT TECHNICAL REVIEW DRAFT REPORT

Prepared for: Skagit County Department of Public Works

Prepared by: Northwest Hydraulic Consultants

nhc

February 2007

TABLE OF CONTENTS

FIGURES

CREDITS AND ACKNOWLEDGEMENTS

This report was prepared by Malcolm Leytham with contributions in the area of hydraulic modeling (Section 3.1) by Bob Elliot, both of Northwest Hydraulic Consultants Inc. (**nhc**). Internal review was provided by Larry Karpack also of **nhc**. The project was managed for Skagit County by Torey Nelson.

We would like to thank: Mark Mastin (USGS, Tacoma) and Ted Perkins (USACE, Seattle) for providing much of the basic data necessary for this work; John England (US Bureau of Reclamation, Denver) for providing the EMA software and related information; and Bob Jarrett (USGS, Denver) for thoughtful insights on flood risk. We would also like to thank Larry Kunzler for assistance in locating and providing copies of historical documents through his remarkable document management system.

Finally, we would like to thank the staff of Skagit County for facilitating this work, with special thanks to Torey Nelson who served as a responsive and effective project manager for the County.

1.0 INTRODUCTION

Northwest Hydraulic Consultants (**nhc**) was retained by Skagit County to provide an independent technical review of hydrologic and hydraulic analyses either relied upon by or conducted by the US Army Corps of Engineers (Corps) in the preparation of its flood damage reduction feasibility study and its flood insurance study for the Skagit River.

The hydrologic and hydraulic analyses of flood flows on the Skagit and the baseline data for those analyses have been subject to extensive previous review by others. Our purpose here is not to go over old ground. Many aspects of Skagit River flood hydrology, such as the reliability of high water marks from historic flood events, have been subjected to detailed review and re-review by others and we see no merit in yet another exhaustive review of those data. The work documented in this brief report provides additional focused review of topics which, in our opinion, merited additional consideration and which may lead to improved confidence in the characterization of the Skagit River flood hydrology. We have also identified a small number of issues that have not been previously raised but which may need to be considered in finalization of the Corps study reports.

2.0 THE FLOOD OF DECEMBER 1921

The estimate of the peak discharge for the flood of December 1921 is crucial to determination of the 100-year discharge, which in turn is the single most important hydrologic parameter for the flood damage reduction feasibility study and the flood insurance study. The peak discharge for the December 1921 flood was determined by J.E. Stewart at the location of the Skagit River near Concrete gage on the basis of indirect discharge measurements. Peak discharge estimates for the other historic floods at issue (water years 1898, 1910, and 1918) rely on a stage-discharge rating drawn through the December 1921 data point together with the elevation of high water marks from those events relative to high water marks from the December 1921 flood.

The following specific aspects of peak flow estimates from the December 1921 flood were subject to additional review:

- verification of Manning's roughness
- consistency of historic data with the stage-discharge rating
- consistency of historic data with evidence of non-inundation

2.1 Verification of Manning's Roughness

Stewart's 1923 estimate of the December 1921 peak discharge of 240,000 cfs was made using indirect measurement techniques. Three slope area measurements were made using high water mark information for a reach of the Skagit immediately below The Dalles and one contracted-opening estimate was made at The Dalles. The average of those four measurements was published as the peak discharge for December 1921. Stewart's slope area calculations relied on an estimate of Manning's roughness "n" for a reach of the Skagit near Sedro-Woolley. Stewart recognized that hydraulic conditions at Sedro-Woolley could be different from conditions at The Dalles and recommended that his assumed value for "n" be verified.

To date, two "n" verification studies have been completed for the reach of the Skagit below The Dalles used by Stewart for his slope area measurements. These studies were conducted in the early 1950s, using data from the flood of 27 November 1949 (peak discharge at Concrete of 153,000 cfs) and in 2004, using data from the flood of 21 October 2003 (peak discharge 166,000 cfs).

High water marks for the flood of 7 November 2006 (peak discharge 145,000 cfs) have also been identified in the field by **nhc** and surveyed by Skagit County. High water marks for this event have also been independently identified and surveyed by the USGS. **nhc** has completed an analysis of the November 2006 data, as discussed later in this section, to provide a further check on channel roughness "n".

Estimates of "n" from data obtained after the 27 November 1949 flood and recalculation of the peak discharge for the December 1921 flood are discussed in a series of brief memos by USGS staff between 1950 and 1954 (Riggs and Robinson 1950, Benson and Flynn 1952, Bodhaine 1954, and Flynn 1954b). The outcome was an “n” value of 0.030 and a recomputed peak discharge for the December 1921 flood of 225,000 cfs that was believed to be “the most logical answer based on the latest methods of computing peak flow” (Bodhaine 1954). The memos recommended that “the highwater rating be extended through a discharge of 225,000 cfs (as computed by Benson) for the stage of the 1921 flood …” (Flynn 1954b). However it was recommended that the published value for the 1921 flood be kept at 240,000 cfs as determined by Stewart “because of the small percentage differences and the fact that these figures have been published in Water Supply Papers” (Bodhaine 1954). The hydraulic conditions during the December 1949 flood are closer to those of December 1921 than current conditions, particularly with respect to the large right-bank gravel/cobble bar below The Dalles, which is discussed in more detail below.

Verification of “n” using data from the flood of October 2003 is discussed by Mastin and Kresch (2005). High water mark (HWM) data from the October 2003 flood were apparently difficult to identify in the field and show a substantial scatter. It was perhaps unfortunate that the USGS chose to publish this “n” verification work since the scatter in the data have been used by some parties in an attempt to call into doubt the applicability of slope-area techniques on this reach of the river. The slope-area measurement is a well established technique for estimating river discharges and in our opinion is reasonably well-suited to this reach, provided reliable water level data are available. The USGS used the HWMs from the October 2003 flood to determine a range of “n” values from a low of 0.024 to a high of 0.032. When applied to the HWMs from the December 1921 flood, these “n” values in turn result in a peak discharge for December 1921 in the range 215,000 cfs (“n” of 0.032) to 266,000 cfs (“n” of 0.024). The range of “n” values determined by the USGS is, in our opinion, so large as to be of little value in verifying Manning’s roughness and does not contribute to improving confidence in the reliability of the 1921 peak discharge measurement.

A water surface profile for the 7 November 2006 flood was staked by **nhc** for the reach in question approximately 10 hours after the peak discharge at Concrete, and high water marks for the event were staked by **nhc** a week later, on 14 November 2006. The staked points were subsequently surveyed by Skagit County. This work was restricted to the right bank because of difficulty accessing the left bank. The HWMs for this event were very obvious, identified primarily by strand lines of light debris at the edge of high water, and are considered to be accurate to within about 0.2 ft. HWMs for this event were also identified and surveyed by the USGS. The **nhc** and USGS data are plotted on Figure 1. Also shown on Figure 1 are the locations of USGS cross-sections 2 and 3, which define the reach used by the USGS in their 2004 “n” verification, along with a best estimate high water profile for the event.

The high water profile data of Figure 1 were used, with cross-section data from 2004 provided by the USGS, to estimate channel roughness for reach 2-3, following the procedures used by Mastin and Kresch (2005). The data were analyzed using the USGS

NCALC software (Jarrett and Petsch, 1985) to produce an estimate of "n" for reach 2-3 of 0.030.

The primary problem in use of data from November 2006 to verify roughness for the purposes of recomputing the December 1921 peak discharge is the effect of changed channel conditions between 1921 and present. Two specific concerns arise; possible changes in channel cross-section, and changes in the large right-bank gravel/cobble bar about 2000 ft below the Dalles which is now well vegetated with trees and shrubs but which is believed to have been clear of any significant vegetation in 1921. While there is no direct evidence of the condition of this bar in 1921, an aerial photograph of 1937 shows the bar to be bare, and it would be reasonable to expect that Stewart would have commented if the bar had been vegetated in 1921.

Mastin and Kresch (2005) compare surveyed cross-sections from 1923 and 2004 and showed differences in channel geometry to be relatively small. The effect of vegetation growth on the gravel/cobble bar is more difficult to deal with.

During the November 2006 event, the entire bar was submerged. Section 2 cuts across the downstream tip of the bar, while Section 3 is a further 2100 ft downstream (a location map is provided in Figure 1 of Mastin and Kresch). The high water data show that the bar was under about 8 feet of water at Section 2. Growth of vegetation on the bar can be expected to have some impact on water levels at Section 2. This impact cannot be reliably quantified, but with Section 2 located close to the downstream end of the bar the effects are probably quite small.

nhc's estimate of "n" from analysis of the November 2006 flood data is consistent with Benson's estimate of "n" based on data from the November 1949 flood and his recomputation of the peak discharge for the December 1921 flood of 225,000 cfs. Since this estimate was only 6% lower than Stewart's original estimate, no change was made to the then-published value of 240,000 cfs. Nevertheless, the USGS n-verification for the November 1949 flood, under conditions closer to the those of 1921 than today's conditions, together with supporting evidence from the November 2006 event, indicate that the 240,000 cfs published value for December 1921 is conservatively high, other possible sources of uncertainty notwithstanding.

2.2 Consistency of December 1921 Data with Published Rating Curve

The Skagit River near Concrete gage has a well defined and stable stage-discharge rating which has been established through a comprehensive long-term program of discharge measurements. The current rating curve (Rating No. 6) is plotted in Figure 2 along with a selection of stage-discharge measurements. The discharge measurement for December 1921 is for the indirect measurement reported by Stewart. The corresponding gage height(s) shown in the figure for the December 1921 measurement are discussed below. All other measurements on Figure 2 are direct measurements (e.g. with a Price meter or more recently by ADCP) as reported by the USGS.

The top end of the stage-discharge rating (from around 80,000 cfs to 150,000 cfs is defined by a set of measurements taken during the flood of February 1932 (highest measured discharge 135,000 cfs, gage height 38.68 ft). A measurement taken in October 2003 (discharge 138,000 cfs, gage height 38.68 ft) agrees very closely with those of February 1932, confirming and validating the rating at least up to about 150,000 cfs. The close agreement between the highest measured discharge in February 1932 and the October 2003 measurement suggests that changed channel conditions downstream of the gage site (primarily changes in vegetation on the right bank gravel/cobble bar) have had no discernible impact on the stage-discharge rating at the gage site, at least for discharges up to 150,000 cfs. At this discharge, we estimate the bar to be covered by from 5 ft to 8 ft of water. The effect of the gravel/cobble bar on upstream conditions at the gage site can be expected to decrease at higher discharges because of the hydraulic control imposed by the contraction at The Dalles. Given the stability of the channel at the gage site, there is no reason to expect a material change in the high water rating between 1921 and present at this location.

