Technical Report - Supporting Data and Analysis for Skagit River RFIS Appeal

March 2011

Prepared For:

City of Burlington
City of Mount Vernon
City of Sedro-Woolley
Town of La Conner

By:

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Technical Report - Supporting Data and Analysis for Skagit River RFIS Appeal

Skagit County, Washington

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1.0 Introduction

FEMA released the preliminary package of the revised Digital Flood Insurance Rate Map (DFIRM) and the Revised Flood Insurance Study (RFIS) report for Skagit County on July 1, 2010 (FEMA 2010). This preliminary release includes an update for the lower basin of the Skagit River below the City of Sedro-Woolley, from just downstream of the Highway 9 Bridge to the confluences of the North and South Forks of the Skagit River with Puget Sound. This update only includes the floodplain and base flood elevations (BFEs). FEMA did not develop a floodway for the lower basin at this time. The FEMA report and DFIRMs were prepared by the U.S. Army Corps of Engineers, Seattle District (USACE).

This Technical Report documents supporting data and analyses to show that the FEMA revised BFEs for the lower basin of the Skagit River presented in the FEMA released preliminary DFIRM and RFIS report are "scientifically and technically incorrect". The term "scientifically and technically incorrect" is as specified in the FEMA published guidelines entitled "Appeals, Revisions, and Amendments to National Flood Insurance Program Maps" dated December 2009 (http://www.fema.gov/library/viewRecord.do?id=4053). This Technical Report was prepared by Pacific International Engineering (PI Engineering) and provides a concise summary of the data and analyses leading to the "scientifically and technically incorrect" determination, as well as an overview of the impact on Base Flood Elevations of corrected data and analyses.

PI Engineering believes that the revised BFEs are scientifically and technically incorrect due to the following reasons:

1. Historic flood data and one major flood in the systematic record included in the flood frequency determinations were incorrectly estimated and led to severe overestimation of flood peaks, and
2. Inconsistent levee methodology and poor-quality topographic data were used in the hydraulic analysis.

This technical report will address these two reasons in the order listed.

2.0 Poor-Quality Hydrologic Data Used in the Hydrologic Analysis

PI Engineering has identified the following unacceptable hydrologic data used in the flood frequency analysis that supports the FEMA RFIS include:

A. Peak discharge estimates for the 1897, 1909, 1917 and 1921 historical floods

B. Unregulated peak discharge estimate for the 1932 flood

A discussion of the 1897, 1909, 1917 and 1921 historical floods, and the results of several methods of reevaluating the magnitudes of these floods, follows in sections 2.1, 2.2, and 2.3. Section 2.4 addresses the unregulated estimate for the 1932 flood.

2.1 Peak Discharges for the 1897, 1909, 1917 and 1921 Historical Floods

2.1.1 USGS Published Peak Discharges for the 1897, 1909, 1917 and 1921 Historical Floods

The RFIS report (FEMA 2010) indicates that the hydrologic analysis for the Skagit River (River Mile 22.4 to 56.61) was based on the most recent USGS published peak discharges developed for the Skagit River near Concrete at River Mile 54.1, including four historic flood events that occurred in 1897, 1909, 1917 and 1921. These peak flows were originally estimated by J. E. Stewart in 1923 (Stewart, 1923); published in USGS Water Supply Paper 1527 (USGS, 1961), and recently revised slightly downward in USGS Scientific Investigation Report (SIR) 2007-5159 (USGS, 2007). The data from the SIR report were used for the RFIS flood frequency analysis. Table 1 summarizes these four historical flood peak discharge estimates.

Table 1. Historical flood peak discharges for Skagit River near Concrete

Date of Historical Flood Event	USGS 2007-Published Peak Discharge (cfs)
Nov. 19, 1897	265,000
Nov. 30, 1909	245,000
Dec. 30, 1917	210,000
Dec. 13, 1921	228,000

2.1.2 Comparison of Stewart Surveyed Historical Flood Marks to HEC-RAS Modeled Flood Stages Based on USGS Published Peak Discharges for the 1897, 1909, 1917 and 1921 Historical Floods

Two HEC-RAS models developed by PI Engineering, one for the Concrete reach (Dalles upstream to the Baker River) and the other for the Hamilton-Lyman reach of the Skagit River (approximately 14 miles downstream from the Dalles), were used to model the water surface elevations for the USGS-published historical flood peak discharges. The flood peak discharges are not expected to vary significantly from Concrete to Hamilton-Lyman, as the additional drainage area and floodplain storage between Concrete and Hamilton-Lyman are small and insignificant. Both models were calibrated for observed high water marks, and use the U.S Army Corps of Engineers (COE) 1911 surveyed channel data to model the historical flood stages under the early 1900's stream conditions. Details of the model development are presented in Technical Memorandum – Hydraulic Analysis, Smith House Flood Stages (PI Engineering, 2007, included in the CD that provides the hydrology supporting data as described in Section 2.7), for the Hamilton-Lyman reach model, and in Section 2.3.3 of this technical report for the Concrete reach model.

Documentation of Stewart's surveyed historical flood marks are presented in his hand-written field notes (Stewart, 1922-23). Figures 1 and 2 show the location and the water surface elevations of Stewart-surveyed flood marks (or highwater marks, HWMs) in the Concrete area and in the Hamilton-Lyman area, respectively. Figure 2 also shows over a dozen other observed 1909 and 1921 flood marks along Lyman-Hamilton Road and along the old Great Northern Railroad (GNRR) parallel to Lyman-Hamilton Road (for discussion of these flood marks, see Section 2.2.3). These flood marks correlate well with Stewart-surveyed historical flood marks in the area.

A comparison between HEC-RAS modeled water surface elevations for the USGS published peak discharges and Stewart-surveyed flood marks available in Concrete and the Hamilton-Lyman area is provided in Table 2. The modeled water surface elevations for the USGS-published historical flood peak discharges are substantially higher (by 6.5 to 8.2 feet) than the historical flood marks surveyed by Stewart in 1922 to 1923, and are not supported by post-flood reports published in local newspapers in the Concrete-Hamilton area (available at www.skagitriverhistory.com). This analysis demonstrates a disconnection between the flood marks gathered by Stewart the year following the 1921 flood, Stewart's estimates (and more recently, USGS estimates) of the peak flows of the historical events, and the stage elevations for these discharges computed by the HEC-RAS modeling. PI Engineering believes this analysis clearly indicates the historical flood estimates published by USGS are significantly overestimated.

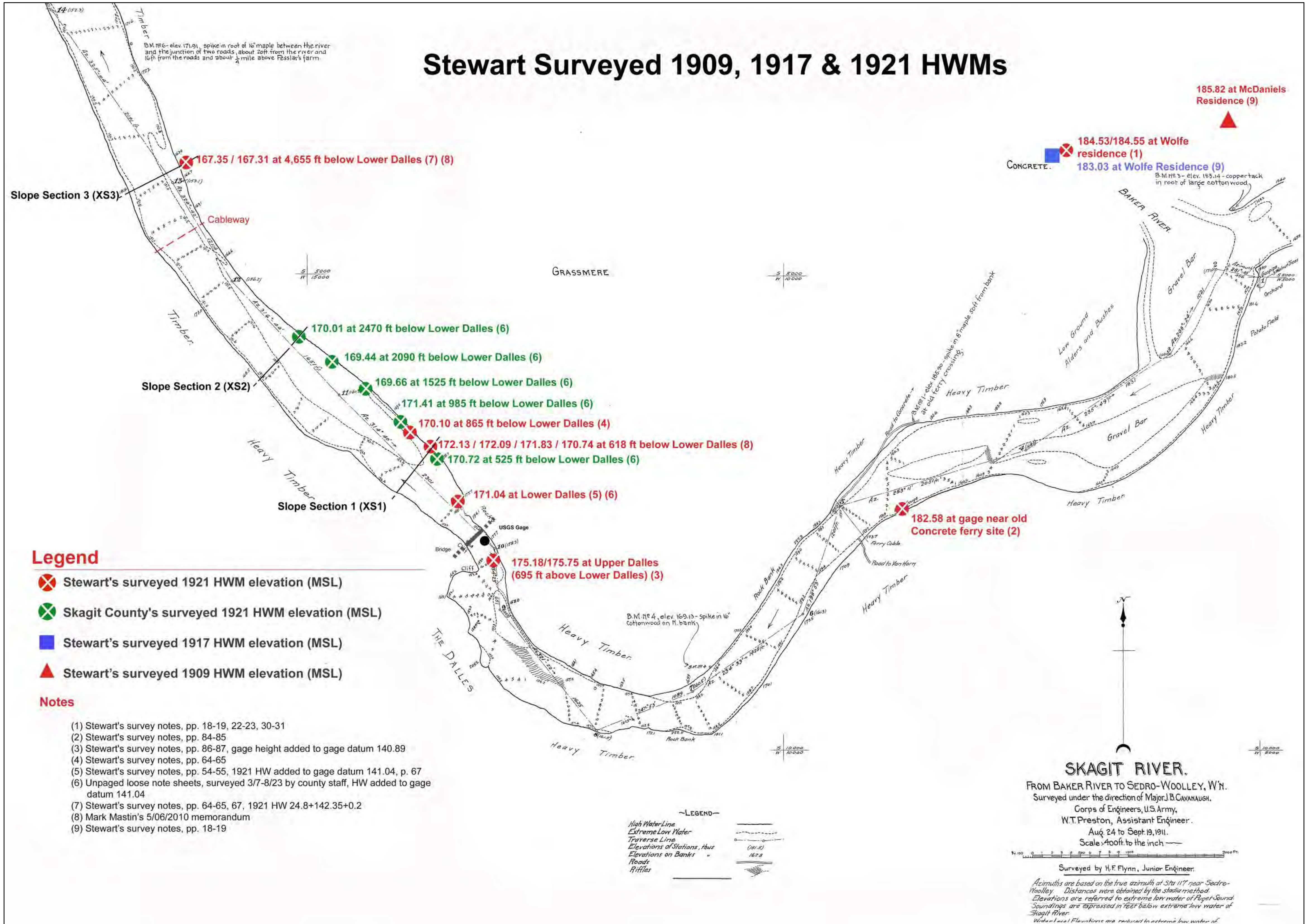

Figure 1. Stewart Surveyed 1909, 1917 and 1921 Highwater Marks (in feet, MSL) in Concrete Area

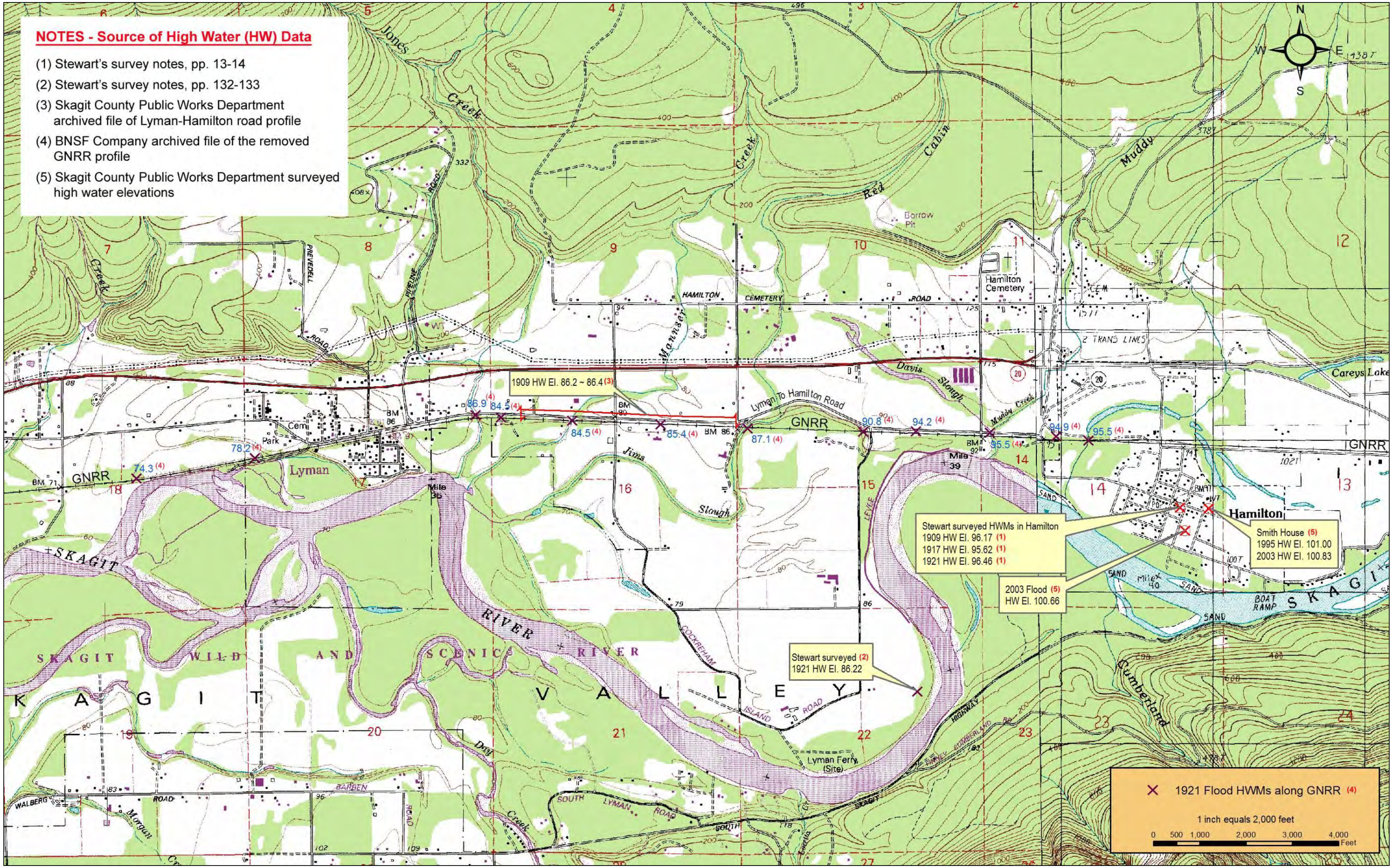

Figure 2. Historical Flood Marks (feet, MSL) in Hamilton-Lyman Area

Table 2. Comparison between HEC-RAS modeled water surface elevations for USGS-published historical flood peaks and Stewart-surveyed historical flood marks in Concrete and the Hamilton-Lyman area.*

	1897 Flood (265,000 cfs)	1909 Flood (245,000 cfs)	1917 Flood (210,000 cfs)	1921 Flood (228,000 cfs)
HEC-RAS modeled W.S. elevations at old Wolfe/McDaniels Residence in Concrete	196.10	194.08	190.77	192.44
Stewart surveyed flood marks at old Wolfe/McDaniels Residence in Concrete (Stewart 1922-23, pp. 18-19, 22-23, 30-31)	NA**	185.82	183.03	184.55 & 184.53
Difference (ft) between HEC-RAS modeled and Stewart surveyed W.S elevations	````	8.16	7.74	7.89 & 7.91
HEC-RAS modeled W.S. elevations at gage near old Concrete ferry site in Concrete	193.88	191.92	188.78	190.36
Stewart surveyed flood mark at gage near old Concrete ferry site in Concrete (Stewart 1922-23, pp. 84-85***)	NA**	NA**	NA**	182.58
Difference (ft) between HEC-RAS modeled and Stewart surveyed W.S elevations	````	````	````	7.78
HEC-RAS modeled W.S. elevations at cigar store building in Hamilton	104.23	103.55	102.25	102.93
Stewart surveyed flood marks at cigar store building in Hamilton (Stewart 1922-23, pp. 13-14)	NA**	96.17	95.62	96.46
Difference (ft) between HEC-RAS modeled and Stewart surveyed W.S elevations	````	7.38	6.63	6.47
HEC-RAS modeled W.S. elevations at old Lyman ferry site in Lyman	94.49	93.98	93.05	93.53
Stewart surveyed flood mark at old Lyman ferry site in Lyman (Stewart 1922-23, pp. 132-133)	NA**	NA**	NA**	86.22
Difference (ft) between HEC-RAS modeled and Stewart surveyed W.S elevations	````	````	````	7.31

Notes: *W.S. elevations and flood marks in feet, NGVD-29 or MSL
**Data not available
***Stewart noted: 1921 HW 32.0 on gage and "0" at gage 150.58 elev

2.1.3 Comparison of Stewart Surveyed Historical Flood Marks to Recent Flood Elevations Surveyed by Skagit County in Hamilton

Stewart surveyed several flood marks in Hamilton during the winter of 1922-23 (see Figure 2). These include the 1909 flood El. 96.17, the 1917 flood El. 95.62 and the 1921 flood El. 96.46 at a cigar store building in Hamilton (Stewart's notes, pp. 13-14). There was no 1897 flood data available in Hamilton from Stewart's survey. Stewart's notes indicate his survey started at a benchmark elevation of 93.9 at ―Top of GN rail in front of Hamilton Depot." This rail top benchmark elevation is identical to the rail top El. 93.9 shown approximately 200 feet east of Pettit Street, the old depot location, on an old Great Northern railroad profile recently obtained by PI Engineering. The old railroad profile was plotted on the Mean Sea Level (MSL), which is approximately the same as the use of NGVD-29 datum. Stewart-used benchmark El. 93.9, ―at top of rail in front of Hamilton station," is listed in the USGS published Bulletin 674 (USGS 1918, p. 78). This confirms that the datum used by Stewart was the same as the use of the NGVD-29 datum.

Flood elevations observed during the recent 1995 and 2003 events were surveyed by Skagit County at the Smith House and vicinity in Hamilton. Table 3 lists the 1909, 1917 and 1921 flood marks surveyed by Stewart, and the 1995 and 2003 flood elevations surveyed by the County for a comparison. Figure 2 shows these surveyed flood mark locations – which are very close to each other (within about a 200-yard distance between County-surveyed and Stewart-surveyed flood marks).