The indirect measurement for the December 1921 event is plotted on Figure 2 using Stewart's discharge estimate but with two alternate "estimates" of gage height. The higher of the two gage heights shown for December 1921 is as published by the USGS and as determined by Stewart for the original site of the Concrete gage (reported to be 200 feet upstream from its current location) adjusted for the difference in datum between the original and current gage sites. The lower of the two gage heights shown for December 1921 reflects a further hypothesized adjustment of 2 feet to account for the water surface slope between the old and new gage sites. All other data shown on Figure 2 are taken from the current gage which was established in 1924.

No attempt has been made in the data published by the USGS to adjust the reported December 1921 gage height for difference in water level between the old and new gage locations. (Gage heights published for the other historic events are, similarly, referenced to the old gage site.) The magnitude of those differences has been the subject of limited discussion. Flynn (memo 16 July 1954), in a discussion of the historic flood peaks, states "from the falls measured in the slope-area determination[1], the fall between the two gage sites is probably on the order of 0.2 ft". Mastin (letter to C. Martin, 10 February 2005) states that "As the flows increase, the draw down through the gorge seems to begin further upstream somewhere upstream of the current gage location. The HWMs from the October 2004[2] peak flow, gage height of 42.14, showed a drop of 0.5 to 1.5 feet from the old site to the current gage site depending on which HWMs are chosen to represent the slope".

Given the hydraulic conditions at The Dalles, the fall in water level between the two gage sites can be expected to increase with increasing discharge. The fall in water level between the old and new gage sites for floods of the magnitude of the 1921 event could thus be in the range 0.5 to 2 feet. (A drop of 2 ft. is consistent with estimates by Pacific

[1] This refers to the slope-area measurement for the 1949 flood
[2] This is a typographical error – the event referred to is October 2003

International Engineering Ltd. Ased on a HEC-RAS model. However, given the complexity of hydraulic conditions at The Dalles, we would not rely on a HEC-RAS model to either determine or validate estimates of water surface slopes at this location.) Based on the slope of the published stage-discharge rating, at a discharge of 240,000 cfs a reduction in gage height of 2 feet corresponds to a reduction in discharge of 20,000 cfs.

Rating curves are typically extrapolated beyond the range of direct measurements as a straight line in log-log space. The exact basis for extrapolation of the rating for the Concrete gage is unknown. As noted in Section 2.1 above, it had been recommended (Flynn 1954) that the highwater rating be drawn through a discharge of 225,000 cfs for the reported stage of the December 1921 flood (47.6 ft at the current gage datum). This recommendation was apparently never adopted since the current rating does not pass through that point. As with the discharge measurements themselves, there is uncertainty associated with the extrapolation of rating curves. A slight variation of the top end of the published rating, as a straight line extrapolation in log-log space, is plotted in Figure 2. This illustrates how a modest adjustment could result in a rating which provides a better fit to the published (higher gage height) December 1921 indirect discharge measurement. The top end of the current rating will only be known with more certainty if measurements of stage and discharge are obtained under those extreme conditions.

Mastin's 10 February 2005 letter to Martin, which deals primarily with issues surrounding the reliability of HWMs, points out that Stewart's estimate of the December 1921 peak discharge was made independently of estimates of the gage height at the (old) Concrete gage. Nevertheless, it is clear there are unresolved (and probably unresolvable) inconsistencies between several of Stewart's data and the stage-discharge rating established for this site. The range of magnitude of the December 1921 peak discharge for various assumptions regarding the stage-discharge rating and gage heights estimates are summarized below:

Assumptions	**Peak Gage Height or Discharge (December 1921)**	
	Gage Height (feet)	**Discharge (cfs)**
Gage height reported by Stewart with discharge from Rating 6	47.6	215,000
Gage height adjusted for 0.5 ft fall to new gage site with discharge from Rating 6	47.1	210,000
Gage height adjusted for 1.5 ft fall to new gage site with discharge from Rating 6	46.1	201,000
Gage height adjusted for 2 ft fall to new gage site with discharge from Rating 6	45.6	196,000
Gage height reported by Stewart with discharge from straight line log-log extension of Rating 6 above 140,000 cfs	47.6	222,000
Discharge reported by Stewart with gage height from Rating 6	50.2	240,000

2.3 Consistency of December 1921 Data with Evidence of Non-Inundation

According to research by Kunzler (2006), the Smith House in Hamilton (307 Maple Street, Hamilton) was built in 1908 and anecdotal reports indicate that it has only once been flooded above its main floor level. The house is reported to have had 2 inches of water above the main floor level during the flood of November 1995 (peak discharge at Concrete 160,000 cfs). Anecdotal reports suggest that the house was **not** flooded in earlier and much larger flood events (1910 – 260,000 cfs, 1918 – 220,000 cfs, 1922 – 240,000 cfs). If flows of the magnitude of these historic events had occurred under **current** river channel conditions, then the water levels should have been several feet above the main floor level. These apparent inconsistencies have a number of possible explanations:

- the anecdotal reports are incorrect and the house was in fact flooded above the main floor level in the earlier floods,
- the peak discharge estimates for water years 1910, 1918, and 1922 are incorrect and are too large, or,
- the hydraulic conveyance capacity of the river channel and/or floodplain in and around Hamilton was historically significantly greater than at present and was able to carry greater flows at lower water levels.

Preliminary work has been undertaken (City of Burlington, 2007) to determine whether or not the Smith House was flooded prior to 1995. The work undertaken involved cutting out two sections of lath and plaster wall covering on the main floor level of the house and inspecting the interior of the wall for signs of flood damage or water marks. No water marks were found. Some discoloration of plaster was evident at the base of the wall, possibly indicating water damage during very shallow flooding of the main floor in 1995.

Although no sign of water damage from large historic floods was evident, it is our present opinion that this does not provide **conclusive** evidence that flooding did **not** occur. Any flood marks from December 1921 would now be 85 years old. From our limited experience with flooding of buildings, we would expect flood marks to fade with age. At the present time, we simply do not know whether a flood mark on the interior of a wall would still be visible after 85 years.

Historic maps and aerial photographs were inspected to assess changes in channel and flood plain conditions since the 1900s. Bank lines of the river were determined from Government Land Office (GLO) maps from about 1886, and from aerial photographs from 1937 and 2001. The changes in channel bank lines with time are shown in the immediate vicinity of Hamilton in Figure 3. Also shown on Figure 3 are the Cockerham Levee and the Great Northern Railway Line, the only man-made features on the flood plain that might affect flood levels in Hamilton in any significant way.

Figure 3 clearly shows a significant shift in channel planform between 1886 and 1937 downstream from Hamilton. The shift between 1937 and 2001 is much less pronounced.

It is presumed that movement of the meander bend downstream from Hamilton was arrested in the 1950's with the construction of the Cockerham Levee.

Figure 3 also shows a substantial narrowing of the river channel downstream from Hamilton between 1937 and 2001. The average channel width for the approximately 1.5 mile reach through the first meander bend below Hamilton was about 750 ft in 1886 and 900 ft in 1937 compared with only 600 ft in 2001. These estimates should be used with some caution since: (1) we do not know with certainty how the river channel was delineated on the GLO maps, and (2) bank lines from the 1937 and 2001 aerials were drawn, in the absence of stereographic coverage, as the edge of continuous vegetation. The greater width in 1937 is due mostly to inclusion within the defined channel of a broad left-bank sand or gravel bar. Nevertheless the river channel in 1937 (and presumably also at the time of the historic floods) was clearly wider than at present and would have had a correspondingly greater conveyance capacity. Just how much greater is not possible to determine with any accuracy since detailed channel surveys from 1937 are not available.

With regard to man-made features, the Seattle and Northern Railway line (later part of the Great Northern system) up the Skagit valley was completed to Hamilton by 1891 and to Rockport by 1901, i.e. before construction of the Smith House. The line has long been abandoned and was converted to a multi-use trail under the Rails-to-Trails program in the 1990s. We are of the opinion that any changes in flood conveyance capacity due to either the presence of the rail line or its conversion to a trail would be minimal.

The Cockerham Levee (see Figure 2) is hydraulically a much more significant man-made feature than the rail line. A representative cross-section through the levee is shown in Figure 4. The levee confines flows downstream from Hamilton and prevents spill over the right-bank floodplain until the levee is overtopped or is otherwise breached, as appears to have been the case on a number of occasions in the past (most recently in November 2006). As far as we are aware, no hydraulic modeling or analyses have previously been performed to assess the impact of the levee on upstream water levels. Since the levee is not certified, it is not included in the hydraulic models developed by the Corps for the current flood insurance study. An approximate analysis of the impact of the levee on water levels in Hamilton at RM 39.8 was undertaken as part of this review. A simple representation of the levee was included in the Corps' final HEC-RAS model and the model was then run with and without the levee for a flow of 160,000 cfs, as reported at Concrete for the November 1995 flood, and for a flow of 240,000 cfs, as published for the December 1921 flood and also, coincidentally, the current estimate of the 100-year regulated discharge at Hamilton. The results are summarized below:

Flood	**Discharge (cfs)**	**Water Surface Elevation in Hamilton (ft)***	
		Without Levee	**With Levee**
December 1921	240,000	102.7	103.9
November 1995	160,000	99.1	99.7

* Water levels estimated using 1975 hydraulic geometry data.

Note that the Corps' model uses cross-section data from 1975. The planform geometry of the river has not changed materially since 1975 and, in the absence of more recent cross-section data, it is assumed that the 1975 cross-section data are reasonably representative of conditions from 1975 to present. An assessment of the geomorphology of the Skagit River below the Baker River confluence was conducted in support of the FERC relicensing of the Baker River Hydroelectric Project (R2 Resource Consultants Inc., 2004). This work notes (pg. 4-13) that the Skagit River downstream from Sedro-Woolley has aggraded by about 1.5 feet since the mid-1960s. It also notes (pg 4-14) that the channel length between Sedro-Woolley and Hamilton has decreased since the early 20th century and that such changes are typically associated with an aggrading channel. There is thus some indication of possibly on-going channel aggradation below Hamilton, but there is no direct evidence in the form of surveyed channel cross-section data.

The approximate analysis conducted for this review indicates that the Cockerham Levee raises water levels in Hamilton by about 1.2 ft for flows in the range of 240,000 cfs. With the levee in place and assuming that the Smith House was just flooded to the level of the main floor at a discharge of 160,000 cfs in November 1995, then a discharge of 240,000 cfs **with the present channel conditions**[3] would have flooded the house to a depth of about 4.2 ft. Without the levee but **with present channel conditions**, the depth of flooding would have been about 3.0 ft. Given that the river below Hamilton was considerably wider in 1937 than today, it is possible that the river could have carried 240,000 cfs in 1921 without flooding the main floor of the Smith House. More definitive estimates of water levels at the Smith House during the December 1921 flood are not possible given the lack of detailed channel geometry data from that period.

---

[3] Strictly speaking, 1975 channel conditions, since the hydraulic model uses channel cross-section data from 1975.