Table 3. Stewart-surveyed historical flood marks and recent flood elevations surveyed by Skagit County in Hamilton (see Figure 2)

Year of Flood	Stewart-surveyed flood marks* at cigar store building	County-surveyed WS elevations* at Smith House & vicinity	USGS published peak discharge (cfs) at USGS gaging station near Concrete (RM 54.1)
1909	96.17**	--	245,000***
1917	95.62**	--	210,000***
1921	96.46**	--	228,000***
1995	--	101.00	160,000****
2003	--	100.83 & 100.66	166,000****

Notes: *W.S. elevations and flood marks in feet, NGVD-29 or MSL
**Stewart 1922-23, pp.13-14
***USGS estimated peak discharge (USGS, 2007)
****USGS recorded peak discharge

Stewart-surveyed 1909, 1917 and 1921 flood marks in Hamilton are 96.17, 95.62, and 96.46, respectively. The County-surveyed flood marks are 101.00 for the 1995 flood, and 100.83 and 100.66 for the 2003 flood. Stewart-surveyed historical flood marks are more than 4 feet lower than the County-surveyed 1995 and 2003 flood elevations. The comparison of Stewart and County surveyed flood marks in Hamilton indicates that the 1909, 1917 and 1921 flood peak discharges should be less than the 1995 and 2003 flood peak discharges. The 1909, 1917 and 1921 historical flood peak discharges currently estimated by USGS at the USGS gaging station Skagit River near Concrete (#12-194000, RM 54.1), are 245,000, 210,000, and 228,000 cfs, respectively. While the 1995 and 2003 peak discharges recorded at the gaging station near Concrete, are 160,000 and 166,000 cfs, respectively. This clearly indicates the 1909, 1917 and 1921 flood historical flood peak discharges published by USGS are significantly overestimated.

2.1.4 Incorrect Application of Slope-Area HWMs in Stewart and USGS Computations of 1921 Discharge

Background

In 1923, following a field investigation conducted in late 1922 and early 1923, Stewart estimated a peak discharge of 240,000 cfs for the historical flood that occurred on December 13, 1921 (USGS, 1961). To arrive at this estimate, Stewart applied the slope-area method and averaged the results from three reaches (XS1–XS2, XS2–XS3, and XS1–XS3) using surveyed high water marks (HWMs) and three cross sections (XS1, XS2, and XS3) of the Skagit River below the Dalles near Concrete, Washington (Stewart, 1923). Figure 3 (likewise Figure 1) shows the location of the cross sections, the Dalles (or Dalles Gorge), and the USGS gaging station #12-194000, Skagit River near Concrete, WA.

Stewart then used his 1921 peak flow estimate to extend a stage-discharge rating for estimating three other large historical flood peaks (275,000, 260,000 and 220,000 cfs) at the Dalles occurring in 1897, 1909, and 1917 (USGS, 1961). The accuracy of Stewart's peak discharge estimate of the 1921 flood has been widely questioned, thus bringing into question the accuracies of the peak discharge estimates of the other three historical floods.

In 2007, the USGS, in its published SIR, reevaluated the 1921 peak discharge, applying a lower Manning's "n" value and an improved computation approach that takes into consideration velocity variations between cross sections that apply to Stewart's data, but only at the lower slope-area reach (XS2–XS3). Based on these differences, the USGS slightly revised downward the Stewart-estimated historical flood discharges (USGS 2007). These revised historic flood peaks were used, in conjunction with the systematic annual peak discharge record observed since 1924 at the USGS gaging station Skagit River near Concrete (12-194000), to estimate the 10-, 50-, 100-, and 500-year (or 10, 2, 1, and 0.2 percent annual chance) synthetic peak flows in the RFIS hydrologic analysis (USACE, 2008).

Incorrect HWM Application in Stewart and USGS Computations

Stewart collected HWMs along right and left bank lines in the slope-area study reach below the Dalles. Both Stewart in 1923 and the USGS in 2007 assumed that the HWMs represented the mean water surface level at the time of the flood crest across the river channel section. PI Engineering found that this assumption is incorrect, as discussed below.

Stewart's HWMs are located on the river bank where the flow velocities are minimal as demonstrated by the USGS flow velocity measurements. A cableway section is located within the lower slope-area reach (XS2–XS3), at approximately 630 ft upstream of the lower slope section (XS3) as shown on Figure 3. The cableway section has been used by USGS since 1924 to measure flow velocities and to estimate discharges for the Dalles gage rating.

PI Engineering recently obtained the data for the two largest discharge measurements of record from USGS: Measurement 475 dated 10/21/2003 for 138,000 cfs, and Measurement 40 dated 3/11/1932 for 135,000 cfs. Both measurements show a similar velocity distribution across the cableway section. The depth-average velocities are lowest (3.5 to 4.3 fps) near the left and right banks, and highest (13.7 to 13.9 fps) approximately 100 to 140 ft from the left bank where the water depth is highest. The section-mean velocity was computed to be 11.09 fps for Measurement 475 and 11.68 fps for Measurement 40 (5 percent difference between the two measurements). These measurement data indicate that the velocity head varies from 0.2-0.3 ft near the banks to 2.9-3.0 ft in the deep channel, and has a section-mean velocity head of 1.9 ft for Measurement 475 and 2.1 ft for Measurement 40.

The water is moving slower near the banks, where the water surface (WS) elevations are closer to the energy grade line (EGL) elevation, which equals the sum of the WS elevation and the velocity head. Figure 4 is a plot of Measurement 40 showing variation of the measured water depth and computed WS elevation based on the EGL elevation, subtracting the velocity head computed from the measured velocity across the cableway section. The plot also shows that the computed mean WS elevation across the cableway section is 2.1 ft (the computed velocity head) lower than the EGL elevation for Measurement 40.

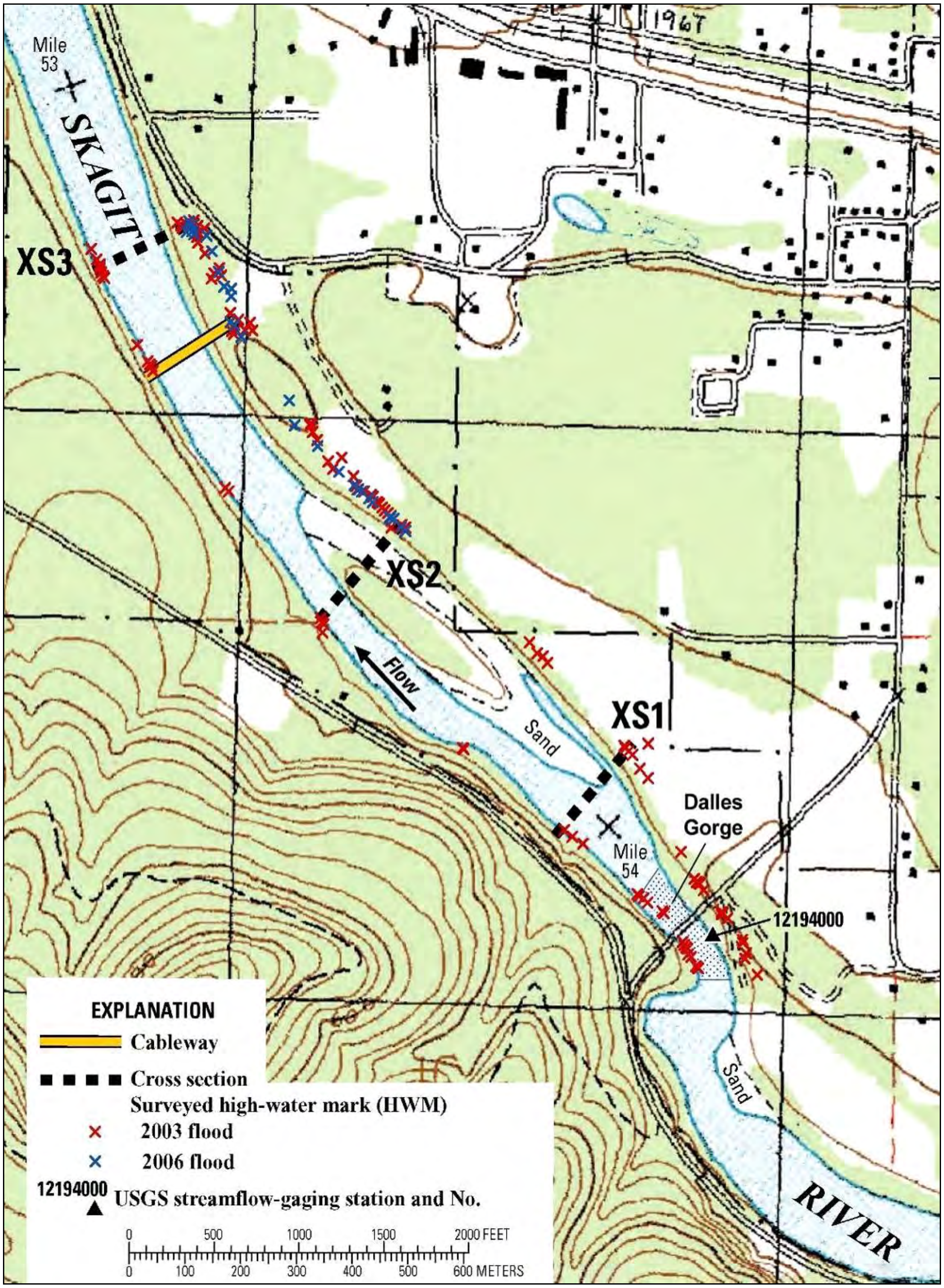

Figure 3. **Topographic map of the slope-area measurement reach on the Skagit River near Concrete showing the three cross sections (XS1, XS2, and XS3), the streamflow gaging station, and HWMs from the 2003 flood and the 2006 flood surveyed by the U.S. Geological Survey (source of data: Scientific Investigation Report 2007-5159, USGS)**

Velocity distribution at the cableway section is expected to be representative of the lower two slope-area sections. Stewart's HWMs collected on the banks represent the EGL elevations, not the mean WS elevations which were assumed by Stewart and USGS when each applied these HWMs in computing the 1921 flood peak discharge. The difference between the EGL and the mean WS elevations is the mean velocity head and is significant, approximately 2 ft at the cableway section as demonstrated above by the Measurements 475 and 40 data. The mean velocity and the velocity head would increase slightly but not significantly with increase of the flood discharge. Stewart estimated and the USGS revised 1921 flood peak discharges would imply a much higher velocity and velocity head (around 16.3-17.1 fps and 4.1-4.6 ft, respectively, see Sections 2.2.5 and 2.2.6) at Stewart-surveyed XS3.

Applying Stewart's 1921 HWMs at the slope sections as the mean WS elevations of the flood led to an incorrect estimate of the 1921 flood peak discharge made by Stewart in 1923 and revised by the USGS in 2007, thus leading to the incorrect peak discharge estimates of the other three historical floods and the subsequent synthetic peak flows used in the RFIS.

The next section provides a review of the historical flood data. Following that, details of PIE Engineering's reevaluation of the historical floods are provided. Our reevaluation includes the corrected application of Stewart's HWMs at the slope-area sections and the use of several analytical methodologies to evaluate the peak flow estimates of the historical floods.

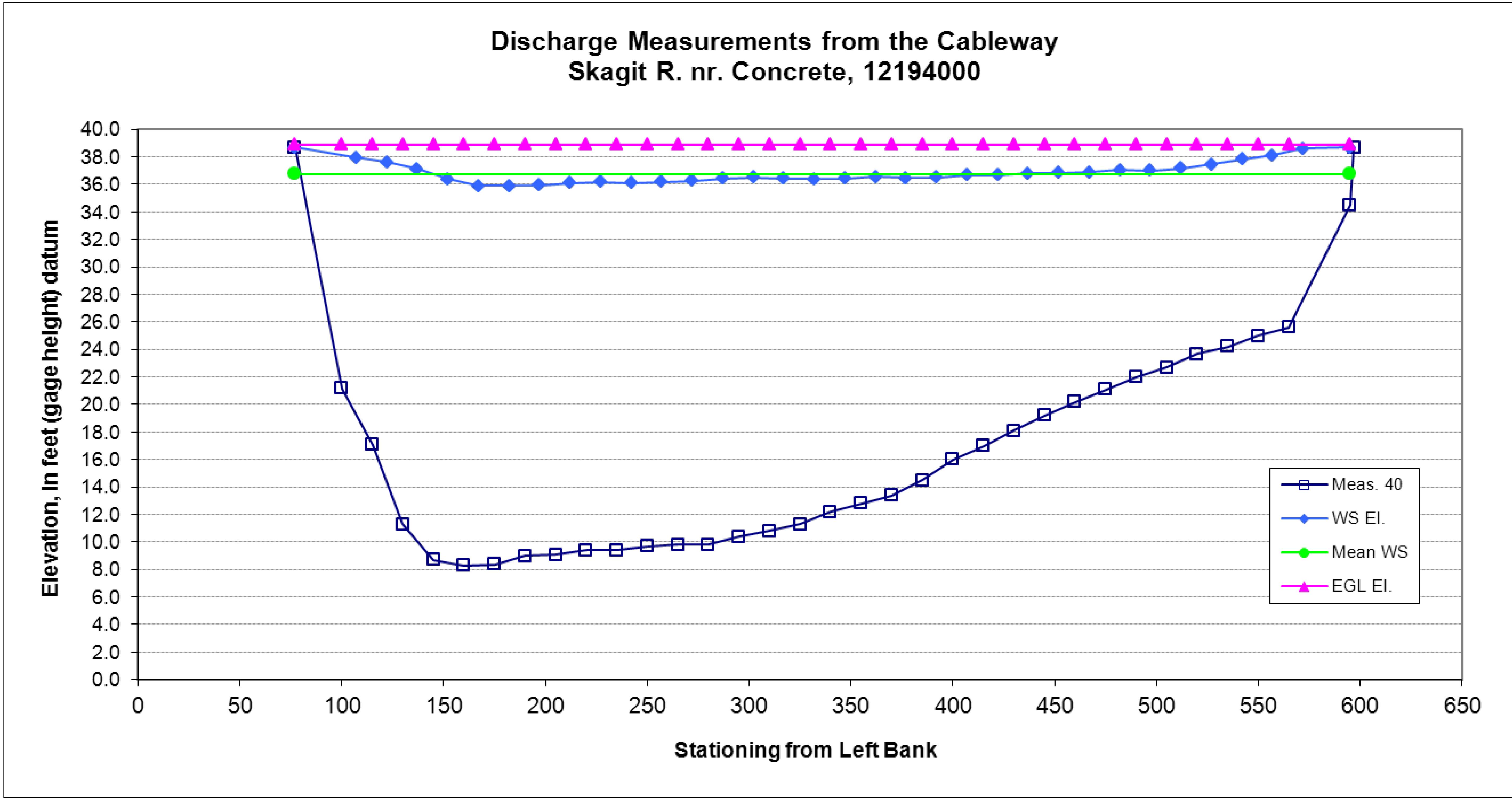

Figure 4. **Discharge Measurement 40 from Cableway at Skagit River near Concrete**
(Note: original source of data and water depth plots provided by USGS, in Microsoft Excel - Concrete_cableway_meas#475.xls)

2.2 Review of 1897, 1909, 1917, and 1921 Historical Floods

2.2.1 Historical Flood Data in Concrete

Four major historical floods occurred before installation of the USGS gage at Concrete. These historical floods were estimated by James Stewart in 1923 (Stewart 1923). The accuracy of Stewart's flood peak estimates was questioned by numerous hydrologists, including hydrologists within the USGS (Bodhaine 1954; Riggs & Robinson 1950). Despite the questions raised regarding Stewart's 1923 estimates, USGS published them in 1961 as Water Supply Paper (WSP) 1527 (USGS, 1961). Table 4 shows the USGS published gage heights and estimated peak discharges for the four historical floods.

Table 4. USGS estimated peak stages and discharges of Skagit River near Concrete for four historical floods (Drainage Area = 2,700 sq. mi.)

Flood	Gage Height at Current Gage* as Published in 1961 (ft)	Gage Height** Estimated by Stewart in 1923*** (ft)	Discharge Estimated by Stewart in 1923*** (cfs)	Discharge Revised by USGS in 2007**** (cfs)
1897[x]	51.1	38.4	275,000	265,000
1909[x]	49.1	36.4	260,000	245,000
1917[x]	45.7	33.0	220,000	210,000
1921[y]	47.6	34.9	240,000	228,000

* Current gage datum El. 130.00 (NGVD29) at RM 54.15.

** At the Upper Dalles gage installed by Stewart for his flood investigation during the winter of 1922-23. Gage Datum El. 140.89 surveyed by Stewart (Stewart's survey notes, pp. 86-87).

*** These unpublished 1923 estimates by James Stewart were documented in the 1961 U.S. Geological Survey Water Supply Paper (WSP) 1527 (USGS 1961).

**** Revised in Scientific Investigations Report 2007-5159 (USGS 2007)

x These HWMs were measured on the bank of the Baker River, 2 miles upstream. Stewart and USGS translated them to this gage location by assuming the differences between the flood and the 1921 flood would be the same at the gage as they were on the Baker River banks.

Y This HWM was translated to the current gage from the Upper Dalles gage established by Stewart. No allowance for slope of the hydraulic grade line was made.

Gage Datum Discrepancy

It is noted that the 1921 flood stages using two Stewart-surveyed gage heights of 34.29 and 34.86 at his upper Dalles gage (gage datum El. 140.89, see Stewart's survey notes, p. 87), are El. 175.18 and El. 175.75 (NGVD-29). Stewart's upper Dalles gage was located about 330 feet upstream from the location of the current USGS gage. The USGS-published 1921 flood gage height of 47.6 and gage datum El. 130.00 at the current gage is El. 177.6 (NGVD-29), approximately 2.4 to 1.8 feet higher than Stewart-surveyed elevations further upstream, which is not reasonable. Based on our review of available data, we could not find any explanation for this discrepancy and there is no scientific evidence to support the USGS-published 1921 flood stage. More discussion of the Dalles gage datum is provided in Section 2.2.2 below.

Incorrect Transfer of Stewart's 1921 HWMs from Upper Dalles to Current Gage

Stewart's 1921 HWMs shown in Figure 1 are 175.18 and 175.75 at the upper Dalles and 171.04 at the lower Dalles, indicating a drop of 4.14 and 4.71 feet (averaging 4.5 feet) at the crest of the 1921 flood between the upper and the lower Dalles. Stewart noted that the upper Dalles is 695 ft above the lower end of the Dalles (Stewart's survey notes, p. 62).