3.0 COINCIDENT FLOWS AT CONCRETE AND SEDRO-WOOLLEY

The adequacy of hydraulic modeling between Concrete and Sedro-Woolley has been questioned because of differences between the attenuation of peak flows indicated by the Corp's hydraulic model and the attenuation indicated by the reported peak flows for the historic events.

Reported peak flows for the historic floods at Sedro-Woolley and Concrete (from USGS Water Supply Paper 1527) together with the Corp's modeled 100-year regulated peak flows are as follows:

Date	**Peak Discharge (cfs)**		**Difference**	
	Concrete	**Sedro-Woolley**	**(cfs)**	**(%)**
Historic Events (WSP 1527)				
19 November 1897	275,000	190,000	-85,000	-31
30 November 1909	260,000	220,000	-40,000	-15
30 December 1917	220,000	195,000	-25,000	-11
13 December 1921	240,000	210,000	-30,000	-13
Modeled Event				
100-yr regulated	235,400	242,000	+6,600	+3

The historic data show attenuation (reduction) in peak flows between Concrete and Sedro-Woolley ranging from 11% to 31% while the hydraulic modeling results for the 100-year regulated event show a 3% increase. The apparent discrepancy between historic data and model results may be due to one or more of the following factors:

- the hydraulic model may be unreliable
- modeled "local" inflows between Concrete and Sedro-Woolley may be too high
- differences between regulated and unregulated hydrology (i.e. unregulated historic flows would have likely been more peaked and thus more likely to show attenuation when compared to regulated flows which are already somewhat attenuated, with drawn out peaks)
- historic peak flows reported at Concrete may be too high
- historic peak flows reported at Sedro-Woolley may be too low

3.1 Hydraulic Modeling

nhc reviewed the Corps' HEC-RAS unsteady-flow hydraulic model of the Skagit system and accompanying hydraulic report (USACE, 2004a). While the hydraulic report focuses primarily on the lower Skagit River downstream of Sedro-Woolley (River Mile [RM] 22.4), the model actually extends upstream to Marblemount (RM 78.87) and includes the lower ends of major tributaries, including 5.4 miles of the largest tributary, the Sauk River. The primary focus of **nhc**'s review was flow attenuation within the reach from Sedro-Woolley upstream to the Baker River confluence at Concrete (RM 55.35). All cross-section data upstream from Sedro-Woolley are taken from 1975 surveys from the effective FIS (published in 1984), and are spaced on the order of 0.5 to 1.0 mile apart (excepting interpolated sections added for model stability). Downstream cross-sections within the area of greater interest to the Corps study were resurveyed in 1999 by Skagit County.

The floods of October 2003 and November 1995, both large events with a decent number of recorded high water marks, were used by the Corps in an attempt to calibrate and verify the hydraulic model. Gaged streamflow data and regression relationships with the North Fork Stillaguamish River (to estimate local inflows between Concrete and Sedro-Woolley) provided the inflow hydrographs required by the model. Calibration was performed for the 2003 event resulting in reasonable n-value roughness coefficients, ranging from 0.04 to 0.045 in the channel and 0.12 to 0.15 on the overbank along the Concrete to Sedro-Woolley reach (and beyond to the railroad bridge by Mount Vernon). Independent verification using the same set of n-values could not, however, be achieved for the 1995 event. In order to reasonably match high water marks for this event, significant changes in n-values were made. Within the first 8 miles or so downstream from Concrete, n-values are about the same as for the 2003 calibration. Continuing downstream from about RM 47.55 to RM 31.45, n-values are increased to 0.05 to 0.06 in the channel and remain 0.12 to 0.15 on the overbank. Along the lowest 8 miles of this reach (upstream from Sedro-Woolley) n-values were decreased to 0.03 to 0.035 in the channel and 0.08 on the overbank (n-values were also decreased downstream of Sedro-Woolley). Possible reasons cited for the difficulty in calibration and independent verification include differences in antecedent conditions and groundwater infiltration (October 2003 was preceded by a dry season whereas November 1995 was more typically wet), as well as questionable accuracy of the older 1995 high water marks. The final model relies upon the October 2003 calibrated n-values which appear more appropriate for the Concrete to Sedro-Woolley reach.

A review of the cross-sections from Concrete to Sedro-Woolley, based on the original FIS work maps and the HEC-RAS model itself, indicates that the cross-sections are representatively located with appropriate alignment and lateral extent. There were no significant areas of off-channel storage beyond the limits of the cross-sections (and therefore not represented in the model), which might act to attenuate flood peaks. Given the age of the cross-section surveys, their rather wide spacing, and the uncertainty in calibration and n-values, the localized accuracy of computed water levels at specific locations within this reach may be questionable. The larger question, as pertains to this review, is how these uncertainties in the hydraulic modeling in the upper reach between Concrete and Sedro-Woolley translate to uncertainties in the lower reach, below Sedro-Woolley. In other words, how sensitive is the computed discharge at Sedro-Woolley to the uncertain model inputs within the upper reach, such as n-values and cross-section data as well as upstream hydrologic inputs?

The following sensitivity tests were conducted using the HEC-RAS unsteady model to help answer these questions:

1) Using the final 2003 geometry file, all n-values (channel and overbank) were increased by 50% from Concrete to Sedro-Woolley. Such n-values would be considered unusually high for this reach of river, but it provides for a useful "what-if" scenario. The increased n-values result in greater flow depths and therefore greater storage of water, both in-channel and on the floodplain. Even with such an extreme modification to n-values, the reduction in 100-year peak flow at Sedro-Woolley (due to increased storage attenuation) is only about 3,000 cfs.

2) The time base of the 100-year inflow hydrographs to the model was halved to investigate the effect of uncertainty in the duration or "peakedness" of those input hydrographs. Longer duration, larger volume floods result in less flow attenuation. Increasing the peakedness (reducing the volume) of the inflow hydrographs should result in greater storage attenuation. Halving the time base of the inflow hydrographs resulted in a 12,000 cfs reduction in the 100-year peak flow at Sedro-Woolley compared to the baseline run.

3) The modeled 100-year local inflow between Concrete and Sedro-Woolley consists of a hydrograph with a peak discharge of 30,000 cfs (see Figure 5), evenly distributed along the reach. For test 3 the local inflows between Concrete and Sedro-Woolley were eliminated to determine their effect on flows at Sedro-Woolley. Eliminating local inflows resulted in the 100-year peak flow at Sedro-Woolley being reduced by 17,000 cfs relative to the baseline run. Since peak flow attenuation between Concrete and Sedro-Woolley is masked by local inflows, this run also demonstrates that floodplain storage is, in fact, causing attenuation in the modeled flows. As shown in Figure 6, the peak flow in this sensitivity run was reduced by approximately 6,000 cfs (2.5%) between Concrete and Sedro-Woolley.

4) Local inflows between Concrete and Sedro-Woolley were eliminated **and** the time base of the other 100-year inflow hydrographs to the model was halved to estimate the peak flow attenuation under the most favorable conditions. The resulting 100-year peak flow at Sedro-Woolley was reduced by 26,000 cfs relative to the baseline run, with the peak flow between Concrete and Sedro-Woolley for this run being attenuated by about 16,000 cfs (7%).

The results of the sensitivity runs are summarized in the peak discharge profile plot of Figure 6.

Even with the most extreme of the above scenarios (no local inflow between Concrete and Sedro-Woolley and the time base of the hydrographs halved) it is not possible for the hydraulic model to reproduce the attenuation implied by the historic flood data from

Concrete and Sedro-Woolley, which strongly suggests that either the historic peak discharges reported for Concrete are too high or the corresponding discharges reported at Sedro-Woolley are too low.

3.2 Estimation of Historical Peak Flows at Sedro-Woolley

By all accounts, estimation of peak flows for the Skagit River at Sedro-Woolley is fraught with difficulty. Stage-discharge ratings for the site have proven to be unstable and extension of the ratings to high discharges is subject to significant uncertainty. Stewart (1923) states (pg 12) "…this is an exceedingly poor station, so far as rating is concerned, and should be abandoned." The principal issues as they affect estimation of peak flows for the floods of water years 1898, 1910, 1918, and 1922 have been identified as follows:

- Difficulty in estimating overbank flows during large floods (Stewart [1923] provides a lengthy discussion of these difficulties and estimates that overbank flow during the 1922 flood amounted to about 25% of the total discharge).
- Changes in channel conditions downstream from the gage, most significantly the Sterling bend cut-off of 1911.
- Effect of downstream levee failures on water surface slope and discharge rate at the gage site.
- Varying backwater effects as a result of varying water levels in the Nookachamps area.
- Lack of a quantitative basis for extension of the rating curve to high discharges, especially for the 1898 and 1910 floods which occurred prior to the Sterling bend cut-off.

A staff gage at Sedro-Woolley was first installed at the Northern Pacific railroad bridge about 1 May 1908. Water levels were recorded at or close to this location for the 1910, 1918, and 1922 floods. The water level for the 1898 flood was presumably based on a local highwater mark (possibly from the Hart Ranch) transferred to the gage site. However, available information on the determination of the 1898 water level is unclear.

Stage-discharge ratings at Sedro-Woolley were established through a program of direct discharge measurements. Stewart reports a total of 81 discharge measurements between 12 June 1908 and 22 January 1923. The highest discharge measurement appears to have been about 90,000 cfs (Measurement 45). The ratings show significant shifts, are apparently dependent on whether the river is rising or falling, and extrapolation to flows of the magnitude of the historic events is subject to considerable uncertainty. The difficulty of estimating peak flows for large events is compounded by the need to measure or estimate overbank discharges. It appears that Stewart estimated the peak discharge for 1921 from the stage-discharge rating and then added 50,000 cfs to account for overbank flow. This latter figure, which represents about 25% of the estimated total discharge, was calculated from the hydraulic characteristics of the three "slough openings" in the Northern Pacific embankment through which all overbank flow would have had to pass. It is not clear how overbank flow was determined for the other historic floods since the stage-discharge rating appears to provide an estimate of discharge in the main channel only.

The peak discharges for the historic floods estimated by Stewart were reviewed by USGS staff in the early 1950s (Riggs and Robinson 1950, Flynn 1951, Bodhaine 1954, Flynn 1954a). All the reviews recommend various levels of reduction in the peak flows for Sedro-Woolley; however, there does not appear to have been consensus on what level of reduction was appropriate. The final and apparently overriding review by Bodhaine (1954) concluded, on the basis of very little quantitative data, that peak discharges estimated by Stewart for 1918 and 1922 were probably "quite reliable" but that peak flows for the earlier events in 1898 and 1910 were probably about 10% high. Bodhaine points out that the "maximum change of 10.8% seems small when all of the possible sources of errors are considered", and recommends that Stewart's values continue to be used. Bodhaine also notes that "the peaks near Concrete probably should be revised if those near Sedro-Woolley are changed."

Given the poorly defined stage-discharge ratings at Sedro-Woolley together with the uncertainty regarding overbank flows, we have little confidence in the magnitude of the published Sedro-Woolley discharges. We do note however, that nowhere in the available documents is there any indication that the estimates of the historic peak discharges may be low at Sedro-Woolley.

As discussed in Section 3.1 above, the attenuation in historic peak flows between Concrete and Sedro-Woolley suggested by the published data appears unreasonably large. From the point of view of hydraulic conditions, the gage site at Concrete is greatly superior to the site at Sedro-Woolley and given the poor quality of the rating at Sedro-Woolley it is not possible to say with certainty whether peak flows reported for Sedro-Woolley are high or low. Nevertheless, the consensus amongst the USGS reviewers of the 1950s was that the published Sedro-Woolley peak flows were high and if that is the case then peak flow estimates at Concrete must also be high.

4.0 FLOOD FREQUENCY ANALYSES

A key step in estimating the design discharge for flood hazard management on the Skagit is flood frequency analysis of unregulated peak discharges for the Skagit River near Concrete. Two particular aspects of the flood frequency analyses conducted by the Corps were investigated as part of this review: the period of unregulated record used for analysis; and the treatment of historic events.

4.1 Period of Record Used in Frequency Analysis

The Corps' frequency analysis of unregulated peak discharges makes use of data from the four historic floods and a 58-year systematic record from water year 1944 through water year 2004, excluding water years 1992, 1993, and 2003. The period of systematic record used by the Corps was governed by the availability of data to estimate the effects of upstream reservoir regulation.

In review of the Corps' work by Pacific International Engineering (PIE), it was suggested that the effects of reservoir regulation prior to 1944 would be minimal and that the record of unregulated peak flows at Concrete should be extended back to water year 1925 by simply using the observed (regulated) peak flow data. PIE's argument was essentially that in the period 1925 through 1944, there were no requirements for flood control storage at the projects then in place (Lower Baker, Diablo, and Gorge), and that peak flows would be passed through those facilities without attenuation. PIE's position implies that there would be no incidental flood control storage, e.g. as a result of reservoir drawdown due to hydropower operations.

The observed record of annual peak discharges at the Concrete gage, the record of unregulated winter (October-March) peak discharges used by the Corps in their frequency analysis, and the record of unregulated winter peak discharges used by PIE are listed in Table 1. Although PIE assumed that the then-existing projects had no impact on peak flows from 1925 through 1944, USGS Water Supply Paper 1527 contains an estimate of the unregulated peak flow for water year 1932 of 182,000 cfs, compared with an observed peak discharge of 147,000 cfs. No other information has been located on unregulated flows in the 1925 through 1944 time period, however, the data from 1932 indicate potential for incidental storage in that period to have a significant affect on peak discharges. A final column of data is included in Table 1 showing two changes to the data series – replacement of the 1932 data with the estimated unregulated value of 182,000 cfs and addition of an estimated unregulated discharge for the November 2006 (water year 2007) flood of 185,000 cfs (Perkins, personal communication, 2007).

Table 1: Peak Discharge Data, Skagit River near Concrete.

	Observed Data		Unregulated Winter Data			
Water Year	Date	Annual Peak Discharge (cfs)	Date	USACE Peak Discharge (cfs)	PIE Peak Discharge (cfs)	Changes to Data Series (cfs)
1898	19-Nov-1897	275,000	19-Nov-1897	275,000	275,000	
1910	30-Nov-1909	260,000	30-Nov-1909	260,000	260,000	
1918	30-Dec-1917	220,000	30-Dec-1917	220,000	220,000	
1922	13-Dec-1921	240,000	13-Dec-1921	240,000	240,000	
1925	12-Dec-1924	92,500	12-Dec-1924	m	92,500	
1926	23-Dec-1925	51,600	23-Dec-1925	m	51,600	
1927	16-Oct-1926	88,900	16-Oct-1926	m	88,900	
1928	12-Jan-1928	95,500	12-Jan-1928	m	95,500	
1929	09-Oct-1928	74,300	09-Oct-1928	m	74,300	
1930	07-Jun-1930	32,200	20-Feb-1930	m	43,692	
1931	26-Jun-1931	60,600	28-Jan-1931	m	64,145	
1932	27-Feb-1932	147,000	27-Feb-1932	m	147,000	182,000
1933	13-Nov-1932	116,000	13-Nov-1932	m	116,000	
1934	22-Dec-1933	101,000	22-Dec-1933	m	101,000	
1935	25-Jan-1935	131,000	25-Jan-1935	m	131,000	
1936	03-Jun-1936	60,000	05-Jan-1936	m	28,223	
1937	19-Jun-1937	68,300	23-Dec-1936	m	35,698	
1938	28-Oct-1937	89,600	28-Oct-1937	m	89,600	
1939	29-May-1939	79,600	02-Jan-1939	m	70,686	
1940	15-Dec-1939	48,200	15-Dec-1939	m	48,200	
1941	19-Oct-1940	51,000	19-Oct-1940	m	51,000	
1942	02-Dec-1941	76,300	02-Dec-1941	m	76,300	
1943	23-Nov-1942	54,000	23-Nov-1942	m	54,000	
1944	03-Dec-1943	65,200	03-Dec-1943	67,639	67,639	
1945	08-Feb-1945	70,800	08-Feb-1945	70,077	70,077	
1946	25-Oct-1945	102,000	25-Oct-1945	108,844	108,844	
1947	25-Oct-1946	82,200	25-Oct-1946	81,490	81,490	
1948	19-Oct-1947	95,200	19-Oct-1947	85,040	85,040	
1949	13-May-1949	55,700	07-Oct-1948	45,180	45,180	
1950	27-Nov-1949	154,000	27-Nov-1949	163,325	163,325	
1951	10-Feb-1951	139,000	10-Feb-1951	151,668	151,668	
1952	05-Jun-1952	43,500	20-Oct-1951	41,628	41,628	
1953	01-Feb-1953	66,000	12-Jan-1953	79,612	79,612	
1954	31-Oct-1953	58,000	01-Nov-1953	61,187	61,187	
1955	11-Jun-1955	56,300	19-Nov-1954	63,268	63,268	
1956	03-Nov-1955	106,000	04-Nov-1955	124,179	124,179	
1957	20-Oct-1956	61,000	20-Oct-1956	66,910	66,910	
1958	17-Jan-1958	41,400	17-Jan-1958	48,846	48,846	
1959	30-Apr-1959	90,700	03-Dec-1958	82,998	82,998	
1960	23-Nov-1959	89,300	23-Nov-1959	101,118	101,118	
1961	16-Jan-1961	79,000	15-Jan-1961	92,134	92,134	
1962	03-Jan-1962	56,000	03-Jan-1962	73,870	73,870	
1963	20-Nov-1962	114,000	20-Nov-1962	107,280	107,280	
1964	22-Oct-1963	73,800	22-Oct-1963	82,130	82,130	

Table 1 (continued).

Water Year	Observed Data: Date	Observed Data: Annual Peak Discharge (cfs)	Unregulated Winter Data: Date	USACE Peak Discharge (cfs)	PIE Peak Discharge (cfs)	Changes to Data Series (cfs)
1965	01-Dec-1964	52,600	01-Dec-1964	65,127	65,127	
1966	06-May-1966	36,800	06-Oct-1965	44,836	44,836	
1967	21-Jun-1967	72,300	16-Dec-1966	82,256	82,256	
1968	28-Oct-1967	84,200	28-Oct-1967	86,529	86,529	
1969	05-Jan-1969	49,500	05-Jan-1969	65,525	65,525	
1970	04-Nov-1969	38,400	23-Jan-1970	43,335	43,335	
1971	31-Jan-1971	62,200	31-Jan-1971	83,194	83,194	
1972	13-Jul-1972	91,900	13-Mar-1972	63,640	63,640	
1973	26-Dec-1972	49,500	26-Dec-1972	58,079	58,079	
1974	16-Jan-1974	79,900	16-Jan-1974	122,033	122,033	
1975	21-Dec-1974	57,500	21-Dec-1974	63,929	63,929	
1976	04-Dec-1975	122,000	03-Dec-1975	150,068	150,068	
1977	18-Jan-1977	58,400	18-Jan-1977	70,984	70,984	
1978	02-Dec-1977	70,300	02-Dec-1977	74,635	74,635	
1979	08-Nov-1978	46,000	08-Nov-1978	59,164	59,164	
1980	18-Dec-1979	135,800	18-Dec-1979	144,608	144,608	
1981	26-Dec-1980	148,700	26-Dec-1980	163,438	163,438	
1982	21-Jun-1982	51,700	15-Feb-1982	67,853	67,853	
1983	04-Dec-1982	101,000	04-Dec-1982	83,792	83,792	
1984	05-Jan-1984	109,000	04-Jan-1984	111,577	111,577	
1985	07-Jun-1985	46,100	03-Nov-1984	42,000	42,000	
1986	19-Jan-1986	93,400	19-Jan-1986	104,351	104,351	
1987	24-Nov-1986	83,500	24-Nov-1986	78,609	78,609	
1988	24-Nov-1987	39,600	10-Dec-1987	44,891	44,891	
1989	16-Oct-1988	74,100	16-Oct-1988	89,300	89,300	
1990	04-Dec-1989	119,000	10-Nov-1989	137,739	137,739	
1991	10-Nov-1990	149,000	10-Nov-1990	200,072	200,072	
1992	29-Apr-1992	53,300	01-Feb-1992	m	54,343	
1993	13-May-1993	39,300	23-Mar-1993	m	40,637	
1994	03-Mar-1994	36,500	02-Mar-1994	57,927	57,927	
1995	20-Dec-1994	59,800	20-Dec-1994	78,793	78,793	
1996	29-Nov-1995	160,000	29-Nov-1995	185,733	185,733	
1997	09-Jul-1997	91,400	19-Mar-1997	104,655	104,655	
1998	05-Oct-1997	76,700	05-Oct-1997	75,040	75,040	
1999	13-Dec-1998	61,400	13-Dec-1998	81,043	81,043	
2000	12-Nov-1999	103,000	12-Nov-1999	135,037	135,037	
2001	20-Oct-2000	30,900	20-Oct-2000	42,670	42,670	
2002	08-Jan-2002	94,300	08-Jan-2002	125,293	125,293	
2003	26-Jan-2003	65,500	26-Jan-2003	m	65,171	
2004	21-Oct-2003	166,000	21-Oct-2003	185,685	185,685	
2005	11-Dec-2004	99,400	m	m	m	
2006	m	m	m	m	m	
2007	07-Nov-2006	145,000	06-Nov-2006	m	m	185,000

Frequency analyses were conducted on the unregulated winter peak discharge data of Table 1 using the Corps' HEC-FFA software following the guidelines of Bulletin 17B. Results for winter floods with return period from 2- to 100-years are provided below along with: 1) analysis of the PIE data with adjustment to the 1932 data and addition of the 2007 data, and 2) analysis of the Corps' data with the addition of the 2007 data. The differences between the various analyses are minimal. In our opinion, the Corps' approach of discounting the systematic record prior to 1944, is acceptable and has only minor impact on the results of the frequency analyses.