The current USGS gage is located on the right bank about 365 feet upstream of the mouth of the Dalles based on USGS provided distance data (see Figure 10 and Section 2.3.2 discussion for this distance). This leads to an estimated distance of 330 ft between Stewart's upper Dalles gage and the current gage (which is more than "*about 200 feet above present gage*" stated incorrectly in the USGS-published Water Supply Paper 612 (USGS 1925, p. 62)). The current gage is located approximately in the middle of Stewart's upper and lower Dalles gages.

The USGS published 1921 flood stage at the current gage is based on a direct transfer of Stewart-surveyed HWM data at the upper Dalles without consideration of the flood stage drop in this 330-foot distance between the upper Dalles gage and the current gage location. This is an error that contributes to the overestimation of flow for the USGS published 1921 flood.

The USGS published 1921 flood stage transferred only the upper Dalles HWM data, ignoring the importance of the lower Dalles HWM surveyed by Stewart. This HWM data transfer is not correct, leading to a biased 1921 flood stage as published and shown in Table 4. A correct HWM transfer using Stewart's upper and lower Dalles HWMs is discussed in Section 2.3.2.

Published 1909 and 1917 Gage Heights Un-supported by Stewart's HWMs at Wolfe and McDaniels Residences in Concrete

Stewart did not survey the 1897, 1909, and 1917 flood HWMs in the Dalles. These three historical flood gage heights published (as shown in Table 4) were based on the relation of the Stewart-surveyed high water marks (HWMs) located on the banks of the Baker River, about 2 miles upstream.

Stewart surveyed 1921 and 1917 HWMs at the old Wolfe residence on the right bank of the Baker River, and 1909 HWM at the old McDaniels residence on the left bank across the Baker River from the Wolfe residence (Stewart's notes, pp. 18-19). Based on a search of archived 1921 property tax rolls, the City of Burlington found the location of the old Wolfe and McDaniels residences (see Figure 5).

Stewart-surveyed 1909, 1917 and 1921 HWMs at the Wolfe and McDaniels residences have the differential gage heights of 1.27 ft between the 1909 and 1921 HWMs, and 1.52 ft between the 1921 and 1917 HWMs (Stewart's notes, pp. 18-19). The differential gage heights calculated from the published gage heights listed in Table 4 and used in Stewart and USGS peak discharge estimates are 1.50 ft between the 1909 and 1921 HWMs, and 1.90 ft between the 1921 and 1917 HWMs. These differential gage heights based on the published data are all higher than those based on Stewart-surveyed HWMs. These published 1909 and 1917 gage heights are not supported by Stewart's surveyed HWMs at the Wolfe and McDaniels residences, and are not appropriate for use to estimate the 1909 and 1917 historical flood discharges.

Un-Reliable 1897 Flood Marks

There is 2-ft differential gage height at the Skagit River near Concrete between the 1897 and 1909 floods listed in Table 4. This flood differential height was transferred by Stewart from Stewart's flood marks on a hotel located over 2 miles upstream on east bank of the Baker River.

Stewart had 1909 flood marks on "the footing of a hotel near the cement plant", and "on the outside galvanized siding of an old Washington Cement Plant shop building" (see USGS 1961, p. 29, and Stewart's notes, p.0). Exact locations of the old hotel and the machine shop building referenced in Stewart's survey notes could not be found.

WSP 1527 (USGS, 1961) and Stewart's 1923 report (Stewart, 1923) referenced two 1897 HWMs transferred to a hotel footing. The first 1897 HWM was found "on a barn on the right bank about a mile upstream from Concrete, was transferred by levels to the footing of a hotel in Concrete on which the other flood mark had been made in 1909." Later, a second HWM was found on a stump that was reported by Magnus Miller to be "1.5 feet out of the water during the flood of 1897." For the 1909 flood, WSP 1527 states that Stewart measured a flood mark on "a hotel near the cement plant [that] was just reached by the water." Exact locations are not given for either the barn or the stump.

Figure 5 is a 1937 aerial photo on which the old Wolfe residence, McDaniels residence, Washington Cement Plant (note an old railroad bridge crossing the Baker River downstream of the plant), upper Dalles, lower Dalles and current USGS gage site at the Dalles are annotated. Figure 6 shows the panoramic view of Concrete in the 1911 and circa 1920. The cement plant is on the left of the view and on the east bank of the Baker River upstream of the old railroad bridge (no longer existing). Likely the machine shop building and the old hotel were also located upstream of the old railroad bridge on the east bank of the Baker River.

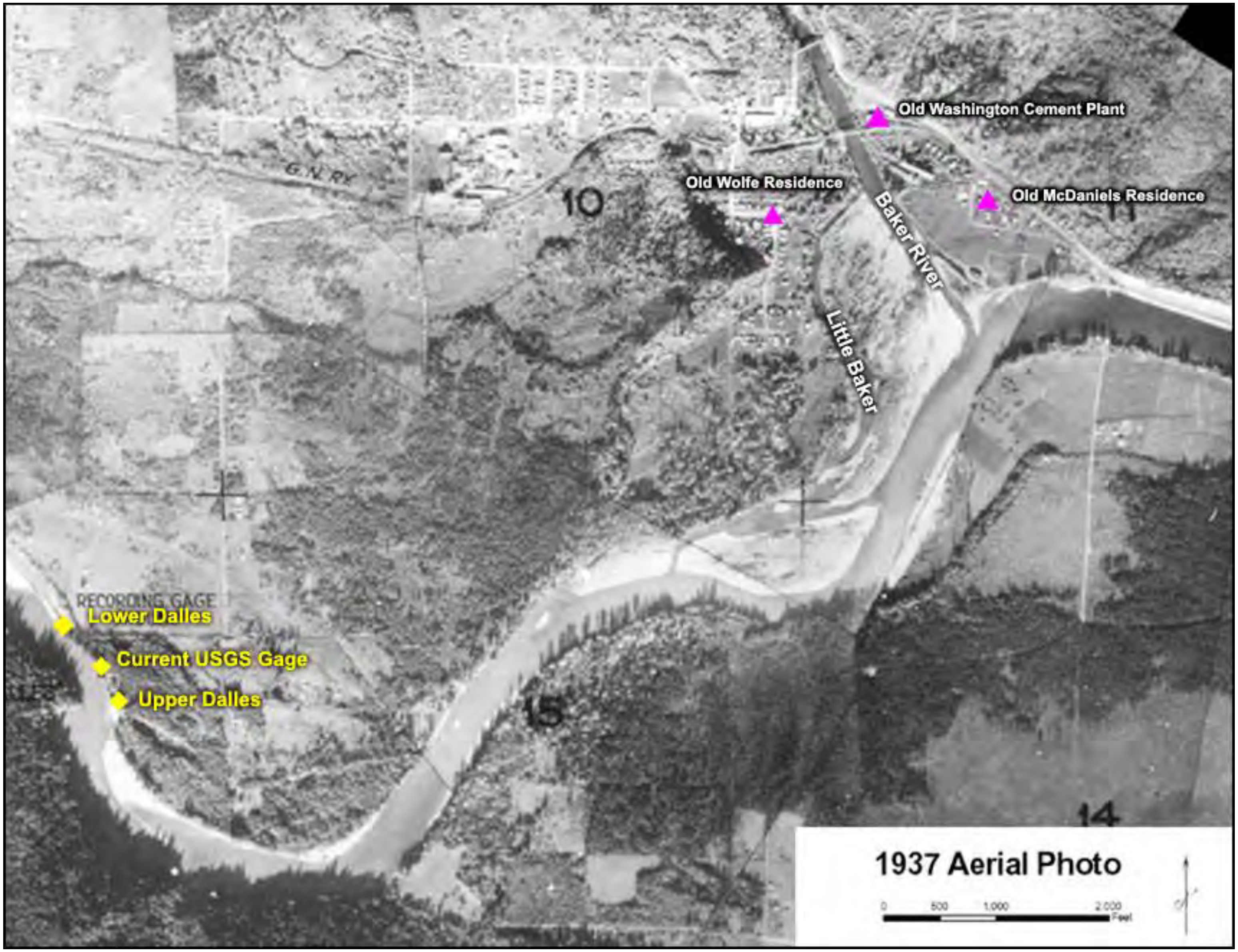

Figure 5. 1937 Aerial photo showing location of the upper/lower Dalles, old cement plant and Wolfe/McDaniels residences

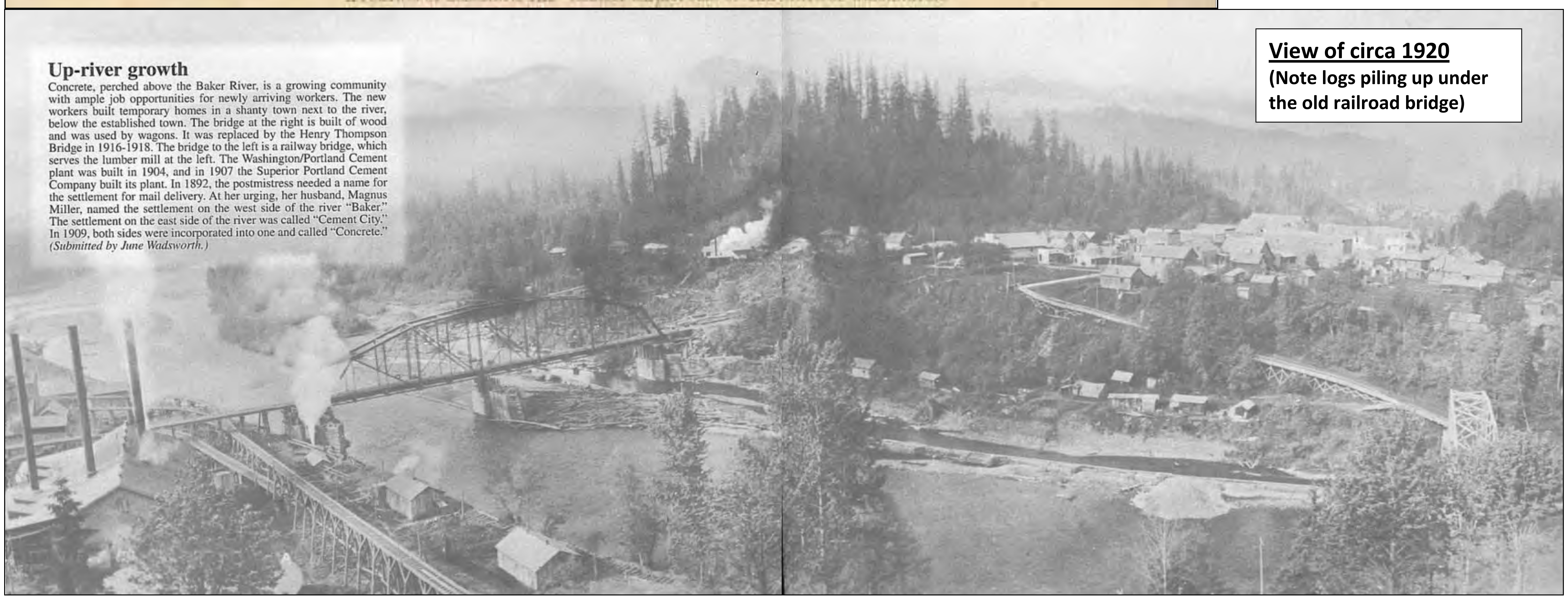

Figure 6. Panoramic views of Concrete in 1911 and circa 1920 (looking the Baker River downstream towards the Skagit River confluence)

The old cement plant and the Wolfe and McDaniels residences were all located in the Skagit River backwater area. However, due to constriction and debris jamming at the railroad bridge during floods, the flood stages at the cement plant site (likely including the hotel footing and the machine shop building) upstream of the old railroad bridge could have been higher due to the Baker River flow than due to the Skagit River backwater in the 1897 to 1921 time period.

These 1897 and 1909 HWMs may have been affected by Baker River flows at the hotel footing or on the machine shop building (as Stewart noted in his survey notes, p. 0, and also noted in USGS 1961, p. 28). In discussion of the transferred 1897 flood marks, WSP 1527 states that "the flood elevations in Concrete probably were affected to a considerable extent by the flow of Baker River. The relationship between the two floods [1897 and 1909] at that point may have been quite different from the relationship at the [Concrete] gaging station site" (USGS 1961, p. 28).

It is our opinion that the 1897 gage height shown in Table 4 is not reliable HWM data to use, given the uncertainties that include transferring the flood marks from two questionable locations to an unknown hotel footing location, and Stewart's interview with Leonard Everett (Stewart's Notes, p. 23) stating that the "log jam in the Dalles raised water 10 ft in 2 hrs" during the 1897 flood.

Un-Reliable 1897, 1909, 1917, and 1921 Flood Peak Discharge Estimates

As discussed in Section 2.1.4, the incorrect application of the 1921 HWMs in the slope-area computations by Stewart in 1923 led to an un-reliable 1921 flood peak discharge estimate (see Section 2.2.5 for Stewart's slope-area computations).

Stewart's transferred 1909 and 1917 HWMs are not supported by his surveyed HWMs at the Wolfe and McDaniels residences. His transferred 1897 HWM was based on un-reliable flood marks. Using these transferred HWMs in combination with the use of his gage rating based on the incorrect 1921 flood peak discharge estimate, Stewart's estimates of the 1897, 1909 and 1917 flood peak discharges are not reliable.

The approach used by the USGS (Mastin) 2007 revised estimate of the 1921 flood peak discharge was also based on an incorrectly application of the 1921 HWMs in the slope-area computations, leading to an unreliable 1921 flood peak discharge estimate (see Section 2.2.6 for the USGS revised slope-area computations). The USGS 2007 revised estimates of the 1897, 1909 and 1917 peak discharges are based on Stewart's transferred HWMs. Mastin utilized the Stewart translation of the HWMs done by Stewart without comment. The flow per foot of gage height is considerably different (See Figure 11, Stage-discharge Rating curve for Skagit River near Concrete gage and Figure 14, Skagit River backwater stage-discharge rating curve at Baker gage for a comparison). This means the Stewart's translation of HWMs (differentials with 1921) should have been adjusted to account for the differing flow rates per foot of stage increase. The USGS revised estimates of the 1897, 1909 and 1917 flood peak discharges are also not reliable.

2.2.2 Gage Datum at The Dalles

Datum of Stewart's Survey

Stewart's survey of flood marks, low-flow water surface, and gage data in Concrete that include the 1917 and 1921 flood HWMs at the old Wolfe residence, and the 1921 flood HWMs near the old Concrete ferry site and at the upper and lower Dalles gages, starts at a USGS benchmark elevation of 230.51 in Concrete (Stewart's notes, p. 22 and p. 30). This USGS benchmark is listed in the USGS published Bulletin 674 (USGS 1918, pp. 78-79) as elevation of 230.506. All elevations at the USGS benchmarks used in Stewart's surveys of the flood marks in Concrete and in Hamilton and on the old GNRR profile are based on mean sea level datum of the early 1900s, which was not significantly different from the NGVD-29 datum.

Stewart twice surveyed the 1921 flood HWM at the Wolfe residence (Stewart's notes, pp. 22-23 and pp. 30-31). Both surveys started at the USGS benchmark and took six turning points to the HWM. The difference between these two surveys of the 1921 HWM (184.53 and 184.55) is 0.02 ft, confirming his survey accuracy of the 1921 HWM at the Wolfe residence.

Incorrect Gage Datum Used by USGS in Transferring Stewart's HWMs

The USGS was not aware of Stewart's gage datum of El. 140.89 at the upper Dalles (Stewart's notes, pp. 86-87). Recently, after learning the existence of this information, USGS began asserting that *"the gage datum of Stewart's historical HWM elevations was likely to be 142.7 ft NGVD-29 and not 140.9 ft,"* (Mastin's November 5, 2008 letter, USGS 2008). This statement indicates there is a 1.8-ft gage datum discrepancy (142.7 – 140.9 = 1.8 ft).

However, in Stewart's survey notes, Stewart clearly noted his survey benchmark and elevations of HWMs, low-flow water surface, and gage datum based on the use of the Mean Sea Level (MSL) which is approximately the same as the use of NGVD-29 datum, and estimated by National Geodetic Survey (NGS) to be within 0.12 (+/-) feet for the average of height shifts (ranging zero to +0.4 ft) from a sample of 1909/1912 benchmarks to NAVD 88 elevations (see email from Malcolm Leytham, 10/16/2008, including NGS spreadsheet: Height Differences in Skagit Co, WA.xls (Leytham 2008)). Stewart set up an upper Dalles gage during his 1922–23 field survey of the 1921 HWMs. Stewart's survey for elevations of the gage datum and HWMs (as well as low-flow water surface) starts at a USGS benchmark in Concrete. Stewart's surveyed upper Dalles gage datum is 140.89 as noted in his survey notes (Stewart's survey notes, pp. 86-87), not 142.7, which is a rounded elevation of an old

Skagit County gage datum of 142.69 (=130+12.69, see USGS 1961, p. 50, "Gage" paragraph for gage datum)[1].

During the 1924–37 period, Skagit County operated a gage at the Dalles with the gage datum of 142.69, or 1.8 ft higher than Stewart's upper Dalles gage datum of 140.89. (See footnote 1 and USGS 1961, P 50, "Gage" paragraph). The County's gage was located at the present gage site as stated in the WSP 1527 (USGS 1961, p. 50).

The USGS-published Water Supply Paper 612 (USGS 1925, p. 62) describes that: "*Gage – Since December 10, 1924, Stevens continuous recorder in concrete shelter, on right bank at the Dalles. Gage used prior to December 10, 1924, was vertical and inclined staff on right bank about 200 feet above present gage. Both gage readings refer to same datum, 163 feet above sea level*." The referenced vertical and inclined staff gage was Stewart's upper Dalles gage (Stewart's survey notes, pp. 86-87). The stated distance for location of Stewart's upper Dalles staff gage "*about 200 feet above present gage*" is incorrect as pointed out in Section 2.2.1. The correct distance based on Stewart's survey is about 330 ft between Stewart's upper Dalles gage and the current gage. The referenced gage datum 163 feet above sea level is incorrect for both Stewart's and the County's gages (USGS 2008, p. 2).

We believe the above statement that "*Both gage readings refer to same datum*" is incorrect. (Note: notwithstanding the incorrect reference to a gage datum of 163 feet, the USGS has not been able to provide direct evidence relating to the gage-datum conversion to support the statement that "Both gage readings refer to same datum." The USGS does, however, point to indirect evidence to support a hypothesis that Stewart's Upper Dalles gage datum was wrong, and subsequently corrected when the new gage datum was established (See Mastin's November 5, 2008 letter, USGS 2008 and attachments)).

In 1937, the current gage was established by USGS (see USGS 1936 letter for an agreement of cooperation between USGS and Skagit County, and USGS 1961, p. 25, for gage history). The USGS has since published all pre-1937 HWM elevation data based on the County's gage datum of 142.69, as the USGS was not aware of Stewart's original gage datum of 140.89 until 2008 (see Mastin's November 5, 2008 letter, 2nd paragraph,

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[1] For the County's gage installation, see the October 21, 1936 letter from G.L. Parker, District Engineer, USGS, to Hugo Baumen, Chairman, Skagit County Commissioners (USGS 1936). The letter states that "*You will recall that Mr. Knapp explained to you that records at the Skagit River gaging station near Concrete were essential in preparing any sort of plan for flood prevention and control.* **He built the gaging station from Skagit County funds in the fall of 1924** *as a consequence of studies made of flood damage and plans for protection undertaken after the disastrous flood of 1921*. --- *For a number of years the gaging station was financed entirely from Skagit County funds because the Federal appropriation did not provide for corporation*." Mr. Knapp was the County Engineer, in accordance with the September 6th 1923 letter from Mr. Knapp, County Engineer, to Mr. D.J.F. Calkins, Acting District Engineer, USGS.

USGS 2008). The USGS-published HWM elevations based on the county gage datum, including Stewart's historical HWM elevations, are therefore 1.8 ft higher than Stewart's surveyed historical HWM elevations based on Stewart's gage datum.

Low-flow Water Surface Elevations Surveyed by Stewart and Others

In an effort to provide additional information for use in objectively analyzing this 1.8-ft gage datum discrepancy, PI Engineering reviewed all low-flow water surface elevations in Concrete and the Dalles, which are available from Stewart's survey notes and are also available from other sources for the same locations and flow conditions.

It is not expected that Stewart's surveyed low-flow water surface elevation would be exactly the same as others' survey for the same location and flow conditions. Factors that could affect low-flow water surface elevations surveyed by different parties include change in channel bottom geometry due to sediment degradation/aggradation, temporary debris deposition, slight flow variation, and survey accuracy. These factors may significantly affect low-flow water surface elevations. However, we would still expect that the majority of Stewart's surveyed elevations would be close to 1.8 feet lower than others' surveyed elevations if the USGS-asserted gage datum is accurate.

Table 5 lists the low-flow water surface elevations surveyed by Stewart in comparison with those surveyed by USACE (or COE, in 1911), Skagit County (in 2008), and PI Engineering (in 2004), for approximately the same survey locations and similar low-flow conditions. Two sets of Stewart's surveyed elevations, one based on his original survey gage datum of 140.89 and the other based on the USGS-asserted 1.8-ft higher datum (142.69), are listed in the table for comparison. All elevations shown in the table are based on the same elevation datum (either MSL or NGVD-29). Figure 7 is a USACE 1911 river survey map of the area on which the survey points, elevations, and notes by various parties shown in Table 5 are annotated.

If the gage datum of Stewart's surveyed elevations were to be 142.7 (rounded from 142.69) or 1.8-ft higher than Stewart's noted 140.89 datum, all of Stewart's elevations (including not only HWM elevations but also low-flow water surface elevations based on Stewart's gage datum 140.89) would have had a difference of approximately 1.8 ft from other parties' surveyed elevations. More specifically, if Stewart's datum were wrong, the low-flow water elevations from his field notes would all be approximately 1.8-ft lower than other data.

Table 5. Comparison of low-flow water surface elevations surveyed by Stewart and others using NGVD-29 datum

Location	Stewart 1922–23 Survey		COE 1911 Survey*	Recent Survey	Difference Between Stewart and Other Surveys (ft)	
	Based on 140.89 Datum	Based on 142.69 Datum (+1.8 ft)			Based on 140.89 Datum	Based on 142.69 Datum
Near old Concrete Ferry Site	**151.92** (01/27/23 – Stewart notes, p. 84, flow 9,740 cfs at Sedro-Woolley)	**153.72** (01/27/23 – Stewart notes, p. 84, flow 9,740 cfs at Sedro-Woolley)	**151.1** (8,570–9,980 cfs at Sedro-Woolley)	**152.1** (Skagit County 04/28/08 – 9,420 cfs at Mt. Vernon and 7,680 cfs at Concrete, surveyed 152.32/150.84 at LB Pt. # 1365/1366)	**0.82 and –0.18**	**2.62 and 1.62**
Upper Dalles Gage	**144.58** (01/27/23 – Stewart's Notes, p. 86, flow 9,740 cfs at Sedro-Woolley)	**146.38** (01/27/23 – Stewart's Notes, p. 86, flow 9,740 cfs at Sedro-Woolley)	**144.5** (8,570–9,980 cfs at Sedro-Woolley)		**0.08**	**1.88**
	147.55 (12/23/22 – Stewart's Notes, p. 34, 6.66+140.89, flow 14,200 cfs at Sedro-Woolley)	**149.35** (12/23/22 – Stewart's Notes, p. 34, 6.66+140.89, flow 14,200 cfs at Sedro-Woolley)		**147.4** (PIE 9/30/04 – flow 13,300 cfs at Mt. Vernon and 12,500 cfs at Concrete)	**0.15**	**1.95**
Lower Dalles Gage	**144.95** (01/25/23 – Stewart's notes, p. 54, 3.91+141.04, flow 10,100 cfs at Sedro-Woolley)	**146.75** (01/25/23 – Stewart's notes, p. 54, 3.91+141.04, flow 10,100 cfs at Sedro-Woolley)	**144.3** (8,570–9,980 cfs at Sedro-Woolley)		**0.65**	**2.45**
Upper Slope Section	**144.12** (01/30/23 – Stewart's notes, p. 64, flow 7,660 cfs at Sedro-Woolley)	**145.92** (01/30/23 – Stewart's notes, p. 64, flow 7,660 cfs at Sedro-Woolley)	**143.7** (8,570–9,980 cfs at Sedro-Woolley)		**0.42**	**2.22**
Lower Slope Section	**142.35** (01/30/23 – Stewart's notes, p. 64, flow 7,660 cfs at Sedro-Woolley)	**144.15** (01/30/23 – Stewart's notes, p. 64, flow 7,660 cfs at Sedro-Woolley)	**142.1** (8,570–9,980 cfs at Sedro-Woolley)		**0.25**	**2.05**
				Range of Difference =	**–0.18 to 0.82**	**1.62 to 2.62**

* Elevations based on extreme low water of Puget Sound were adjusted by –8.93 ft to NGVD-29 (see USGS 1961, p. 52, "Gage" description). The Skagit River survey was conducted between August 24 and September 19, 1911 by COE from Baker River to Sedro-Woolley (see the title and notes of the original COE surveyed map on lower right corner of Figure 4). We assume the survey in Concrete area was conducted in August 1911 for conservatism, as the Sedro-Woolley gage data indicate that the Skagit River flows in August 1911 were lower than those in September 1911.

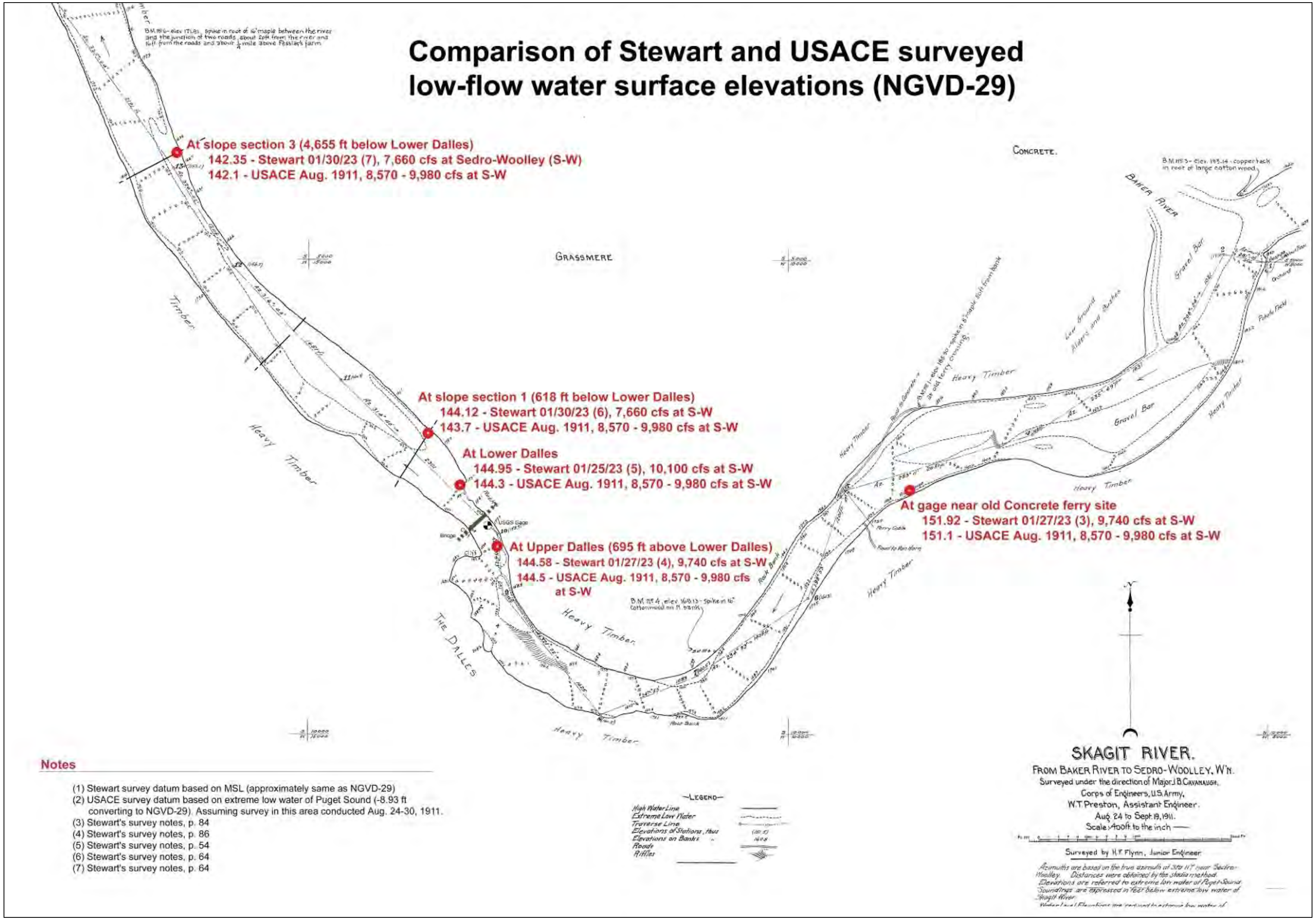

Figure 7. Comparison of Stewart and COE surveyed low-flow water surface elevations (NGVD-29)

As shown in Table 5, Stewart's surveyed low-flow water elevations (based on his gage datum of 140.89) are between 0.18 feet lower and 0.82 feet higher than those surveyed by other parties.[2] Stewart's data are mostly higher, not lower. None of Stewart's surveyed elevations are near the 1.8-ft difference they should have been, if the USGS-asserted datum were correct. And as shown in Table 5, if using the USGS-asserted 1.8-ft higher gage datum of 142.69, Stewart's surveyed low-flow water surface elevations are between 1.62 and 2.62 feet higher than those surveyed by other parties, which is not reasonable.

Converting Stewart's 1921 HWMs to Others' Survey Datum

Assuming Stewart's gage datum was wrong as the USGS asserted, an alternative approach to estimating the 1921 flood peak discharge is not to use directly Stewart-surveyed HWM elevations that are based on his survey datum, but to use his HWM data after converting to others' survey datum (based on NGVD-29). The procedure for converting Stewart's HWMs to others' survey datum is described below.

At several locations as documented in his 1922-23 survey notes, Stewart surveyed the elevations (or gage heights) based on his gage datum for both 1921 HWM and low-flow water surface on his noted field date. At each of these survey locations, a relative gage height of Stewart's surveyed 1921 HWM can be calculated in relation to his surveyed low-flow water surface level. The calculated relative gage height equals the surveyed HWM elevation subtracting the surveyed low-flow water surface elevation. This relative gage height is no longer associated with Stewart's gage datum (or the benchmark used by Stewart). This relative gage height is also not affected by any carried-over errors potentially accumulated during his survey. A converted 1921 HWM elevation can then be obtained by adding this relative gage height to others' surveyed low-flow water surface elevation, so long as both Stewart's and others' survey locations and low-flow conditions are approximately similar. The converted 1921 HWM would be slightly affected by the low-flow water level difference between Stewart's and others' surveys.

[2] General Notes for Table 5: the stream flow at the Dalles is approximately 90% of the stream flow at Sedro-Woolley based on the ratio of the drainage area; and the flow-stage rating at the Dalles is approximately 2,000 cfs (+/-) per one-foot stage increase during low flows. It is also noted that Stewart's surveyed low-flow water surface elevations at slope sections are higher than the USACE surveyed elevations even though the stream flow was lower during Stewart's survey. This adds evidence that the datum used by Stewart is rather on a high side than on a low side.

Table 6 shows the conversion of four Stewart-surveyed 1921 HWMs, one at the old Concrete ferry site located approximately one mile upstream of the upper Dalles, two at Stewart's upper Dalles gage site, and one at Stewart's lower Dalles gage site (see Figure 1 for these locations and Stewart's surveyed HWM elevations based on his gage datum). The low-flow water surface elevations surveyed by Stewart based on his gage datum, and by others based on NGVD-29 at these sites, are listed in Table 6. A comparison between Stewart's HWM elevations based on his gage datum and the converted HWM elevations based on others' survey datum is also provided in the table. This comparison indicates that the differences between Stewart's surveyed HWM based on his gage datum and the converted HWM based on others' survey datum are not significant, within a range between –0.18 and +0.65 ft, which is within the data accuracy. This indicates that Stewart's gage datum and his surveyed HWM elevations are consistent with those based on the use of the NGVD-29 datum.

We have used these four Stewart's surveyed HWMs to estimate the 1921 flood peak discharge by two different methods. These two methods are: 1) using the backwater stage-discharge rating of the Baker River gage in combination with the HEC-RAS modeling and Stewart's HWM at the Concrete ferry site (see Section 2.3.3), and 2) using the stage-discharge rating at the current Concrete gage and Stewart's HWMs at the upper and lower Dalles gages (see Section 2.3.2). Alternatively, the four converted HWMs shown in Table 6 could be used to replace Stewart's original elevation data to estimate the 1921 flood peak discharge. We would not expect any significant difference of the estimates between uses of Stewart's data and the converted data, since the data difference as shown in Table 6 is not significant.

Table 6. Comparison of 1921 HWMs independent of datum difference

Location	Stewart 1922–23 Surveyed Elevation (ft) Based on 140.89 Gage Datum at Upper Dalles: 1921 HWM*	Stewart 1922–23 Surveyed Elevation (ft) Based on 140.89 Gage Datum at Upper Dalles: Low-Flow Water Level**	Relative Gage Height (ft) 1921 HWM Above Low-Flow Water Level	Similar Low-Flow Water Level (NGVD-29) Surveyed by Others**	Converted 1921 HWM Elevation (NGVD-29) Not Associated w/ Stewart's Gage Datum	Difference between Stewart's and Converted 1921 HWM Elevations (ft)
Near old Concrete Ferry Site	182.58[a]	151.92	30.66	152.1 (Skagit County, 2008)	182.76	–0.18
Upper Dalles Gage	175.75[b]	144.58	31.17	144.5 (COE, 1911)	175.67	0.08
	175.18[c]	147.55	27.63	147.4 (PIE, 2004)	175.03	0.15
Lower Dalles Gage	171.04[d]	144.95	26.09	144.3 (COE, 1911)	170.39	0.65
					Range of Difference =	**–0.18 to 0.65**

* See Figure 1 for Stewart's HWM elevations and location

** See Table 5 for Stewart's and others' low-flow water surface elevations

Notes: a. 182.58 = 32.0 (gage height) + 150.58 (gage datum), Stewart's survey notes, p. 85

b. 175.75 = 34.86 (gage height) + 140.89 (gage datum), Stewart's survey notes, p. 87

c. 175.18 = 34.29 (gage height) + 140.89 (gage datum), Stewart's survey notes, p. 87

d. 171.04 = 30.0 (gage height) + 141.04 (gage datum), Stewart's survey notes, pp. 54-55 & p. 67

Summary of Gage Datum Issue

The above-discussed comparison of low-water surface elevation surveys between Stewart's surveys and other partys' surveys (Table 5) demonstrates that Stewart's gage datum and HWM elevations are consistent with those based on the use of the NGVD-29 datum. And as discussed whether based on Stewart's surveyed gage datum or based on gage datum surveyed by others, Stewart's surveyed 1921 HWMs would produce similar flood stages (Table 6) and therefore, similar estimates. The USGS-published historical HWM elevations based on the use of the County's gage datum, instead of Stewart's gage datum, are incorrect and should be lowered by 1.8 ft.

As a result of this 1.8-ft gage datum downward correction, the USGS-revised 1921 flood peak discharges (see Table 4) would also need to be revised downward based on the stage-discharge rating at the current Concrete gage. More discussion on the reevaluation of the 1921 flood peak discharge using the stage-discharge rating of the Concrete gage is provided in Section 2.3.2.

2.2.3 Historical Flood Marks in Lyman – Hamilton Area

The towns of Lyman and Hamilton are located on the right bank floodplain of the Skagit River between RM 34 and 41. Both towns have historically experienced extensive flooding. Available flood marks in the area were recently collected and plotted on Figure 2.