Run	Data Series	Peak Discharge by Return Period (cfs)				
		2-year	**10-year**	**25-year**	**50-year**	**100-year**
1	USACE: 4 historic events + 58-yr systematic record	84,400	158,000	203,000	242,000	284,000
2	PIE: 4 historic events + 80-yr systematic record	80,300	152,000	196,000	232,000	272,000
3	PIE Adjusted: As Run 2 with 1932 adjusted and 2007 added	81,200	156,000	201,000	239,000	281,000
4	USACE Adjusted: As Run 1 with 2007 added	86,000	160,000	205,000	242,000	282,000

4.2 Treatment of Historic Data

The flood frequency analyses conducted by the Corps follow the guidelines of USWRC Bulletin 17B. This is the widely accepted standard approach to flood frequency analysis. However, as pointed out by several researchers (e.g. Stedinger and Cohn 1986), the Bulletin's approach to treatment of historic data is inefficient and Bulletin 17B itself (page 28) acknowledges the need for "Alternative procedures for treating historic data".

More comprehensive treatment of historic data is available through the EMA (Expected Moments Algorithm) software package developed by the US Bureau of Reclamation (England 1999). This package was developed specifically to make better use of the information typically available for historical or paleo floods. EMA supports more efficient statistical parameter estimation procedures than Bulletin 17B, allows analysis of historic data with multiple thresholds for multiple historical periods, and allows for the explicit incorporation of uncertainty in historical flood estimates through specification of a range of discharges for those estimates. It also provides for considerably greater flexibility in terms of the type of historical or paleo flood data that can be incorporated into flood frequency analyses. For example, in addition to the ability to specify a range for flood magnitude, EMA can incorporate information that no floods exceeded some specified level, or that a flood exceeded some level but its magnitude is unknown. None of this type of information can be used in the Bulletin 17B procedures.

Improvements in Bulletin 17B are being actively considered by the Hydrologic Frequency Analysis Work Group. This is a work group of the Hydrology Subcommittee of the Advisory Committee on Water Information (ACWI), for which the USGS is the lead agency (see http://acwi.gov/aboutus.html). The Hydrologic Frequency Analysis Work Group (HFAWG) was formed in December 1999 to recommend improvements or alternatives to the Bulletin 17B procedures. The work program of the HFAWG specifically includes evaluation of EMA and comparison of EMA against Bulletin 17B. The HFAWG includes members from Federal agencies, academia, consultants, and several special interest groups. Members include representatives from FEMA, US Army Corps of Engineers, USGS, and Michael Baker amongst others.

A number of exploratory flood frequency analyses were performed using EMA on the Corps' record of unregulated instantaneous peak flows for the Skagit River near Concrete. These analyses, and the resultant 100-yr peak flow estimates, included the following (note that years referenced are water years):

Run	**Assumptions**	**Q_{100} (cfs)**
1	Base Case: 107-yr period 1898 – 2004 46-yr historic record 1898 – 1943 4 historic peaks 1898, 1910, 1918, 1922 61-yr systematic record 1944 – 2004 3 yrs missing in systematic period (1991, 1992, 2003)	284,000
2	Extend Period back to 1870: 135-yr period 1870 – 2004 29-yr first historic period 1870 – 1898 1 historic peak above first threshold 1898 45-yr second historic period 1899 – 1943 3 historic peaks above second threshold 1910, 1918, 1922 61-yr systematic record 1944-2004 3 yrs missing in systematic record (1991, 1992, 2003)	276,000
3	Uncertain Gage Heights: As Run 2 but with range of values for historic data reflecting uncertain gage height (see Section 5.0 for details).	248,000
4	Eliminate 1898: As Run 1 but 106-yr period 1899 – 2004 (eliminate 1898)	266,000
5	Uncertain Gage Heights, Uncertain Roughness: As Run 2 but with range of values for historic data reflecting uncertain gage height and uncertain roughness (see Section 5.0 for details).	241,000

Note that the base case 100-year discharge in the above table above agrees exactly with the Corps analysis performed using the HEC-FFA software for the flood insurance study. Note also that the above figures do not include the expected probability adjustments applied to the Corps frequency analyses for the flood damage reduction feasibility study.

Quite a strong case can be made for extending the period of historic information for the Concrete gage back to 1870 (Run 2). Reports from the time of the 1898 flood (compiled by Kunzler 2006) seem to be reasonably consistent in claiming this to be the largest flood since settlement of the valley around 1870. However, it is not known whether the period 1870 – 1897 included flood events larger than the other historic floods in the record (i.e. 1910, 1918, and 1922). Since EMA can handle multiple historical periods with multiple flood thresholds, it is now possible to analyze the situation where the 1898 flood is the largest in the period 1870 – 1898, and the 1910, 1918, and 1922 floods are the largest in the period 1899 – 1943. This type of analysis is not possible under the Bulletin 17B procedures of HEC-FFA.

A strong case could also be made for incorporating uncertainty in to the analyses of historic data (Run 3 and Run 5) as discussed in more detail in the following Section 5.0.

5.0 UNCERTAINTY

USGS staff have repeatedly stressed that all discharge measurements are uncertain and, depending on circumstances, may be good to only within ±25%. Furthermore, upon review, the USGS has taken the position that measurements of peak discharges for the historic floods of 1898, 1910, 1918, and 1922 for the Skagit River near Concrete should not be downgraded and will remain part of the official record. The US Army Corps of Engineers has in turn accepted the USGS position and has determined that the historic events be incorporated into its analysis of flood risk in the Skagit Valley.

We agree with the USGS and the US Army Corps of Engineers basic positions with respect to the historic events for the following rather simple reasons:

- there is convincing evidence that significant floods occurred in those years
- exclusion of those data from the analyses could result in an understatement of flood risk

We are also of the opinion that uncertainty should be incorporated into the analysis of flood risk in the Skagit Valley and that planning for flood hazard management, including the current flood damage reduction study, should incorporate safe-fail features.

The principal challenge to incorporating uncertainty into the analyses is characterizing that uncertainty. Incorporation of uncertainty into flood frequency analyses can be achieved using the EMA software by specifying a range of values for the historic flood events. The pre-settlement floods of 1815 and 1856 could also be included in analyses with EMA provided a suitable range of values for those events could be agreed on and provided that the historical time period for which the 1815 flood was the largest could be established.

Our review of the historic data has identified or confirmed a number of indications that the discharge estimate for the December 1921 flood is likely high. Given the manner in which the discharge estimates for other historic events are dependent on the December 1921 estimate, this would imply that discharge estimates for the historic events of water years 1898, 1910, and 1918 are also high. The various indications that the December 1921 peak flow estimate is likely high include the following:

- the discharge estimate is inconsistent with extrapolation of the established stage-discharge rating and plots to the right of the curve (i.e. the discharge is higher than would be expected from the rating for the reported gage height).
- no account has been made for the drop in water level between the old and new gage sites.
- the “n” verification study of 1950 indicates that the published peak discharge is high.

- the reported attenuation in peak discharges between Concrete and Sedro-Woolley appears to be excessive.

(Evidence that the Smith House in Hamilton did not flood in 1921 when it was flooded at an appreciably lower discharge in 1995 has been cited by others as a further indication that the published peak discharge for 1921 is high. However, as discussed in Section 2.3 above, changes in hydraulic conditions downstream from Hamilton appear to have significantly reduced the channel conveyance capacity between 1921 and present. With the information currently available it is not possible to say with certainty whether the Smith House would or would not have been flooded in 1921 at a discharge of 240,000 cfs).

While each of the above points could be argued, the preponderance of the evidence suggests that the current estimate for the December 1921 peak discharge is toward the high end of the range of plausibility.

We have attempted to identify specific indications that the December 1921 peak discharge estimate may be low. The only specific source of understatement appears to be the possibility of flow bypassing the gage site at Concrete on the right overbank and being unrecorded. However, there is no known evidence of bypassed flows and it could (and has) been argued that reported water levels or mainstem discharges that imply bypassed flows are, in the absence of evidence of such bypass, simply another indication that those water levels or discharges are overestimated. Given the published gage heights, a bypass is obviously most likely to have occurred in 1898.

The "n" verification studies notwithstanding, uncertainty also remains in the roughness at the higher discharges experienced in 1921 and earlier historic events. The value of 0.030 determined in 1950 is probably good to within ±0.002, corresponding to uncertainty in the computed discharge of ± 7%.

Exploratory frequency analyses were conducted with EMA using a range of values for the historic floods to reflect uncertainty. Two alternative analyses were carried out: 1) with uncertain gage height and fixed roughness, and 2) with uncertain gage height and uncertain roughness. It should be stressed that at this point, these analyses are primarily for illustrative purposes. Further work would be required to establish defensible flow ranges for the historic events.

Uncertain gage height and fixed roughness

For the floods of 1910, 1918, and 1922, the top of the range was taken as the published discharge value, and the bottom end of the range was taken as the discharge values indicated by the current rating for the published gage height less 2 ft to account for the fall in water level between the old and new gage sites at Concrete. A similar approach was used for the 1898 flood expect that a nominal 15,000 cfs was added to the top end of the range to account for flows possibly bypassing the gage site. The range of discharge values assumed for historic events is thus as follows:

Flood Event	Range of Peak Flow (cfs)	
November 1897	229,000	290,000
November 1909	210,000	260,000
December 1917	179,000	220,000
December 1921	196,000	240,000

The frequency curve as estimated by EMA with uncertain historic data values together with the discharge data is shown in Figure 7. With the above range of discharges for the historic floods and assuming that the 1898 flood is the largest in the period 1870 to 1898, the estimate of the 100-year unregulated discharge is reduced from 284,000 cfs to 248,000 cfs.

Uncertain gage height and uncertain roughness

The effects of uncertain roughness were approximately accounted for by widening the above range of historic discharges by ±7%, representing uncertainty in Manning's roughness of ±0.002. The discharge values for the historic events were assumed as follows:

Flood Event	Range of Peak Flow (cfs)	
November 1897	213,000	310,000
November 1909	195,000	278,000
December 1917	166,000	235,000
December 1921	182,000	257,000

With the above range of discharges for the historic floods and assuming the 1898 flood is the largest in the period 1870 to 1898, the estimate of the 100-year unregulated discharge is reduced from 284,000 cfs to 241,000 cfs.