- *1909 Flood Marks along Lyman-Hamilton Road*

  A 5,700-foot long profile of Lyman-Hamilton road that extends east from Jones Creek to Jims Slough was recently discovered at the County Public Works Department. Jones Creek joins the Skagit River at RM 35.1 on the east side of Lyman. The Lyman-Hamilton Road crosses the north side of Cockreham Island and is on the Skagit River floodplain. The discovered road profile presents a flood water line from a high point about 700 feet east of Jones Creek and continuing east about 5,000 feet. This flood water line is at El. 86.2 to El. 86.4, entitled ―H.W. of flood 1909".

- *1921 Flood Marks along old GNRR*

  The removed Great Northern Railroad (GNRR) used to run parallel to Lyman-Hamilton Road through the area. PI Engineering recently obtained a GNRR profile from BNSF Company, on which many 1921 flood marks were annotated, as well as finished track elevations. The 1921 flood marks range between El. 74.3 at RM 33.4, one half mile downstream of Lyman, and El. 95.5 at RM 39.5 in Hamilton. The 1921 flood marks vary between El. 84.5 and El. 85.4, along the reach of Lyman-Hamilton Road west of Jims Slough on the north side of Cockreham Island. Caution is required to interpret the plotted flood marks. For example, the flood mark El. 86.9 shown at the Jones Creek crossing was likely due to Jones Creek flows and not Skagit River flows. Similar situations probably occurred at the Muddy Creek crossing.

- *Stewart-Surveyed Flood Marks*

  Stewart surveyed several flood marks in the Hamilton-Lyman area during the winter of 1922-23. These include the 1909 flood El. 96.17, the 1917 flood El. 95.62 and the 1921 flood El. 96.46 at a cigar store building in Hamilton (Stewart's notes, pp. 13-14), (about RM 39.9), and the 1921 flood El. 86.22 at the old Lyman ferry site (Stewart's notes, pp. 132-133) (about three quarter mile upstream from then new Lyman ferry site, or about RM. 37.9).

These flood elevations compare reasonably well with the flood marks shown on the Lyman-Hamilton Road and the old GNRR profiles.

A comparison of Stewart's historical HWMs at the cigar store building and the recent flood elevations surveyed by Skagit County at Smith house in Hamilton is discussed in Section 2.1.3. The comparison clearly indicates that the 1909, 1917 and 1921 flood historical flood peak discharges published by USGS (Table 4) are significantly overestimated.

2.2.4 Correlation with Flows in Sedro-Woolley

The USGS also published estimated peak flows at the site of the USGS gage location at Sedro-Woolley (32 miles downstream from the Dalles) for the four historic flood events. A gage has been in place at Sedro-Woolley since 1908. The flood peaks were estimated by Stewart at the same time he estimated the flood peaks at Concrete and are published by the USGS in Water Supply Paper 1527 (USGS 1961). Stewart had also made earlier estimates in 1918. In subsequent USGS studies, Bodhaine (1954) suggested values for the four floods; other estimates were made by Riggs & Robinson in 1950 and by Hidaka in 1954 for the 1897 and 1909 events (Table 7).

Table 7. Stewart and USGS peak discharge estimates for historical floods at Sedro-Woolley

	Stewart		USGS		
Flood	**1918**	**1923**	**Rigg & Robinson**	**Hidaka**	**Bodhaine**
1897	171,000	190,000	170,000	145,000	170,000
1909	169,000	220,000	190,000	175,000	200,000
1917	157,000	195,000	160,000	----	195,000
1921	----	210,000	170,000	----	210,000

(Source: Stewart 1918 & 1923 Reports; Proposed Revision of Skagit River Peaks, H.C. Riggs & W.H. Robinson, 11/16/50; Skagit River near Sedro-Woolley, Wash., Proposed revisions of historical flood_peaks, F. L. Hidaka, 1/12/54; Skagit River Flood Peaks, Memorandum of Review by G.L. Bodhaine, USGS, 5/13/54). Available at www.skagitriverhistory.com

Flood peaks for flood events are expected to be approximately the same (within a few percentage points) at Concrete and Sedro-Woolley, with an expectation that the furthest downstream location (Sedro-Woolley) will usually be a little higher. The incremental drainage area between Concrete and Sedro-Woolley is 270 square miles, about ten percent of the total drainage area of 2,737 square miles above the Concrete gage. There are no large floodplain areas that would add storage between Concrete and Sedro-Woolley that could reduce flood peaks significantly more than increases to the flood peak due to the local inflow in the same reach. Comparison of flood peaks for recent floods in 1990, 1995, and 2003, demonstrates that flows modeled by PI Engineering at the USGS Sedro-Woolley gage average 1.6% higher than flows recorded at the USGS Concrete gage. Recent analysis by the Corps (2005) and Northwest Hydraulic Consultants (2007) arrived at a similar conclusion.

Assuming that the relationship between flows at Sedro-Woolley and Concrete as discussed above is valid, Stewart's flow estimates at Concrete should be approximately 2% lower than his estimates at Sedro-Woolley. In fact, Stewart's estimates at Concrete for the historical floods average 15% higher than his concurrent estimated flood peaks at Sedro-Woolley for the years during which USGS gage records are available at Sedro-Woolley. For the 1897 flood, Stewart's flow estimate is 45% higher at Concrete than at Sedro-Woolley.

Table 8 presents a comparison of the peak flows estimated by Stewart at Sedro-Woolley and Concrete for the historic flood events. (Note: the differences shown in Table 8 would be about 5% less if using the USGS 2007 revised estimates at Concrete). The magnitude of the difference between Sedro-Woolley and Concrete for the 1897 flood is not consistent with any of the other flood events. This observation indicates that the HWM for the 1897 event at Concrete may have been inaccurately observed or recorded; or, this could have been the result of debris blockage at the Dalles, according to Stewart's interview with Leonard Everett (Stewart's Notes, p. 23) who stated that in 1897, the "log jam in the Dalles raised water 10 ft in 2 hrs." HWMs of other three events at Sedro-Woolley are based upon records of the USGS gage installed in 1908.

Table 8. Comparison of Stewart's peak discharge estimates (cfs) for four historical floods in the Skagit River at Concrete and Sedro-Woolley

Flood Date	Stewart Estimates @ Sedro- Woolley	Stewart Estimates @ Concrete	% Diff
Nov. 19, 1897	190,000	275,000	-45%
Nov. 30, 1909	220,000	260,000	-18%
Dec. 30, 1917	195,000	220,000	-13%
Dec. 13, 1921	210,000	240,000	-14%

Although reliable stage records at Sedro-Woolley are available for the period starting in 1908, it has always been difficult to establish a rating curve at that location. At this time, it is impossible to develop a rating curve that would reflect the river channel characteristics current at the time of the four historical floods. Part of this difficulty arises from the effect of debris blockage of the SR-9 Bridge and the abandoned railroad bridge at the gage, and a significant factor is the changes in river bank levee and channel geometry that have occurred in the course of nearly a century, particularly immediately downstream of Sedro-Woolley (cutting off the Sterling Bend). These uncertainties preclude an accurate estimate of river flows based upon the stage records at Sedro-Woolley.

2.2.5 Review of Stewart's Slope-Area Computations - Background Information

This section provides a background review of Stewart's slope-area computations; revisions made in the USGS 2007 estimates and in PI Engineering 2011 estimates, using the slope-area method, are discussed in Section 2.2.6 and Section 2.3.1, respectively. The data used and the 1921 flood computations performed by Stewart for the slope sections below the Dalles are provided in Exhibit B of Stewart's unpublished report (Stewart 1923). Table 9 summarizes the slope-section hydraulic parameters used in Stewart's computations and the 1921 flood peak discharges computed by Stewart.

Table 9. Slope-section hydraulic parameters and 1921 flood peak discharges computed by Stewart

Slope-Area Reach	Mean Flow Area (sq. ft)	Mean Hydraulic Radius (ft)	Water Surface Fall (ft)	Reach Length (ft)	Slope of Hydraulic Grade Line	Manning's "n" Value	Computed 1921 Peak Discharge (cfs)
XS1-XS2	18,500	26.1	2.11	1,860	0.00113	0.033	244,000
XS2-XS3	18,000	24.2	2.62	2,190	0.00120	0.033	234,000
XS1-XS3	18,200	25.1	4.73	4,050	0.00117	0.033	240,000

Note: Stewart used flow areas in his computations were 18,000, 19,000, and 16,900 for XS1, XS2, and XS3, respectively. His original surveyed XS3 area was 16,200.

A discussion of Stewart's slope-area computations is provided below.

Incomplete Energy Equation used in Stewart's Computations

Stewart assumed a uniform flow (Chow 1959, Chapter 5) (meaning flow velocity remains constant from section to section). Stewart therefore used the incomplete energy equation which ignores variation of velocity head between slope sections when applying the slope-area method. The flow in the slope-area reaches is a gradually varied flow (Chow 1959, Chapter 9), not a uniform flow. The full energy equation that includes the velocity head variation should be used in this case when applying the slope-area method.

The uniform flow assumption made by Stewart was probably necessary and likely due to limited application of the slope-area computation method to only situations involving uniform flow in Stewart's time. Stewart, in a memorandum enclosed in his June 1, 1950 letter to USGS stated that, *"In choosing a slope section, the most important feature is the selection of one where the stream is neither gaining or losing velocity; i.e., selecting a section where the average velocity at the upper end of it (and throughout) is the same as for the lower end. If this is not done, there is a gain or loss in velocity head which cannot be taken care of in the regular formula. In practice, the ideal cannot be attained, but it should be approached as closely as possible."* After Stewart's time, the slope-area computation method was improved and expanded to enable application to situations involving gradually varied flow (Chow 1959, pp. 147–148).

Incorrect Flow Area used for Lower Slope Section (XS3)

Stewart surveyed the lower slope section (XS3) on January 29, 1923 (see Stewart's survey notes, p. 78) (Stewart 1922–23). On January 31, 1923, while surveying the upper slope section (XS1), Stewart checked the graduations on rope he used for the survey and noted this: *"Checked graduations on rope as follows. Markers 80 to 160, 84 1/2 on steel tape = 80 on rope 0 - 320 portion of rope. Taped dry and before stretching, 41.2 on steel tape = 40 on rope. This will apply from graduation 320 on. Checks on rope graduation were made while rope was still stretched across river. It is not certain that these checks are applicable to the lower cross section also but probably will have to be assumed so"* (see Stewart survey notes, p. 69). This meant that Stewart assumed his survey rope was also stretched by approximately 5% when he was surveying the lower slope section (XS3) and went ahead to modify his originally surveyed lower slope section without an actual verification of his assumption. (For Stewart's originally surveyed and modified data, see Stewart's survey notes, p. 78.)

Figure 8 shows plots of Stewart's originally surveyed and later-modified lower slope section. In 2004, the USGS (Mastin and his field crew) surveyed this section. The USGS 2004-surveyed lower slope section is also shown in the figure for a comparison. This comparison indicates that Stewart's originally surveyed section appears accurate, matching the channel width and bottom elevations with the 2004-surveyed section better than his modified section. This comparison assumes that the 1923 and 2004 surveys are approximately at the same location, and that the change in cross section from 1923 to 2004 at this location is not significant.

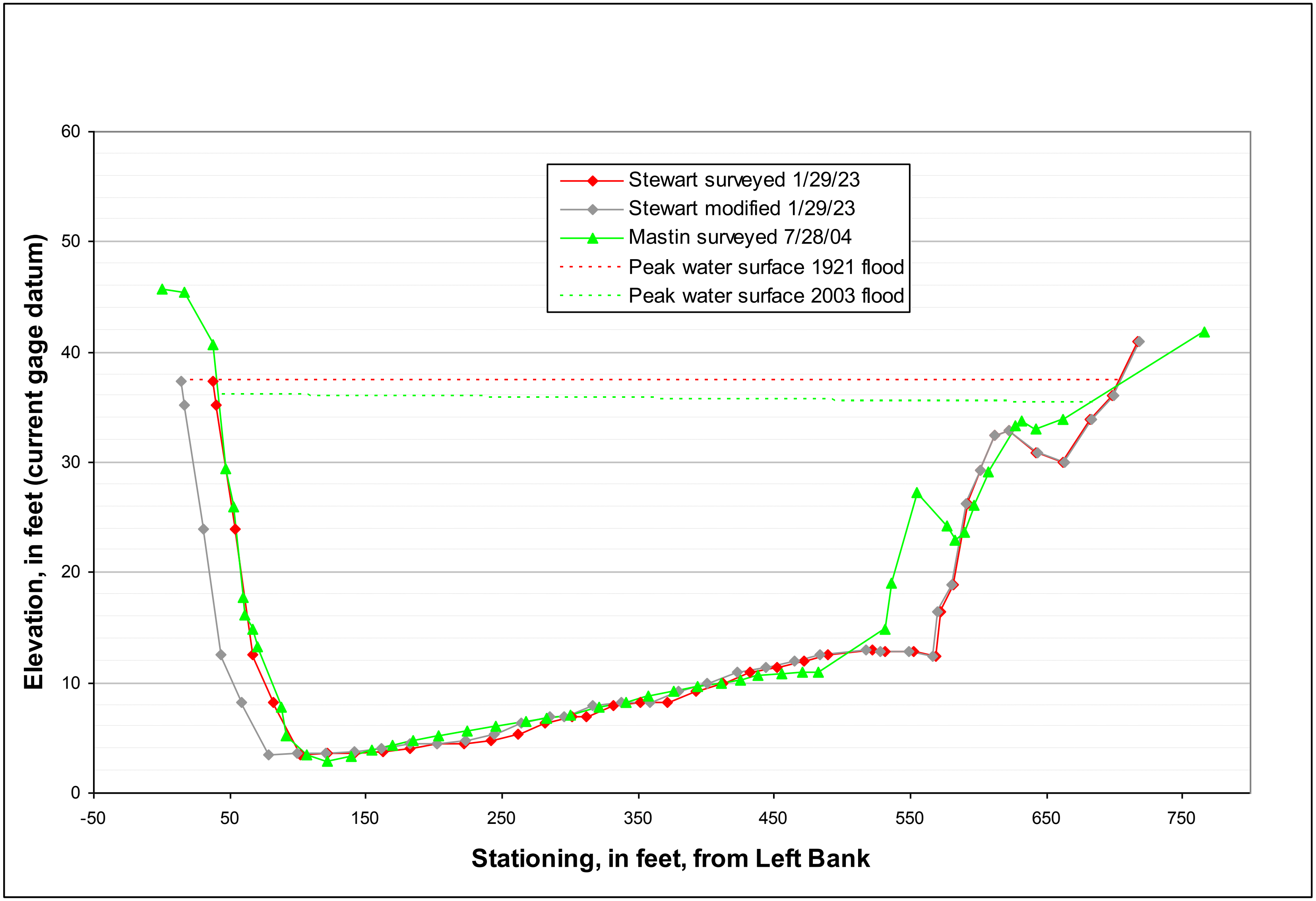

Figure 8. Slope Section XS3 – Skagit River near Concrete, WA

Unreasonable Hydraulic Grade Line Slope Used for Upper Slope-Section Reach (XS1–XS2)

Stewart used 2.11 ft for the 1921 water surface fall between the upper and middle slope sections (XS1–XS2). As the flow is expanding from XS1 to XS2 and contracting from XS2 to XS3, the slope of hydraulic grade line should be flatter between XS1 and XS2 than between XS2 and XS3. Stewart's calculation used a water surface fall value corresponding to a hydraulic grade line slope of 0.00113 between XS1 and XS2, which is not reasonable since this is very similar to the slope of 0.00120 between XS2 and XS3 (Table 9).

Stewart, in a memorandum enclosed in his June 1, 1950 letter to USGS stated that, "If not too difficult, it is suggested that for this important check work five cross-sections be taken, say about 700—1,000—2,700--3,700 and 4,700 feet downstream from the mouth of "The Dalles". These five cross-sections will make four stream sections available. It is important that the first one of these below "The Dalles" be far enough below so that all of the velocity head gained in "The Dalles" is lost; i.e., that the water has at least reached its maximum level resulting from the loss in velocity."

Figure 9 shows elevation plots of Stewart-surveyed 1921 HWMs (see Figure 1 for HWM location) and USGS-surveyed 2003 HWMs (surveyed in summer 2004 and provided by Mastin in the spreadsheet Concrete_03_SAM.xls) in the Dalles and slope-area reaches. The slope sections XS1 and XS2 are located below the Dalles at 618 and 2,479 feet, respectively (Stewart 1923, Exhibit B, p. 2). Stewart's suggestion would exclude the use of the HWM data (particularly, the three higher data points, 171.83, 172.09 and 172.13, due to the residual of "the velocity head gained in The Dalles") at XS1 located within the Stewart suggested 700 foot distance and not far enough below the Dalles to dissipate "all of the velocity head gained in The Dalles".

Excluding the HWMs at XS1 and using the other HWMs below XS1, we interpolated a fall of the 1921 water surface between XS1 and XS2 to be 0.85 foot. The corresponding hydraulic grade line slope between XS1 and XS2 was then calculated to be 0.000457 (= 0.85/1860). As shown in Figure 9, the 2003 flood HWMs also support the flatter hydraulic grade line slope between XS1 and XS2. Using the USGS-estimated average peak elevations 38.45 and 37.50 provided in the spreadsheet and listed in Figure 9, we estimate the hydraulic grade line slope between XS1 and XS2 for the 2003 flood to be 0.000511 (= (38.45 – 37.50) / 1860), which is very similar to the above-estimated slope of 0.000457 for the 1921 flood.

The hydraulic grade lines used by Stewart for the upper and lower slope-section reaches (XS1–XS2 and XS2-XS3) are plotted on Figure 9, as well as those estimated by PI Engineering for the upper slope-section reach (XS1–XS2). Our comparison assumes no revision to Stewart-used lower reach hydraulic grade line, which likely has great uncertainties as there are no intermediate HWM data available between XS2 and XS3 from Stewart's survey to support this hydraulic grade line. (Note: in our view, a drop close to 2.0 ft from XS2 to XS3 based on the 2003 HWMs is more reasonable than the 2.62-ft drop used by Stewart and the USGS.)