6.0 OTHER CONSIDERATIONS

6.1 Reservoir Regulation

The usual practice amongst the various regulatory agencies involved in flood hazard management is to assume that only dedicated or mandatory flood control storage should be considered when analyzing the effects of reservoir regulation on downstream flood flows.

The Corps hydrologic analyses (USACE 2004b, Section 4.4) assumes the availability of 120,000 acre ft of flood control storage at Ross Dam and 74,000 acre ft of flood control storage at Upper Baker Dam. It is assumed that no flood control storage is available at other projects. Required flood control storage at Ross and Upper Baker varies seasonally as follows:

Date	**Required Flood Control Storage (acre ft)**	
	Upper Baker	**Ross**
October 1	0	0
October 15	8,000	20,000
November 1	16,000	43,000
November 15	74,000	60,000
December 1	74,000	120,000
March 1	74,000	120,000

Note that the full amount of flood control storage assumed in the Corps analysis is not actually required until December 1 of each flood control season. Many of the large floods in the historical record have however occurred in November or even October (e.g. the flood of October 2003) when required flood control is substantially less than that assumed by the Corps.

If the Corps analyses were intended to rely only on required flood control storage, then the analyses to date would overstate the degree of flood control regulation afforded by the upstream reservoirs and by Ross in particular. In practice, we expect that normal power generation practices would result in the Ross and Baker reservoirs being drawn down more than required under the current flood control operating curves prior to the start of the flood control season. However, such "incidental" flood control storage is typically not relied on in either delineation of flood hazards or in the design of flood control projects.

6.2 Hypothetical Flood Events

The Corps' hydrologic and hydraulic analyses rely on a series of hypothetical flood hydrographs for future regulated conditions for a range of return periods from 25- to 500-years. These hydrographs were developed by mean of a complex chain of regression and flood frequency analyses, the details of which are described in the Corps' Hydrology Technical Documentation (USACEb, 2004). The approach requires development of hypothetical flood hydrographs for the various sub-basins of the system, routing of hydrographs through the system's reservoirs consistent with flood control operating policies, addition of inflows downstream of the system reservoirs and then routing to Concrete and Sedro-Woolley.

Development of the hypothetical hydrographs relied heavily on regression relationships between peak instantaneous, peak one-day, and peak three-day discharges. The Corps analyses show high degrees of correlation between the instantaneous peak and one-day vales and between the one-day and three-day values which are somewhat misleading.

Taking regression between one-day and three-day values as an example, in most of the basic data analyzed by the Corps, the annual one-day peak is embedded in the annual three-day peak. Regression between one- and three-day values is thus effectively a regression of the one-day peak against itself plus flows for two adjacent days, a procedure which leads to a spuriously high correlation coefficient. Data for the Sauk River near Sauk (USGS gage 121895000, period of record water years 1930 – 2005) were analyzed to assess the effects of this shortcoming. Regression of the one-day winter peak flow against the three-day winter peak flow resulted in a correlation coefficient of 0.92. Regression of the one-day peak flow against the flows for the adjacent two days gave a correlation coefficient of 0.75. Similar results would be expected from analysis of data from other gage sites. A similar issue also arises in regression between one-day peak and instantaneous peak discharges.

The Corps' hypothetical hydrographs assume a nested construction with the T-year instantaneous, one-day and three-day peak discharges all contained within the same event. As a result of overstatement of the regression coefficients, the Corps' hydrographs appear to exhibit a somewhat conservative combination of high peak and high volume which will result in some overestimation of routed peak discharges. This overestimation is likely small compared with other sources of uncertainty, and in our opinion the Corps' approach to synthesis of hypothetical events is appropriate given the practical difficulties of alternatives techniques, such as generation of hydrographs by rainfall-runoff modeling.

7.0 CONCLUSIONS AND RECOMMENDATIONS

Based on a focused review of the large body of information on the hydrology and hydraulics of the Skagit River, we are of the opinion that estimates of the peak discharges for the historic flood events of water years 1898, 1910, 1918, and 1922 should continue to be incorporated in analyses of flood hazard and flood hazard management in the Skagit Valley. We are also of the opinion that uncertainty in the magnitude of the historic floods should be accounted for in future hydrologic analyses.

The estimated peak discharge for the 1922 flood is of critical importance to flood hazard management since estimates of the peak discharges for the other historic events are directly dependent on the estimate for the 1922 event. The peak discharge estimates for the historic events collectively determine the magnitude of the 100-year discharge, which in turn is the single most important hydrologic parameter for the flood damage reduction feasibility study and the flood insurance study.

It is widely recognized that the peak discharge estimates for the historic events are uncertain. Review of various factors affecting the discharge estimates indicates that the published peak discharge for the 1922 flood of 240,000 cfs at the Concrete gage is most likely toward the high end of the range of uncertainty. We tentatively estimate that the peak discharge for the 1922 flood falls in the range of 180,000 cfs to 260,000 cfs. This is between 75% and 108% of the published value of 240,000 cfs, and between 80% and 116% of the best estimate of the peak discharge for this event of 225,000 cfs as determined by Benson in 1952. Similar tentative ranges have been estimated for the other historic events. Further work is required to establish agreement on defensible flow ranges in consultation with the USGS and US Army Corps of Engineers.

Exploratory frequency analyses of the instantaneous unregulated peak flows at Concrete have been conducted with the EMA (Expected Moment Algorithm) software package. EMA supports more comprehensive and more flexible frequency analysis of historic and paleo flood data than is available in the standard approach to flood frequency analysis using the procedures of USWRC Bulletin 17B. The exploratory analyses with EMA indicate that more rigorous frequency analyses, incorporating uncertainty in the historic peak discharge estimates and taking advantage of EMA's ability to handle multiple historic periods with multiple flood thresholds, could result in a 10% to 15% reduction in the estimate of the 100-year peak **unregulated** discharge. With our current estimates for the range of uncertainty of the historic flood magnitudes, EMA analysis would result in an estimated 100-year peak unregulated discharge for the Skagit River near Concrete of between approximately 240,000 and 250,000 cfs, compared with the current estimate of 284,000 cfs.

Recommendations arising from this review are as follows:

1) Given the past occurrence of major storms early in the flood control season, agreements should be negotiated with Seattle City Light and with Puget Sound

Energy to ensure the availability of 120,000 acre-ft of flood control storage at Ross Dam and 74,000 acre-ft of flood control storage at Upper Baker Dam by no later than November 1 of each flood control season. Consideration should be given to conditioning flood control storage requirements in the early part of the flood control season on watershed moisture conditions and intermediate term weather forecasts.

2) The County should seek clarification from the USGS regarding the potential for proposed paleoflood studies to contribute to a more reliable characterization of flood risk. The USGS has previously proposed a paleoflood study which targets the pre-settlements floods of around 1856 and 1815. From currently available information, it is not clear whether the proposed work can be expected to both produce estimates of the magnitude of these events **and** establish a time period within which the 1815 flood was the largest such event. Information on both magnitude and time frame are necessary for risk-based analysis.

3) The County should determine whether the potential for a 10% to 15% reduction in the 100-year peak **unregulated** discharge, based on more rigorous flood frequency analysis, warrants additional investment in hydrologic and hydraulic studies.

4) More rigorous flood frequency analysis using the EMA software holds the potential for producing more defensible flood quantile estimates by accounting for uncertainty in the magnitude of historic flood discharges. Should the County decide to proceed with additional hydrologic and hydraulic studies (see Recommendation #3 above), discussions should be initiated with the USGS, the US Army Corps of Engineers, and FEMA to explore the technical and institutional feasibility of using EMA to refine the current estimates of the design flood quantiles for the Skagit River near Concrete. Initial discussions should focus on:

    - the merits of using EMA as opposed to Bulletin 17B,
    - the acceptability of EMA to the three agencies,
    - institutional issues resulting from possible revisions to the current estimates of flood quantiles, including scheduling and budgetary considerations,
    - the process for review and acceptance of revised flood quantiles, and,
    - characterization of the range of uncertainty in historic peak flow values.

8.0 REFERENCES

Benson, M.A. and F.J. Flynn, 1952. Skagit River near Concrete. USGS memo August 1952.

Bodhaine, G.L., 1954. Skagit River Flood Peaks. Memorandum of Review. USGS memo 13 May 1954.

Burlington, City of, 2007. Preliminary Investigation into Historic Flood Marks in the "Smith" House, Hamilton, Washington. Undated report.

England, J.F., Jr., 1999. Draft User's Manual for EMA (Expected Moments Algorithm) Beta Release. At-Site Flood Frequency Estimation with Historical/Paleohydrologic Data. U.S. Bureau of Reclamation, Technical Service Center, Flood Hydrology Group, Denver, Colorado.

Flynn, F.J., 1951. Skagit River near Sedro-Woolley. Revision of historic flood peaks. USGS memo 25 January 1951.

Flynn, F.J., 1954a. Skagit River at Sedro-Woolley, Wash. Historic Flood Peaks. USGS memo 15 July 1954.

Flynn, F.J., 1954b. Skagit River near Concrete, Wash. Historic Flood Peaks. USGS memo 16 July 1954.

Jarrett, R.D. and H.E. Petsch, Jr., 1985. Computer Program NCALC User's Manual – Verification of Manning's roughness coefficient in channels. USGS Water-Resources Investigations Report 85-4317.

Kunzler, L.J., 2006. James E. Stewart Skagit River Flood Reports and Assorted Documents. A Citizen Critical Review Whitepaper. Available at www.skagitriverhistory.com, 11July 2006.

Mastin, M.C. and D.L. Kresch, 2005. Verification of 1921 Peak Discharge at Skagit River near Concrete, Washington, Using 2003 Peak-Discharge Data. U.S. Geological Survey Scientific Investigations Report 2005-5029.

Pacific International Engineering Ltd., 2005. Hydrology and Hydraulics. Skagit River Flood Basin – Existing Conditions. Report prepared for Skagit County.

R2 Resource Consultants Inc., 2004. Baker River Hydroelectric Project (FERC No. 2150). Hydrology and Geomorphology of the Baker and Middle Skagit Rivers (Study A-24). Part 2: Sediment Transport and Channel Response. Unpublished report prepared for Puget Sound Energy Inc.

Riggs, H.C. and W.H. Robinson, 1950. Proposed Revision of Skagit River Flood Peaks. USGS memo 16 November 1950.

Stedinger, J.R. and T.A. Cohn, 1986. Flood Frequency Analysis with Historical and Paleoflood Information. Water Resources Research, 22(5):785-793.