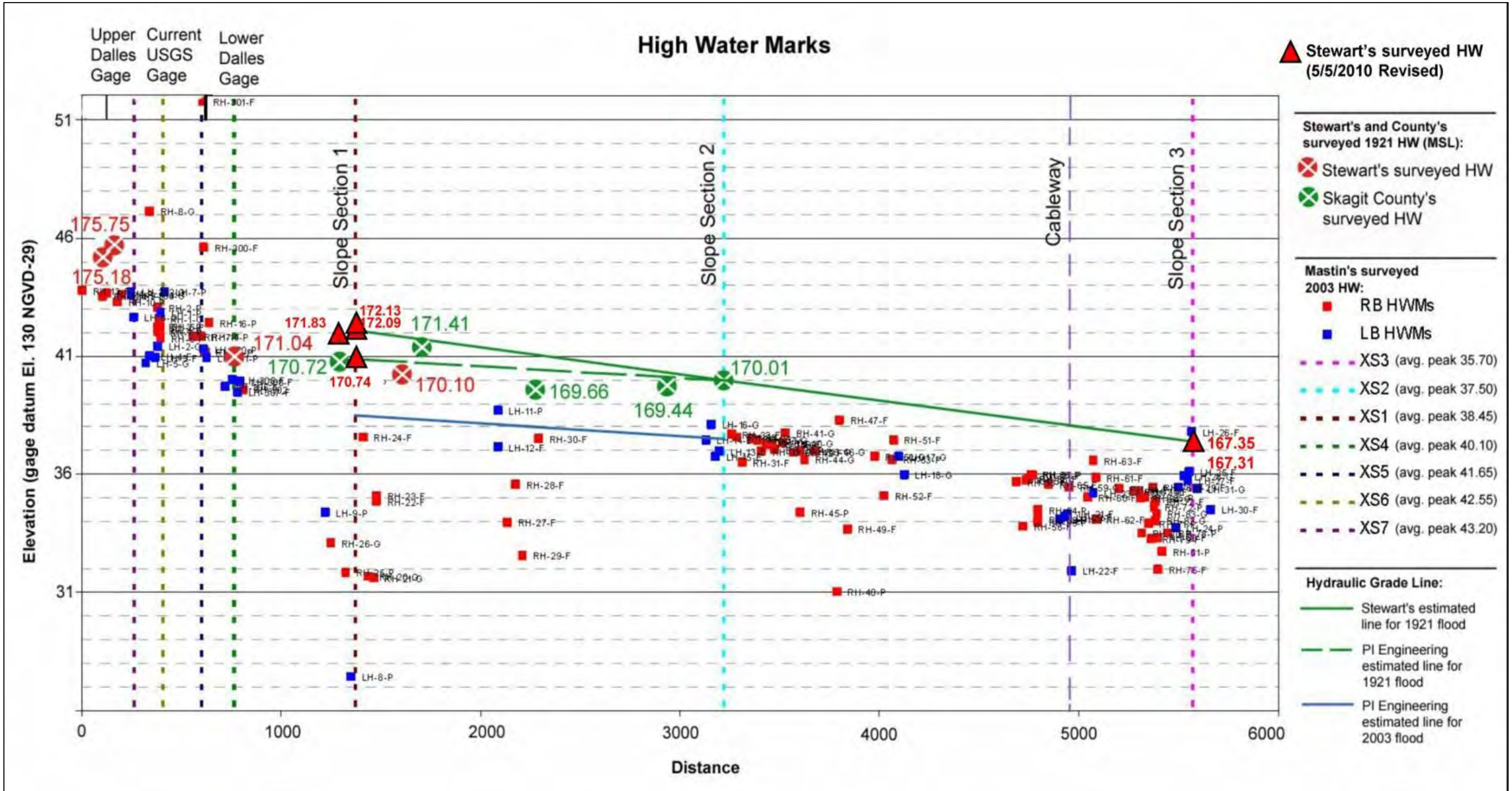

Figure 9. 1921 and 2003 flood high water marks surveyed by Stewart (in 1922–23) and USGS (in summer 2004)

<u>Unknown Quality of High Water Marks; Stewart's Termed "Surge Effects"</u>

Stewart's HWMs in the slope-area reaches are based on natural indicators such as sand deposited in moss on trees, moss scoured from trees, mud marks, and drift along bank lines. Stewart did not attempt to characterize the quality of his observed HWMs (as a general practice today, each observed HWM is assigned to one of these four categories: poor, fair, good, or excellent—P, F, G, or E—based on the condition, type, and accuracy of the HWM). Therefore, there is no way to know the quality of Stewart's HWMs in the slope-area reaches. And we would not be able to characterize the accuracy of the 1921 peak discharge estimate if we were to rely only on these HWMs.

Regarding surge effects on the slope-section HWMs, Stewart, in his memorandum enclosed in his June 1, 1950 letter to USGS stated that, ―Another feature of some importance, although how much is uncertain, is the amount of surging in the stream at the ends of the sections during the crest of the flood. Manifestly the only elevations available, when the flood crest is based on high water marks, is the crest of the surges, whereas what is needed is the mean level of the water at the time of the flood crest. Information as to this feature can be obtained by determining the amount of surging at the cross-sections for a lower flood, and then by means of the relation of the surging at the water stage records for both floods, determine the surging for the higher flood at the cross-sections." These statements indicate that, first, Stewart's surveyed 1921 HWMs at the slope sections were the crest of the surges and that, second, Stewart suggested these HWMs be adjusted for the amount of surging observed during other flood events at these slope sections so that the mean water level at the time of the 1921 flood crest could be determined for use to more accurately estimate the peak discharge of the 1921 flood.

Wind and wave actions can cause the debris lines to be higher than the actual water surface. Floating debris can cause large surges. The Skagit River is known to carry heavy floating debris during large flood events (As an example, Stewart's survey notes, p. 23, state ―Leonard Everett says 1897 flood about 9" lower than 1909. He says that log jam in Dalles raised water 10 ft in 2 hrs."). All of these factors could contribute to the amount of ―surging effects" termed by Stewart.

<u>Incorrect HWM Application in Stewart's Slope-Area Computations Leading to Incorrect 1921 Flood Peak Discharge Estimate</u>

As discussed in Section 2.1.4 about the velocity distribution at the cableway near XS3, the overbank water is moving slower and is closer to the energy grade line, which is higher than the water surface elevation by an amount of the velocity head. High water marks in (or near) the overbank area are higher than in the channel.

Stewart collected bank-line HWMs at the slope sections are more representative of the energy gradient line elevations than the section-mean water surface elevations at the peak of the 1921 flood. Stewart in his slope-area computations incorrectly assumed that the HWMs represented the mean water surface levels at the time of the flood crest across the river channel sections. Based on Stewart's estimated 1921 flood peak discharge of 240,000 cfs and our estimated XS3 flow area of 14,000 square feet (after reducing the area corresponding to the velocity head from Stewart's originally surveyed XS3 area of 16,200 square feet at his HWM gage height), the velocity and velocity head would be 17.1 fps and 4.6 ft, respectively. These velocity and velocity head are much higher than the ranges of 11.09 to 11.68 fps, and 1.9 to 2.1 ft, respectively, based on the USGS two measurements at the cableway, as discussed in Section 2.1.4, and are unreasonable.

Applying Stewart's 1921 HWMs at the slope sections as the mean water surface elevations of the flood led to an incorrect estimate of the 1921 flood peak discharge made by Stewart.

Use of Un-Supported HWMs Leading to Un-Reliable 1897, 1909 and 1917 Flood Peak Discharge Estimates

Stewart's transferred 1909 and 1917 HWMs as discussed in Section 2.2.1 are not supported by his surveyed HWMs at the Wolfe and McDaniels residences. His transferred 1897 HWM was based on un-reliable flood marks. Using these transferred HWMs in combination with the use of his gage rating based on the above-discussed incorrect 1921 flood peak discharge estimate, Stewart's estimates of the 1897, 1909 and 1917 flood peak discharges are not reliable.

2.2.6 Review of USGS 2007 Slope-Area Computations

The USGS in 2007 revised Stewart's 1923 estimates (USGS, 2007) as follows:

(1) A complete energy equation was used. This is an improvement over Stewart's slope-area computations.

(2) A lower Manning's n value was used. Stewart used 0.033 in his 1923 computations, and the USGS revised computations use 0.0315 for the slope sections.

(3) The USGS computations use only data collected by Stewart at the lower two slope sections (XS2 and XS3), including Stewart's HWMs and channel cross section data at these two sections.

As a result of the USGS revised slope-area computations, Stewart's 1921 flood peak discharge estimate was reduced by approximately 5 percent. A similar percentage reduction was applied by the USGS to Stewart's other three flood peak discharge estimates (rounded to the nearest 5,000 cfs).

Stewart's 1923 original estimates and the USGS 2007 revised estimates of the 1897, 1909, 1917 and 1921 flood peak discharges are listed in Table 4.

Our review comments on the USGS 2007 revised estimates are discussed below.

Incorrect Flow Area used for Lower Slope Section (XS3)

As previously discussed (Section 2.2.5) and shown in Figure 6, it is our opinion that Stewart's originally surveyed XS3 is more accurate than his later-modified XS3. Stewart assumed his survey rope was stretched by approximately 5% when he was surveying XS3 and went ahead to modify his originally surveyed XS3 without an actual verification of his assumption. Stewart's assumption is incorrect based on a comparison with the USGS 2004 surveyed section at XS3 location as shown in Figure 8.

Only Lower Two Slope Section Data Used in USGS Revised Slope-Area Computations

The USGS 2007 revised estimate of the 1921 peak discharge is based only on data taken at the lower two slope sections (XS2 and XS3), ignoring all HWM data between slope sections XS1 and XS2. There are only three HWMs surveyed by Stewart, one at XS2 and two at XS3 (see Figure 9). There is no evidence to suggest that the data taken at the lower two slope sections have less uncertainties or better quality than other HWMs. There are no other intermediate HWMs between XS2 and XS3 to support the certainty and the use of the hydraulic gradient line slope, 0.00120 for XS2-XS3 (see Table 9), in the slope-area computations. (As we previously noted, a drop close to 2.0 ft from XS2 to XS3 based on the 2003 HWMs surveyed and plotted by the USGS in Figure 9 is more reasonable than the 2.62-ft drop used by Stewart and the USGS in their estimates of the 1921 flood peak discharge.)

To counteract the uncertainties, Stewart suggested, "to take several sections and average the results obtained from them" (Stewart 1950). There would be less uncertainties associated with the use of data from more sections than from only two sections. Even excluding the use of the XS1 data as suggested by Stewart due to the residual of "the velocity head gained in The Dalles", as discussed in Section 2.2.5, there are several Stewart's HWMs between XS1 and XS2 that can be used to provide a more balanced estimate of the 1921 flood peak discharge. It is our opinion that the USGS' revised estimate, which uses only HWM data from the lower two slope-sections, is a biased estimate.

Incorrect HWM Application in USGS Revised Slope-Area Computations Leading to Incorrect 1921 Flood Peak Discharge Estimate

As discussed in Section 2.1.4 about the velocity distribution at the cableway near XS3, the overbank water is moving slower and is closer to the energy grade line, which is higher than the water surface elevation by an amount of the velocity head. In this case, using the USGS revised 1921 flood peak discharge of 228,000 cfs and our estimated XS3 flow area of 14,000 square feet (after reducing the area corresponding to the velocity head from Stewart's originally surveyed XS3 area of 16,200 square feet at his HWM gage height), we would have a section-mean velocity of 16.3 fps and a velocity head of 4.1 ft at XS3. These velocity and velocity head at XS3 are much higher than the ranges of 11.09 to 11.68 fps, and 1.9 to 2.1 ft, respectively, based on the USGS two measurements at the nearby cableway, as discussed in Section 2.1.4, and are unreasonable.

Applying Stewart's 1921 HWMs at the slope sections as the mean water surface elevations of the flood led to an incorrect revision of the 1921 flood peak discharge made by the USGS.

We note here that the Manning's "n" value of 0.0315 used in revising the 1921 flood peak discharge estimate by the USGS was based on their "n" value verification study using the surveyed 1949 flood HWMs in the slope-area reach but at different cross section locations than the XS1, XS2, and XS3 (see USGS 2007). Due to the use of the HWMs at different locations, and mainly also due to the similar incorrect application of the HWMs in the "n" value verification study, we view this "n" value verification is not valid but would not rule out the use of the USGS suggested 0.0315 at the slope sections, XS1, XS2, and XS3.

Use of Un-Supported HWMs Leading to Un-Reliable 1897, 1909 and 1917 Flood Peak Discharge Estimates

Stewart's transferred 1909 and 1917 HWMs as discussed in Section 2.2.1 are not supported by his surveyed HWMs at the Wolfe and McDaniels residences. His transferred 1897 HWM was based on un-reliable flood marks. The USGS 2007 revised estimates of the 1897, 1909 and 1917 peak discharges are based on Stewart's transferred HWMs, and are also not reliable.

2.3 Reevaluation of 1897, 1909, 1917 and 1921 Historical Flood Peak Discharges

To reduce the uncertainty, PI Engineering used several analytical methodologies to reevaluate the peak flow estimates of the historic flood events. These include:

(1) Estimating the 1921 flood peak discharge by reevaluating the slope-area methodology using correct application of Stewart's HWMs;

(2) Estimating the 1921 flood peak discharge using the stage-discharge rating of the Concrete gage

(3) Estimating the 1909, 1917 and 1921 flood peak discharges using the backwater stage-discharge rating of the Baker River gage, in conjunction with the HEC-RAS modeling and Stewart's HWMs at the old Wolfe/McDaniels residences in Concrete, and

(4) Estimating the 1897, 1909 and 1917 flood peak discharges using HEC-RAS modeling and Stewart's flood marks downstream of Concrete.

The remainder of this section provides details into these methodologies and conclusion resulting from this reevaluation.

2.3.1 Estimate of the 1921 Flood Peak Discharge by Reevaluating the Slope-Area Methodology Using Correct Application of Stewart's HWMs

Stewart's field notes convey a picture of an extraordinary and conscientious study effort. It is clear Stewart focused on collecting as much data as possible and then drawing accurate conclusions from that data. But this was in 1923, and Stewart did not have the benefit of the 85 years of gage data, made available through his efforts, that we have access to today and which provides perspective regarding his conclusions; further, Stewart did not have access to modern hydraulic modeling techniques, routinely used today, and relevant for reevaluating Stewart's data. As discussed in Section 2.2.5, Stewart's slope-area computations contain errors that include the use of an incomplete energy equation, the use of an incorrect flow area for the lower slope section (XS3), and the use of unreasonable hydraulic grade line slope for the upper slope-section reach (XS1-XS2). PI Engineering performed a new reevaluation of the 1921 flood using the slope-area method, incorporating corrections of Stewart's computation errors, and using the correct application of Stewart's surveyed HWMs. A summary of this reevaluation is provided below.

- As discussed in Section 2.1.4, the 2-foot velocity head, based on the velocity measurements at the cableway section, is representative of the lower slope reach (XS2-XS3). Based on Stewart's surveyed data, the flow velocity in the upper slope reach (XS1-XS2) is less than that in the lower slope reach, and therefore the velocity head is smaller in the upper reach. We believe that the use of a 1.5-ft adjustment to Stewart's HWMs in the slope-area sections is appropriate and is the correct application of Stewart's HWMs in the slope-area calculations.
- Table 10 presents the revised estimate of the 1921 flood peak discharge using the slope-area method with the 1.5-ft adjustment to Stewart's HWMs. The slope-area reevaluation was conducted following the procedure of computation (involving six computation steps) outlined in Chow's Open-Channel Hydraulics, pp. 147–148 (Chow 1959). The reevaluation uses: (1) the complete energy equation for the slope-area method applicable to the gradually varied flow observed in the slope-area reaches; (2) the revised flow area based on Stewart's originally surveyed lower slope section (XS3); (3) the revised hydraulic grade line slope of the upper slope-section

reach supported by all intermediate HWMs surveyed by Stewart and the County between XS1 and XS2 (see Figure 7); and (4) all associated computational revisions. Stewart's original estimates using Manning's "n" value of 0.0330 for all three slope-area reaches and the USGS-revised estimate using Manning's "n" value of 0.0315 for the lower slope-area reach are also shown in the table for comparison.

Table 10. Revised estimates of 1921 flood peak discharge using slope-area method and Stewart-surveyed data

	Adjustment (ft) to Stewart's HWMs	Manning's "n" value	Reach XS1–XS2	Reach XS2–XS3	Reach XS1–XS3	Average
	-1.5	0.0330	148,100	180,500	169,000	165,867
	-1.5	0.0315	155,500	186,900	176,100	172,833
Stewart's original estimates (1923)	0.0	0.0330	244,000	234,000	240,000	**240,000**
USGS revised estimate (2007)	0.0	0.0315	N/A	228,000	N/A	**228,000**

- Based on the slope-area method with the corrected application of Stewart's HWMs, PI Engineering concludes that a reasonable 1921 flood peak discharge estimate would be in the range of 166,000 to 173,000 cfs, using Manning's "n" values between 0.0315 and 0.033.
- This range of estimates, from 166,000 cfs to 173,000 cfs, is very close (with a difference of -2.2 to +1.9 percent) to 169,700 cfs estimated by PI Engineering using the backwater stage-discharge rating of the Baker River gage in conjunction with the HEC-RAS model and Stewart's HWMs at the Wolfe residence in Concrete, a different methodology (discussed in Section 2.3.3) than the slope-area method.

2.3.2 Estimate of 1921 Flood Peak Discharge Using Stage-Discharge Rating of Concrete Gage

The stage-discharge rating at the gage near Concrete (#12-194000) is very stable, as demonstrated by the two highest discharges measured about 71 years apart. These highest two discharges are 135,000 cfs on 2/27/1932 and 138,000 cfs on 10/21/2003, with a gage height of 38.68 above the current gage datum 130.00 for both discharges (data available at http://waterdata.usgs.gov/wa/nwis/measurements/?site_no=12194000&agency_cd=USGS). Using Stewart-surveyed HWMs at the Dalles and the extended stage-discharge rating provides another method to estimate the 1921 flood peak discharge. PI Engineering performed a new reevaluation of the 1921 flood peak discharge using the current gage stage-discharge rating in conjunction with the use of Stewart's surveyed HWMs at Upper and Lower Dalles (see Figure 1), as discussed below.

Transferring of Stewart's 1921 HWMs to Current Gage Site

Three Stewart-surveyed 1921 HWMs (expressed in MSL) are shown in Figure 10 (an enlarged plot of Figure 9 showing only HWMs in the Dalles gorge), including 175.18 and 175.75 at his upper Dalles gage and 171.04 at his lower Dalles gage. Stewart noted that the upper Dalles is 695 ft above the lower end of the Dalles (Stewart's survey notes, p. 62). Figure 10 shows that the current gage (XS6) is located approximately 365 ft above the lower Dalles (XS4). This leads to an estimated distance of 330 ft between Stewart's upper Dalles gage and the current gage (which is more than "about 200 feet above present gage" stated incorrectly by the USGS-published Water Supply Paper 612 (USGS 1925, p. 62)).

Since the current gage is located approximately in the middle of Stewart's upper and lower Dalles gages, all of his 1921 HWMs at both gages are used (in order to avoid a biased estimate) to estimate the 1921 HWM at the current gage.

To transfer Stewart's HWMs from the upper and lower Dalles to the current gage site, we first estimate two hydraulic grade lines between the upper Dalles and the current gage, and between the current gage and the lower Dalles, using the USGS-surveyed 2003 HWMs. These two hydraulic grade lines are plotted in Figure 10 (green lines 1 and 2). The 1921 flood hydraulic grade lines are expected to be closely parallel to the estimated 2003 flood hydraulic lines, as the channel geometry through the Dalles did not substantially change at high flood stages over the last 80 years, and the difference between the USGS-published 2003 flood peak discharge (166,000 cfs) and PI Engineering-estimated 1921 flood peak discharge (169,700 cfs, discussed later in Section 2.3.3) is not substantial. As shown in Figure 10, parallel lines (red lines 3, 4 and 5) are drawn to connect each of the three Stewart-surveyed 1921 HWMs to the current gage site. The estimated 1921 HWMs at the current gage site are thus determined by the elevation points where the hydraulic grade lines intersect the XS6 vertical line.

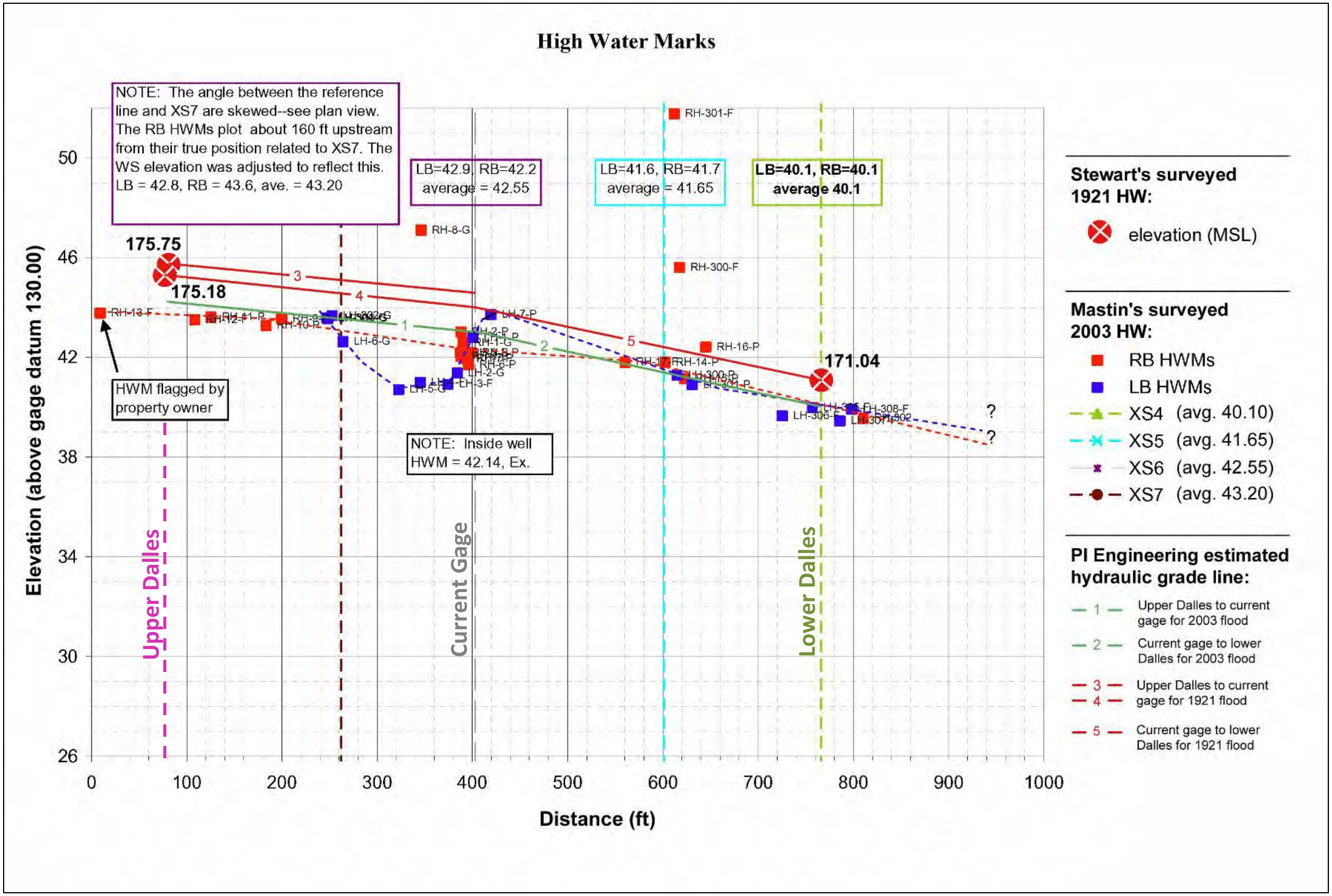

Figure 10. 1921 and 2003 HWMs in the Dalles gorge (original source of data: USGS-provided spreadsheet – Concrete_03_SAM.xls)

Table 11 lists Stewart's HWMs at the upper and lower Dalles and the estimated 1921 HWMs at the current gage site. As shown in the table, the estimated 1921 HWMs at the current gage are between 174.00 and 174.57, averaging 174.19 (or 44.19, rounded to 44.2, above the current gage datum 130.00).

Table 11. Transferring of Stewart's HWMs to current gage site

Stewart-surveyed 1921 HWM (MSL)		Estimated Water Surface Drop (ft) to current gage	PI Engineering – Estimated 1921 HWM (MSL) at current gage
Upper Dalles	175.75	1.18	174.57
	175.18	1.18	174.00
Lower Dalles	171.04	–2.96	174.00
			Average = 174.19

We believe that our estimated water surface drop of 1.18 ft for a distance of 330 ft from Stewart's upper Dalles gage to the current gage is conservative. A crest stage gage was installed in 2010 by Skagit County about 200 ft upstream of the USGS Concrete gage. This new county gage and the USGS Concrete gage recorded a water surface drop of 1.05 ft during a flood peak of 81,900 cfs on December 12, 2010. Based on this new data, we would expect that the water surface drop from Stewart's upper Dalles gage to the USGS current gage would be much higher than 1.18 ft, more likely closer to 2 ft for the 1921 flood.

Determination of 1921 Flood Stage Inside the Gage

The above-estimated 1921 average HWM elevation of 174.2 (rounded from 174.19) cannot be used directly on the current gage stage-discharge rating to estimate the 1921 flood peak discharge. The current gage stage-discharge rating is based on the stage readings inside the gage well, while the estimated HWM is outside the gage. The flood stages inside and outside the gage well can be significantly different due to surge effects.

The amount of surging and wave wash for the 2003 flood is estimated to be 0.9 to 1.6 ft at the gage site, based on the USGS-surveyed HWMs (see Figure 10, the 0.9 to 1.6 range calculated between HWMs inside well HWM = 42.14 and outside well HWMs = 43.021 and 43.715 for surveyed points RH-2 LH-7, respectively). Using 0.9 and 1.6 ft for the estimated amount of surging for the 1921 flood at the current gage site, the 1921 flood stage inside the gage well would be 173.3 and 172.6, respectively (or 43.3 and 42.6 above the current gage datum of 130.00).

1921 Flood Peak Discharge Based on the Stage-Discharge Rating

Figure 11 shows the stage-discharge rating curve for the current gage. Based on the estimated 1921 gage height of 42.6 and 43.3 at the gage, the

1921 flood peak discharge is estimated to be 169,000 and 175,000 cfs, respectively. This range of estimates, from 169,000 to 175,000 cfs is very close (with a difference of -0.4 to +3.1 percent) to 169,700 cfs estimated by PI Engineering using the backwater stage-discharge rating of the Baker River gage in conjunction with the HEC-RAS model and Stewart's HWMs at the Wolfe residence in Concrete, a different methodology (discussed in Section 2.3.3) from this method using the stage-discharge rating of the Concrete gage. It is noted that Mastin in the USGS 2007 estimate did not attempt to utilize the existing rating curve for the Skagit River near Concrete gage to estimate the 1921 flood. Instead, he modified the rating curve to fit his Slope-Area calculated flow for the 1921 flood. We believe the existing stage-discharge rating curve is more reliable than the Slope-Area calculation and the curve should not be modified based on an extremely approximate calculation.

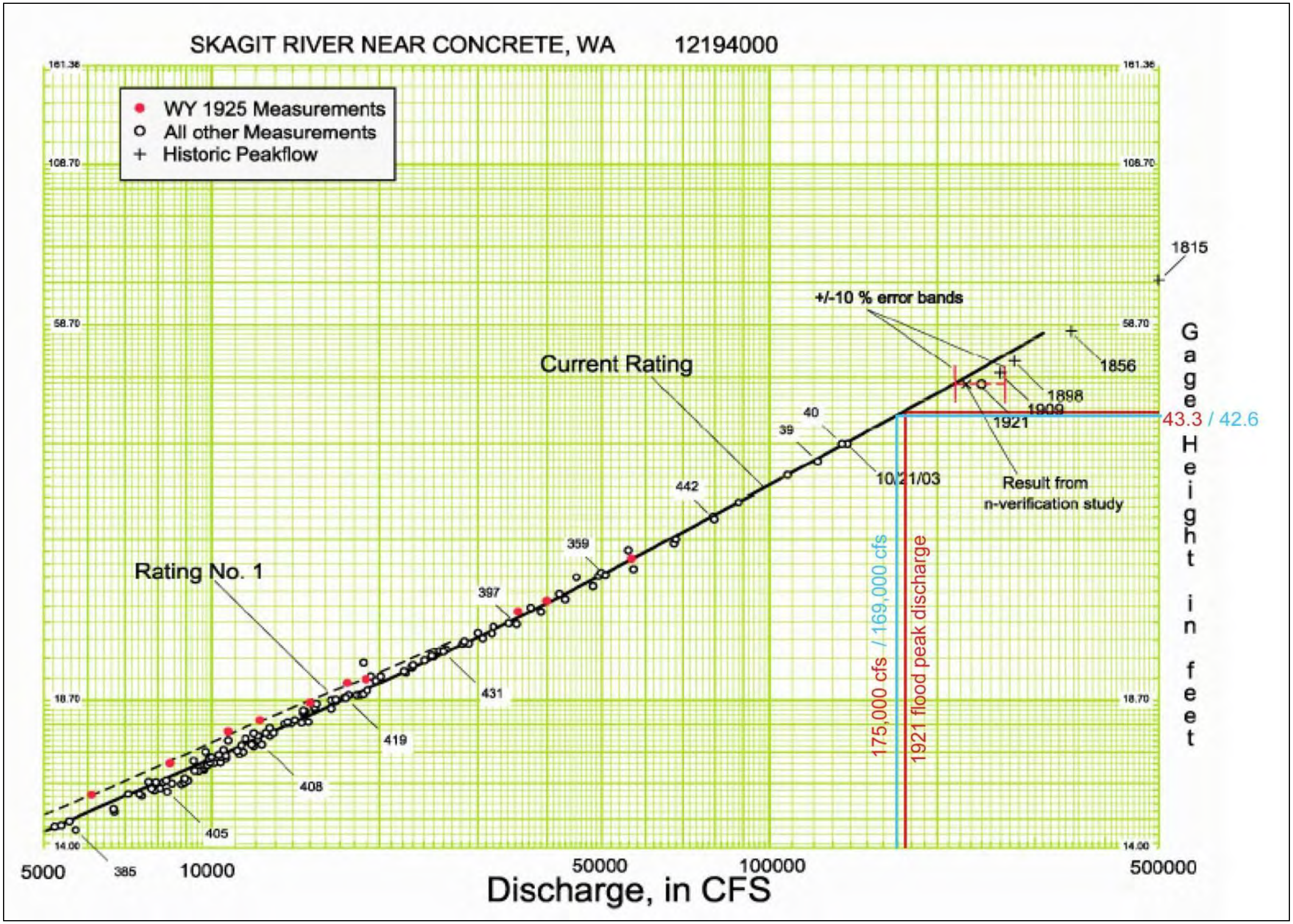

Figure 11. Stage-discharge Rating Curve for the Skagit River near Concrete (provided by USGS, May 2004)

1921 Flood Peak Discharge Estimate Using Transferred Stewart's HWM Independent of Stewart's Upper Dalles Gage Datum

The above-estimated 1921 flood peak discharge range of 169,000 and 175,000 cfs, is based on the estimated 1921 gage height range of 42.6 and 43.3 at the current gage. This estimated gage height range is based on transferring Stewart-surveyed HWMs that are dependent on his upper Dalles gage datum of El. 140.89. However, as discussed in Section 2.2.2, the USGS has asserted that Stewart's gage datum should be El. 142.69 (or rounded to 142.7), 1.8-ft higher. We have pointed out that Stewart-surveyed low-flow water surface elevations match well with other parties' surveys as shown in Table 5; it is our opinion that Stewart's upper Dalles gage datum was correctly surveyed, not as asserted by the USGS to be 1.8-ft too low.

Alternatively as illustrated in Table 6, we can convert Stewart's HWMs to other's gage datum, and use these converted HWMs independent of Stewart's gage datum to estimate the 1921 flood peak discharge. Following provides an example of this alternative approach using Stewart's HWM data independent of his gage datum.

This example illustrates the conversion of Stewart's 1921 HWM data to the USGS current gage datum, and the transfer of the converted HWM data to the current gage site to use the discharge-rating at the current gage to estimate the 1921 flood peak discharge. The steps for the conversion, transfer and estimate include:

(1) Use Stewart-surveyed two gage heights of 34.29 and 34.86 for the 1921 flood at his upper Dalles gage (Stewart's survey notes, p. 87; or Table 6 - Notes b and c). These gage heights are measured above Stewart's upper Dalles gage datum El. 140.89. The average gage height of 34.6 (rounded from 34.58), above his gage datum El. 140.89, is used below for the 1921 flood peak discharge estimate.

(2) Use Stewart-surveyed 12/23/1922 water surface level at a gage height of 6.66, or rounded to 6.7, above his upper Dalles gage datum El. 140.89 (Stewart's survey notes, p. 34; or Table 5). Based on the USGS published streamflow data, the mean daily flow at the Sedro-Woolley gage is 14,200 cfs and no flow data is available at the current gage near Concrete on 12/23/1922. We estimate the concurrent flow at the current gage near Concrete on 12/23/1922 is about 13,000 cfs, or about 90 percent of the flow at Sedro-Woolley (rounded to nearest 1,000 cfs), based on the drainage area ratio.

(3) The differential gage height is then calculated to be 34.6-6.7=27.9 between Stewart-surveyed 1921 HWM data and 12/23/1922 water surface level for the estimated 13,000 cfs at Stewart's upper Dalles

gage. This differential gage height is not associated with Stewart's upper Dalles gage datum, and can be converted to a 1921 HWM elevation based on other party's surveyed water surface level for 13,000 cfs at the upper Dalles using a datum that is independent of Stewart's gage datum.

(4) We note here that Stewart's 12/23/1922 survey indicated that there was only a 0.14 foot change in stage between the upper Dalles gage and below the Dalles ("6.52 WS below the Dalles", see Stewart's survey notes, p. 34). Therefore, it is evident that for low flows (13,000 cfs) the water surface elevation at the USGS current gage and the upper Dalles gage are essentially the same. This allows us to change the datum to the USGS current gage datum by substituting the USGS current gage reading for the Upper Dalles gage reading for this low flow condition. We then convert and transfer this differential gage height of 27.9 to the USGS current gage near Concrete, using the stage-discharge rating curve based on the USGS gage datum El. 130.0 to estimate the 1921 flood peak discharge, as explained below.

(5) The gage height for 13,000 cfs is 17.7 above the USGS gage datum El. 130.0, using the stage-discharge rating curve shown in Figure 11. A direct conversion and transfer of the 27.9 differential gage height between Stewart's 1921 HWM data and the water surface level for 13,000 cfs, results in a gage height 45.6 (=17.7+27.9) above the USGS gage datum, or El. 175.6 (=45.6+130.0, NGVD-29).

(6) However, this 45.6 gage height above the USGS gage datum needs to be adjusted for:

   a. the 1921 water surface drop conservatively estimated to be about 1.2 ft (rounded from 1.18 ft as discussed previously and presented in Table 11) for the 330-ft distance between Stewart's upper Dalles gage and the USGS current gage; and

   b. for the surge and wave effect estimated to be between 0.9 and 1.6 ft, averaging 1.3 ft between the bank-line HWM and the water surface reading inside the gage well, as previously discussed.

(7) With these adjustments, the gage height of Stewart's 1921 HWM data converted and transferred to the USGS current gage becomes 43.1 (=45.6-1.2-1.3) above the current gage datum, or El. 173.1.

(8) Based on the estimated 1921 gage height of 43.1 applying to the stage-discharge rating curve in Figure 11, the 1921 flood peak discharge is estimated to be 173,000 cfs. This estimate, 173,000 cfs is very close to the range of estimates between 169,000 and 175,000 cfs using Stewart's HWM data based on his gage datum.

(9) This estimate, 173,000 cfs, using Stewart's HWM data independent of his gage datum, is very close (with a difference of +1.9 percent) to 169,700 cfs estimated by PI Engineering using the backwater stage-discharge rating of the Baker River gage in conjunction with the HEC-RAS model and Stewart's HWMs at the Wolfe residence in Concrete, a different methodology (discussed in Section 2.3.3) from this method using the stage-discharge rating of the Concrete gage.