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

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

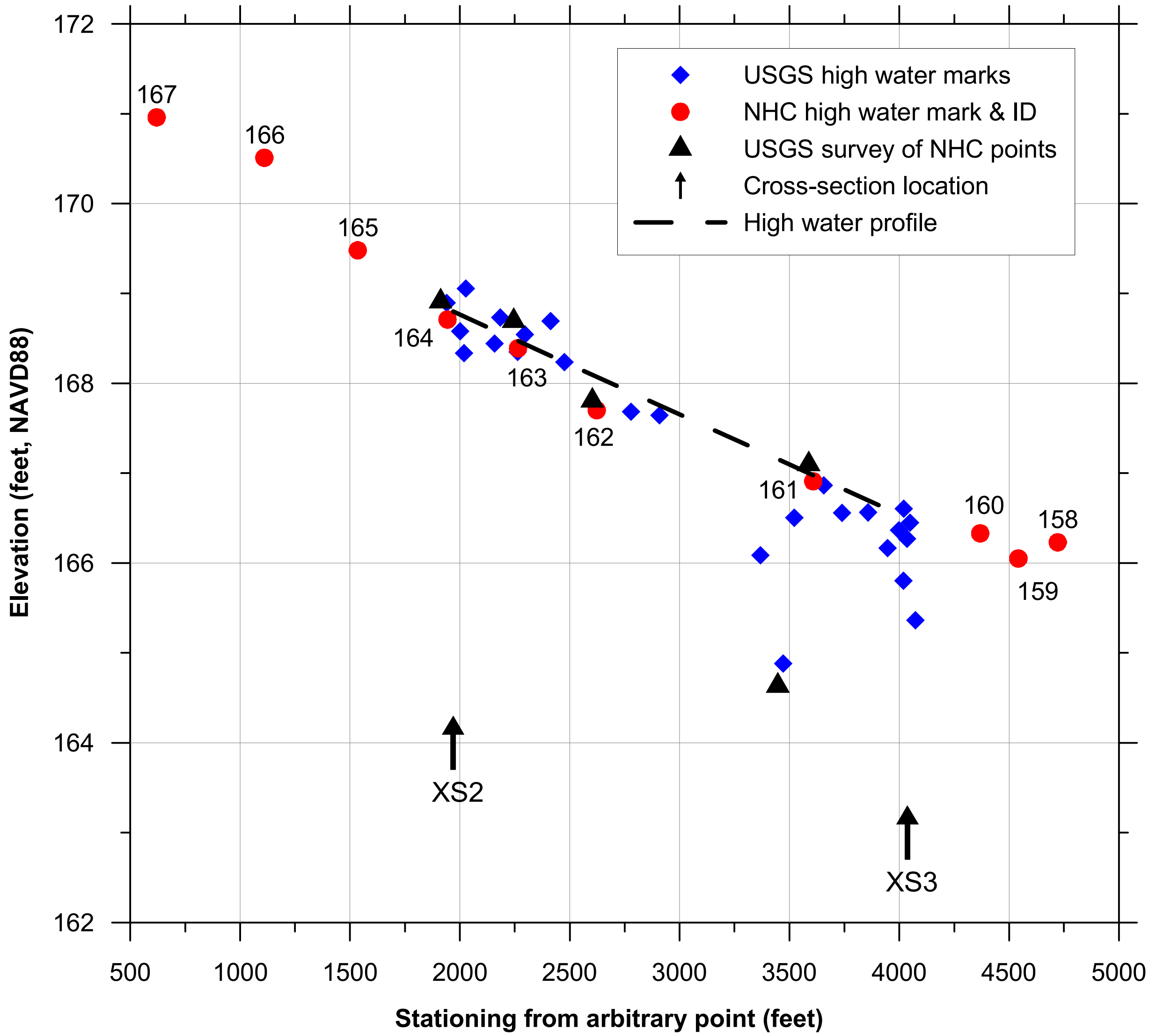


Figure 1: High water marks and water surface profile for flood of 7 November 2006, Skagit River near Concrete

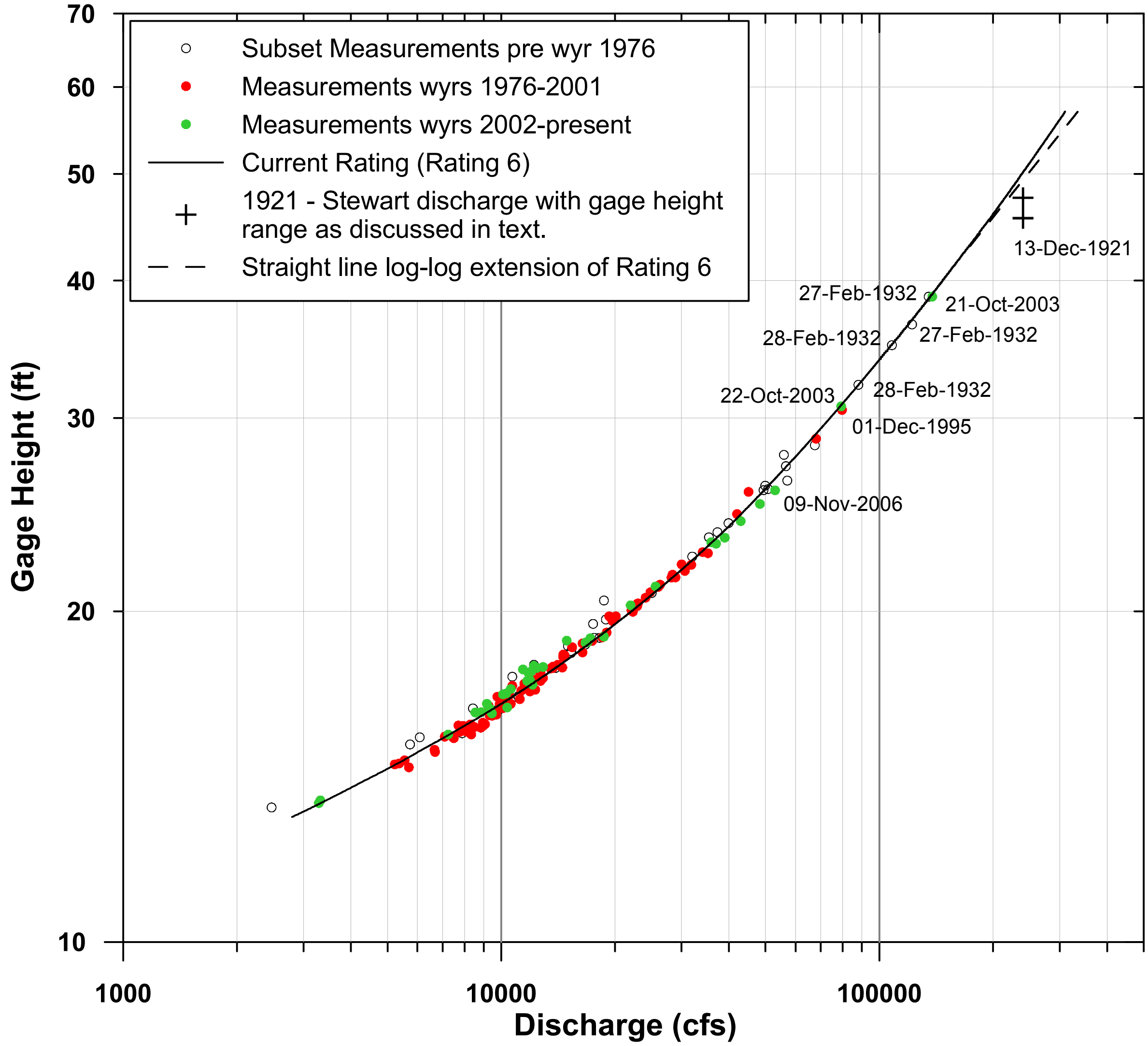
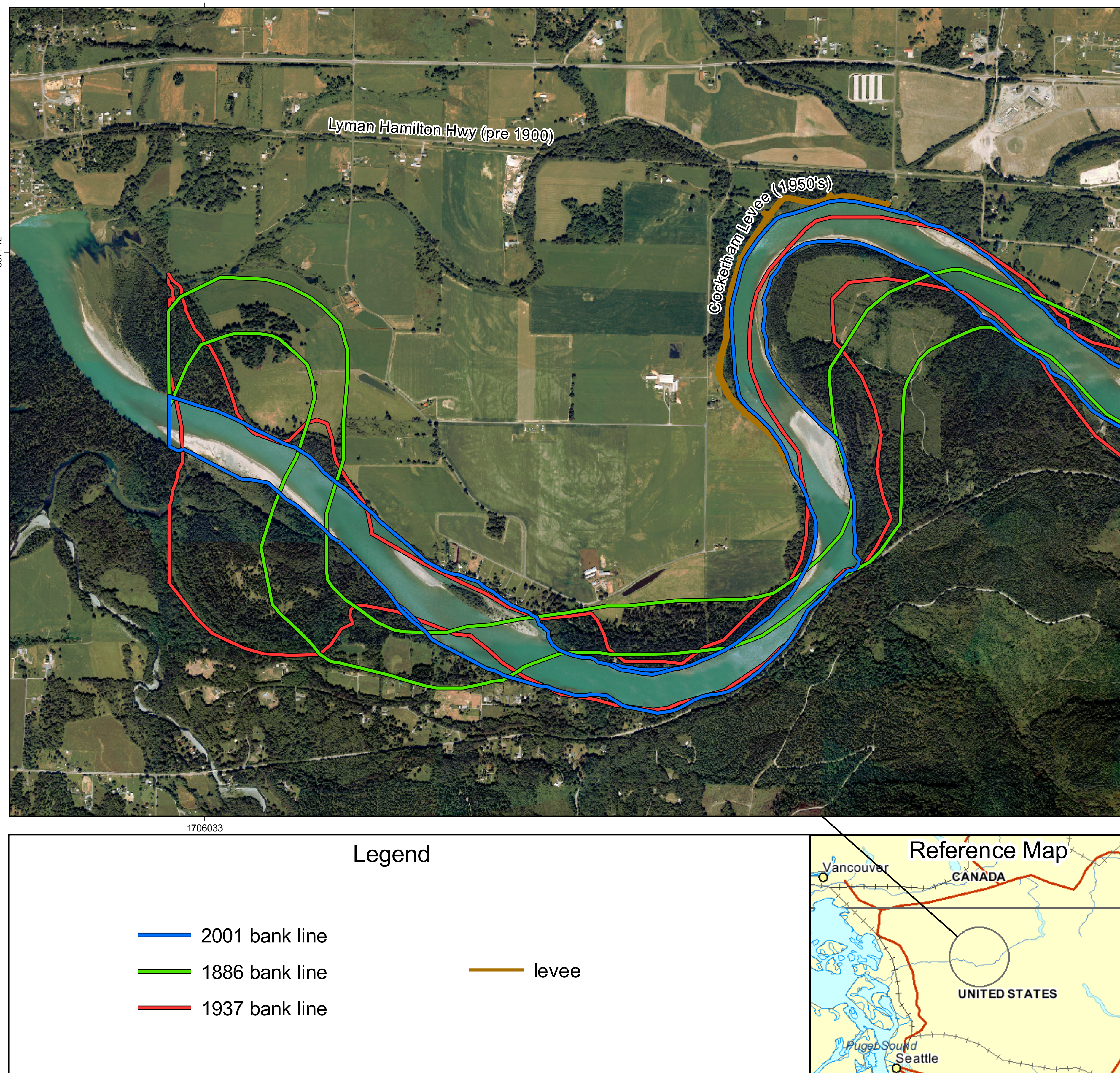


Figure 2: Stage-discharge rating, Skagit River near Concrete



SKAGIT RIVER

Skagit River Channel Change Over Time

coord. syst.: State Plane Wash N	horz. datum: NAD 27	horz. units: Foot
northwest hydraulic consultants	project no. 21487	26-Jan-2007

nhc, G:_proj\21487_Skagit\Skagit_Final.mxd

Figure 3

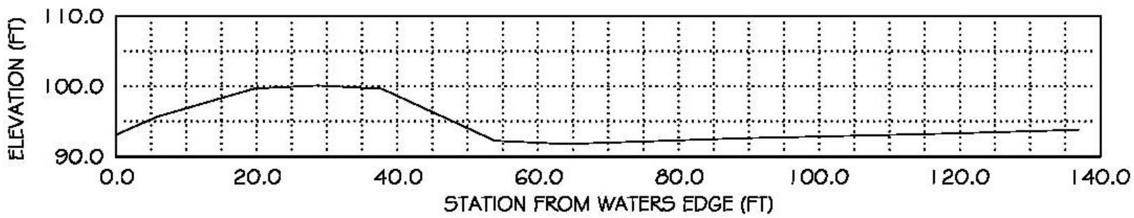


Figure 4: Representative Section through Cockerham Levee (looking d/s)

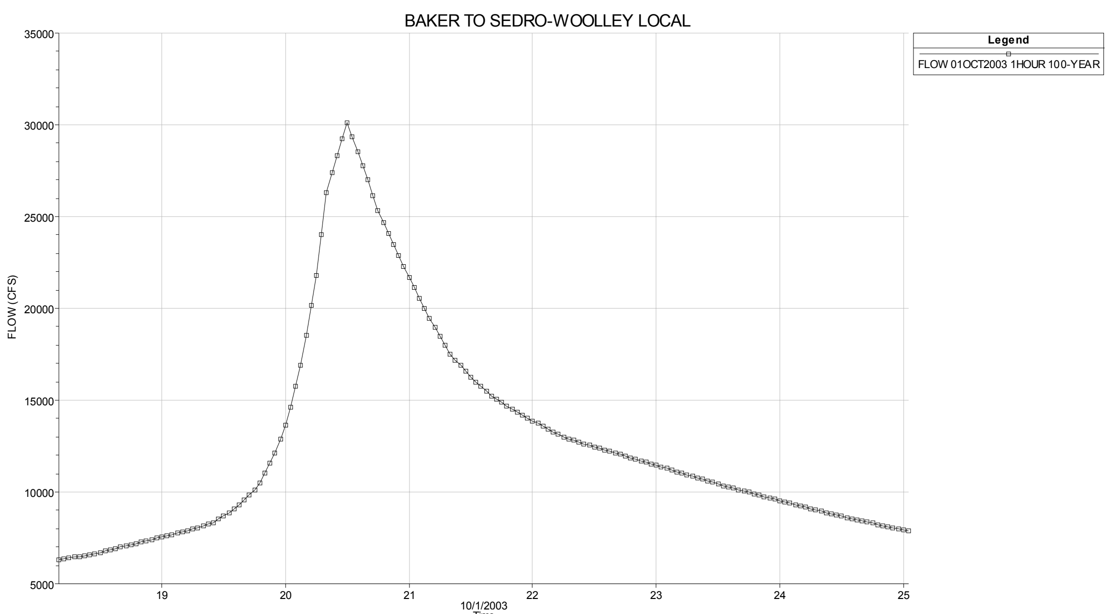


Figure 5: 100-year Local Inflow Hydrograph Concrete to Sedro-Woolley

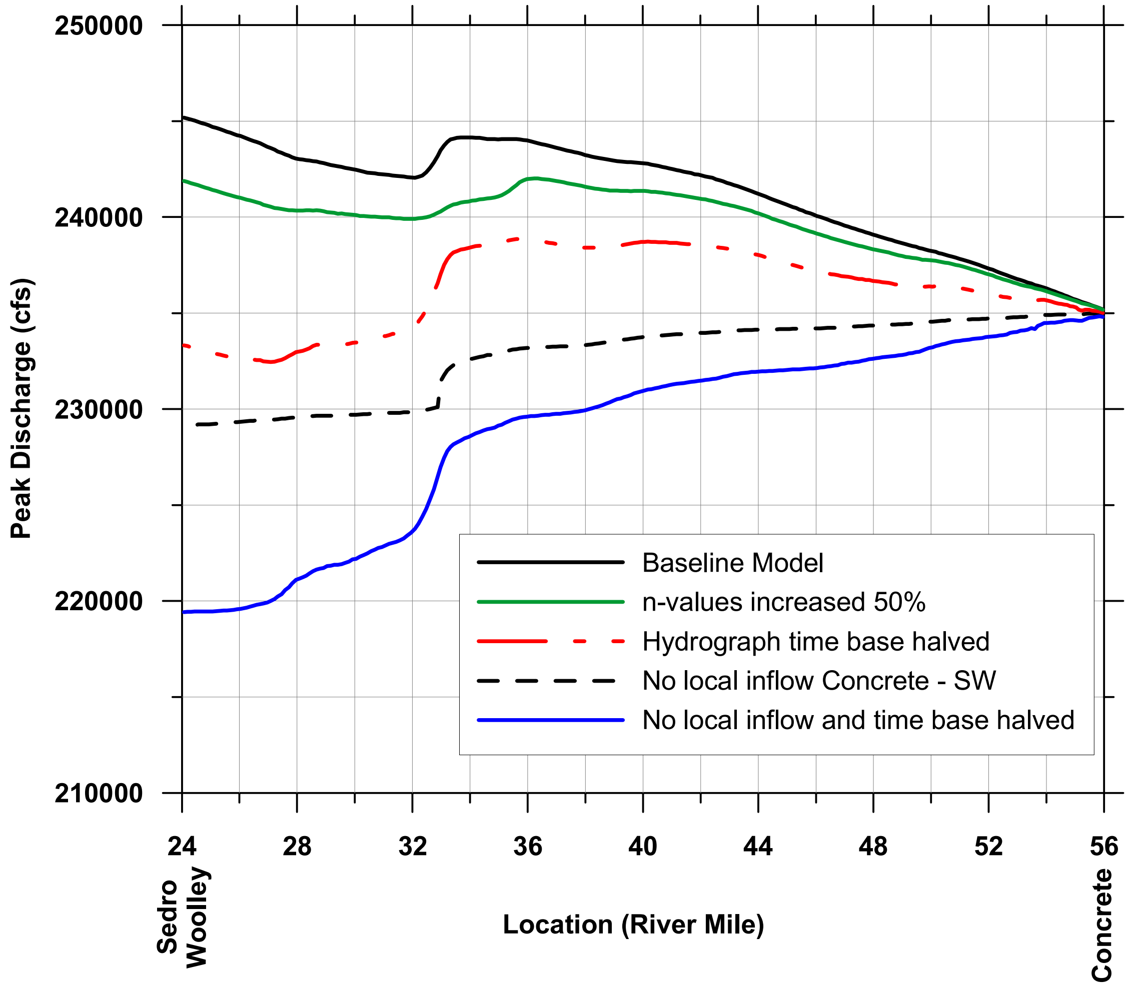


Figure 6: 100-year Regulated Peak Discharge Profiles

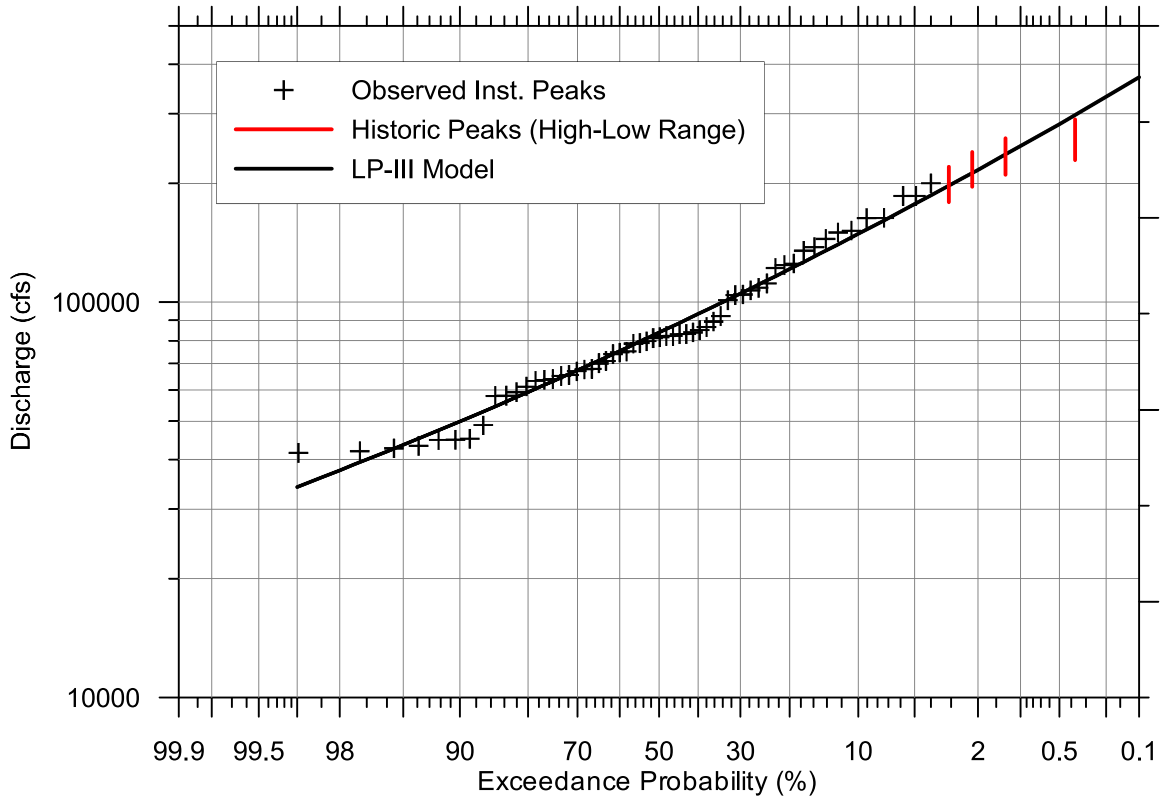


Figure 7: Annual Frequency Analysis, Skagit River near Concrete Unregulated Instantaneous Peak Discharge – Run 3