2.3.3 Estimates of 1909, 1917 and 1921 Flood Peak Discharges Using Backwater Stage-Discharge Rating of Baker River Gage in Conjunction with HEC-RAS Modeling and Stewart's HWMs at the Wolfe/McDaniels Residences in Concrete

During his field investigation in 1922-23, Stewart surveyed a 1917 HWM once and 1921 HWM twice at the old Wolfe Residence in Concrete, and a 1921 HWM at a gage near the old Concrete ferry site (see Figure 1 and Stewart's notes, pp. 18-19, 22-23, and 30-31). Stewart also surveyed a 1909 HWM, 1.27 ft above 1921 HWM, "east of Washington Cement Plant in still water of McDaniels residence" (Stewart notes, pp. 18-19), located across the Baker River from Wolfe residence (Figure 1 and Figure 12). PI Engineering used these Stewart-surveyed HWMs in conjunction with the use of the backwater stage-discharge rating of the nearby Baker River gage and the use of the HEC-RAS model and the 1911 channel geometry to modify the backwater stage-discharge rating as an alternative methodology to estimate the peak discharges of the 1909, 1917 and 1921 historical floods. Details of the backwater stage-discharge rating and HEC-RAS model development, and the discharge estimates are summarized below.

Backwater Stage-Discharge Rating of Baker River Gage and HEC-RAS Model to Modify the Rating for 1911 Channel Sections

- The Wolfe residence, the McDaniels residence, and the Baker River gage are located within the same Skagit River backwater area (Figure 12). The Baker River gage observes not only the Baker River flood peak flow and stage earlier in a flood event, but also the backwater stage later when the Skagit River flood peak arrives. When the Skagit River peaks (8 to 10 hours after the Baker River peaks), the concurrent flow contribution from the Baker River is insignificant. The Skagit River flood backwater stage observed at the Baker River gage is identical to the Skagit River flood backwater stage at the old Wolfe and McDaniels residences during the Skagit River flood peak hours. However, based on the USGS gage records, the flood stage observed at the Baker River gage due to the Baker River natural flow, usually creates a higher stage than the backwater stage during the Skagit River flood peak. The flood stage at the Wolfe and McDaniels residences (located about a quarter mile downstream of the Baker River gage) is always higher due to the backwater of the Skagit River flood peak than due to the Baker River natural flood flow.
- A backwater stage-discharge rating curve at the Baker River gage and the Wolfe/McDaniel residences can be readily established using the USGS published 2003 flood flows at the Concrete gage and the backwater stages at the Baker River gage, respectively. However, the backwater rating curve based on the 2003 flood event is representative of the current stream channel conditions, not the conditions existing

during the 1909, 1917, and 1921 events. Earlier data recorded at the Baker River gage were not sufficient to provide useful Skagit River backwater stage readings as the gage was at a different location upstream. To establish a reliable backwater stage-discharge rating curve for the purpose of using Stewart's HWMs at the Wolfe/McDaniels residences to estimate the 1909, 1917, and 1921 flood peak discharges, PI Engineering developed a HEC-RAS model. This model was calibrated and is capable of reproducing the rating curve for the current channel conditions. We then used the Corps surveyed 1911 Skagit River channel sections to modify the model and produce a modified backwater rating curve at the Baker River gage and the Wolfe/McDaniels residences for the early 1900's channel conditions.

- This HEC-RAS model was developed for a 2-mile reach of the Skagit River and 0.5-mile reach of the Baker River, from the USGS Skagit River gage (RM 54.15) near Concrete to upstream of the USGS Baker River gage at Concrete (#12-193500). The model incorporates ten new Skagit River cross sections surveyed in April 2008 by Skagit County, seven Skagit River channel sections surveyed in 2004 by PI Engineering, and remaining sections of the Skagit and Baker Rivers surveyed in 1977 for the original FEMA FIS study. Figure 12 shows locations of the model cross sections, the location of the Wolfe residence, the McDaniels residence, the Jenkins house, the gage site near the Old Ferry Crossing site, the Dalles, and the USGS gage sites.
- The HEC-RAS model was calibrated and verified for the 2003 flood HWMs observed at 1) the Baker River gage, 2) the Jenkins house (RM 56.18), and 3) at the old staff gage site (RM 54.19), using six discharges observed during flood peak hours between 150,956 and 165,655 cfs of the Skagit River (provided by USGS) and concurrent discharges between 4,647 and 4,822 cfs of the Baker River (provided by Puget Sound Energy). Table 12 shows the model calibration results.
- A comparison of the Corps 1911 surveyed channel and the 2008 channel sections indicates that the channel bottom has experienced scouring throughout the years. The base map used in Figure 1 is the Corps 1911 survey map showing locations of the surveyed low-flow channel sections of the Skagit River from the Baker River mouth to downstream of the Dalles. Also, the lower Baker River was dredged when the fish barrier dam at the current Baker River gage site was under construction in the late 1950's. Figure 13 shows the aerial views of the lower Baker River and the Little Baker side channel prior to and after the dredging. From the aerial views, it appears that the dredging extended to the Skagit River downstream to a point below the Little Baker channel mouth.

- After calibration of the model based on the 2008 surveyed Skagit River channel sections, the model was modified using the Corps 1911 surveyed low-flow channel sections. The Corps 1911 surveyed sections most closely represent the Skagit River channel bottom geometry present during the 1909, 1917 and 1921 flood events. Figure 14 shows the Skagit River backwater stage-discharge rating curves at the Baker River gage and the Wolfe/McDaniel residences, from the model runs for both 1911 and 2008 channel sections. There is only a slight increase (less than 3 percent) of the channel hydraulic capacity from 1911 to 2008 due to the channel scouring and dredging, based on a shift of the rating curve shown in Figure 14.

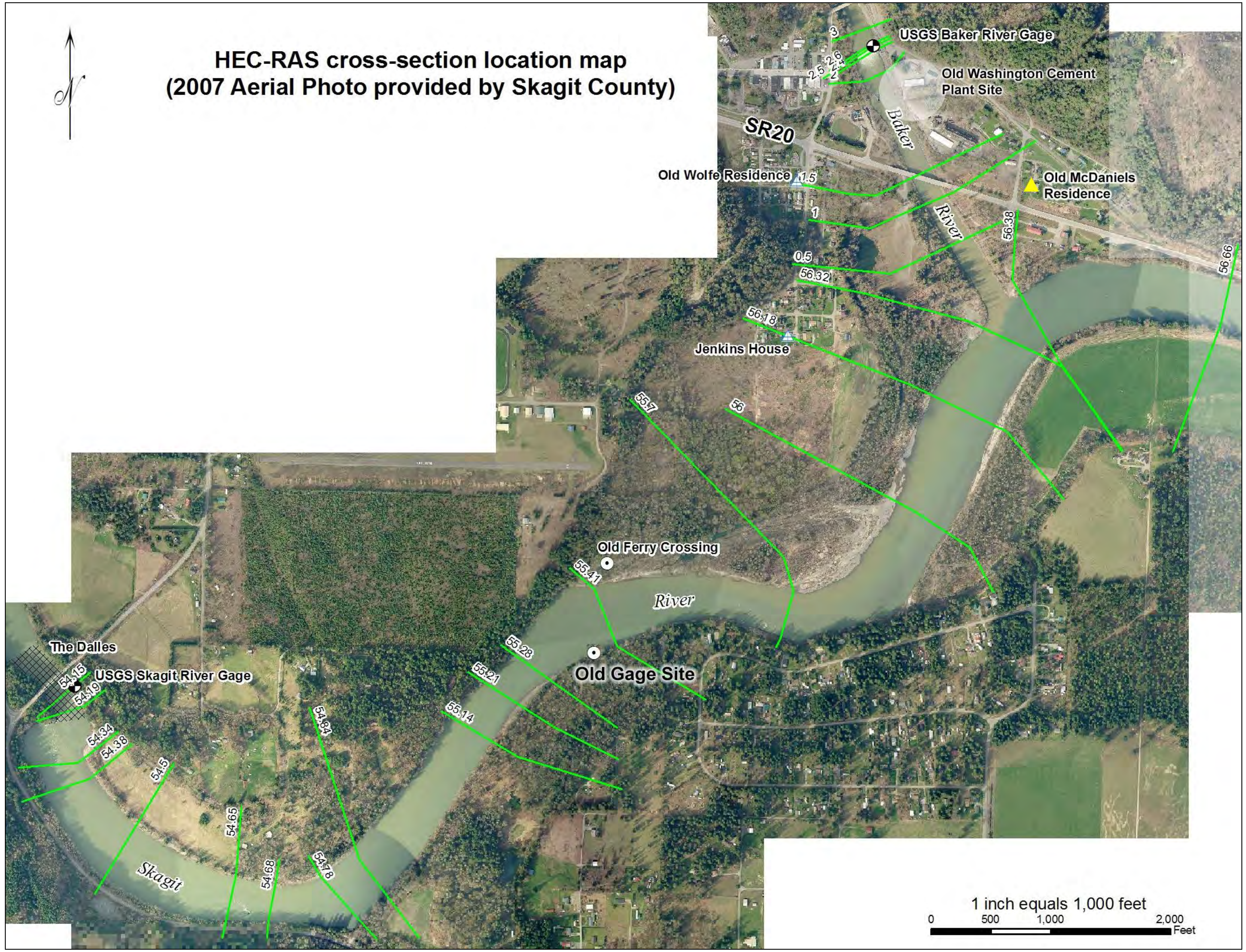

Figure 12. HEC-RAS cross-section location map for Concrete reach of the Skagit and Baker Rivers

Table 12. Comparison of Modeled and Observed 2003 Flood Elevations (NGVD-29) in Concrete

Date of Flood	Time	Skagit River Flow* (cfs)	Baker River Flow** (cfs)	High Water Mark Location	Source of Data	Flood Elevation (NGVD-29) Observed (ft)	Flood Elevation (NGVD-29) Modeled (ft)	Difference (ft) btw. Modeled and observed flood elev.
21-Oct-03	6:15 AM	165,655	4,647	Baker River gage	USGS gage record	183.49	183.70	0.21
21-Oct-03	6:30 AM	164,169	4,655	Baker River gage	USGS gage record	183.48	183.50	0.02
21-Oct-03	7:15 AM	162,602	4,710	Baker River gage	USGS gage record	183.32	183.29	-0.03
21-Oct-03	7:30 AM	162,342	4,747	Baker River gage	USGS gage record	183.22	183.25	0.03
21-Oct-03	9:30 AM	150,956	4,822	Baker River gage	USGS gage record	181.77	181.70	-0.07
21-Oct-03	9:45 AM	151,538	4,822	Baker River gage	USGS gage record	181.54	181.78	0.24
21-Oct-03	6:15 AM	165,655	4,647	Jenkins House	Resident provided photo	182.75	182.78	0.03
21-Oct-03	6:30 AM	164,169	4,655	Jenkins House	Resident provided photo	182.75	182.57	-0.18
21-Oct-03	9:30 AM	150,956	4,822	Jenkins House	Resident provided photo	181.15	180.74	-0.41
21-Oct-03	9:45 AM	151,538	4,822	Jenkins House	Resident provided photo	181.15	180.82	-0.33
21-Oct-03	6:15 AM	165,655	4,647	Old staff gage at the Dalles	USGS 2004 Survey	173.30	173.39	0.09
21-Oct-03	6:30 MA	164,169	4,655	Old staff gage at the Dalles	USGS 2004 Survey	173.30	173.21	-0.09

***USGS provided flow data (15-minute interval) at the Skagit River gage near Concrete**
****PSE provided hourly flow data (interpolated for 15-minute interval) below Lower Baker Dam and powerhouse**

Historical view of the Baker River and Little Baker side channel circa 1956, prior to dredging of the Baker River.

Baker River after dredging, circa 1967, showing the dried-up Little Baker former side channel.

Figure 13. Aerial views of the Baker River and Little Baker side channel prior to and after dredging of the Baker River (source of data: Little Baker River Side Channel Project, Skagit Fisheries Enhancement Group, Mount Vernon, WA)

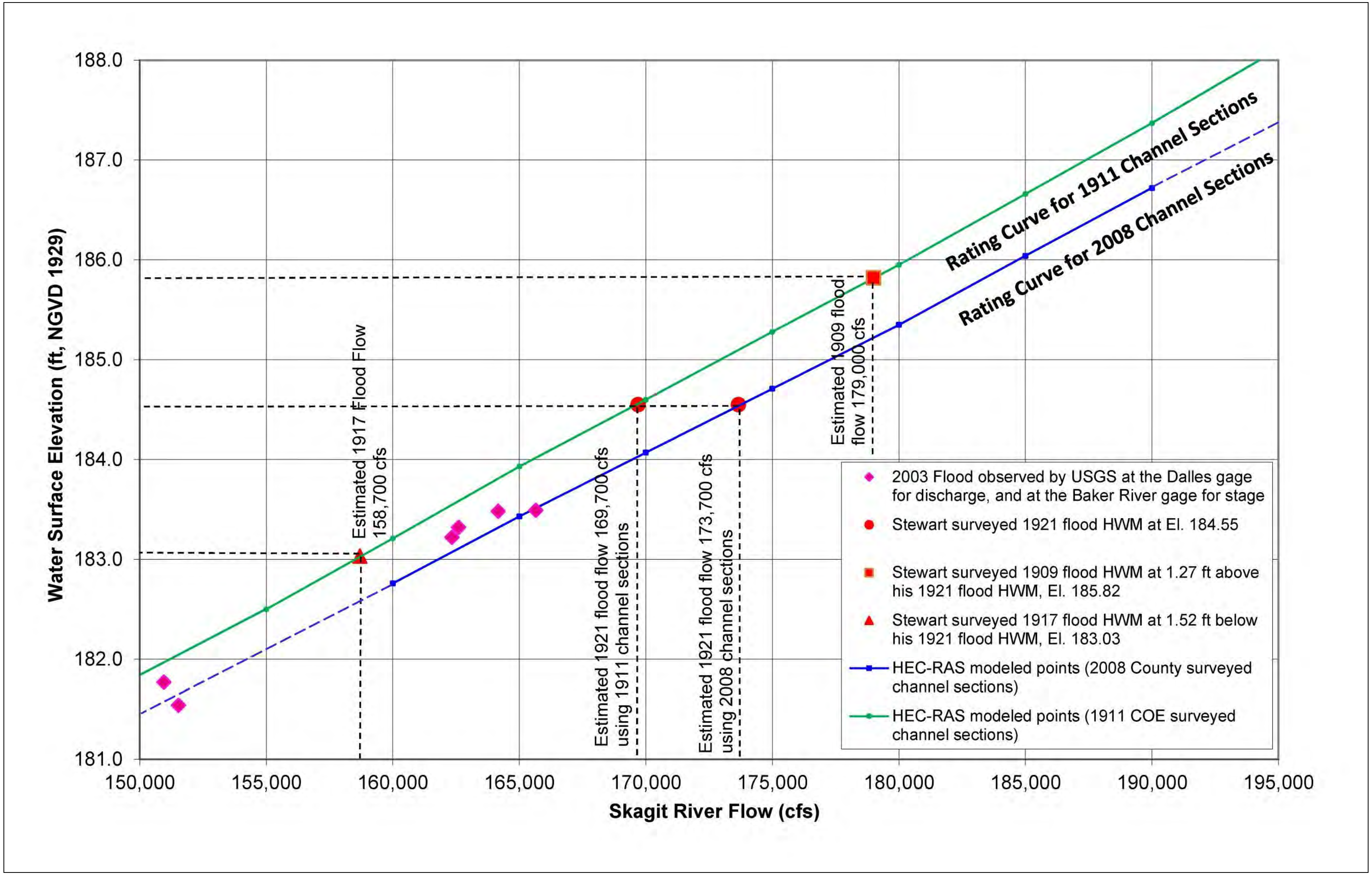

Figure 14. Skagit River backwater stage-discharge rating curves at Baker River gage and old Wolfe/McDaniels residences in Concrete

Estimates of 1909, 1917 and 1921 Flood Peak Discharges Based on the Backwater Stage-Discharge Rating at Baker River gage

- The Stewart-surveyed 1921 flood stage El. 184.55 at the Wolfe residence corresponds to a Skagit River peak discharge of 173,900 cfs using 2008 channel sections, and 169,700 cfs using 1911 channel sections. The difference is 4,000 cfs less (or -2.4 percent). Since the use of the 1911 channel sections is more representative of conditions in 1921, 169,700 cfs is the estimated 1921 flood peak discharge of the Skagit River (Figure 14).
- The Stewart-surveyed 1917 flood stage at the Wolfe residence is 1.52 ft below the 1921 flood stage, or at El. 183.03. From the flood stage discharge curve shown in Figure 14 using the 1911 channel sections, 158,700 cfs is the estimated 1917 flood peak discharge of the Skagit River.
- The Stewart surveyed 1909 flood stage at the McDaniels residence is 1.27 ft above the 1921 flood stage, or at El. 185.82. From the flood stage discharge curve shown in Figure 14 using the 1911 channel sections, 179,000 cfs is the estimated 1909 flood peak discharge of the Skagit River.
- As shown in Figure 14, the backwater stage-discharge rating curve produced by the HEC-RAS model for the 2008 channel sections matches well with the six observed 2003 flood data (see Table 12), also plotted and annotated in the figure. The rating curve is considered to be very accurate, as its accuracy is directly dependent on the accuracy of the observed data at the two USGS gages (the Concrete and Baker River gages), and would not be affected by any uncertainties associated with the flow conditions modeled at the Dalles, 2 miles downstream.
- The HEC-RAS model developed here provides the most reasonable methodology to produce the modified backwater stage-discharge relationship at the Baker River gage and the Wolfe/McDaniels residences for the stream channel conditions existing during the 1909, 1917, and 1921 flood events. Since the channel changes (due to scouring and dredging) resulted in only a small (less than 3 percent) hydraulic capacity increase between 1911 and 2008, the rating curve shown in Figure 14 for the 1911 channel sections is also considered to be very accurate. Surge issues are eliminated because the HWMs were identified in a backwater area where flow velocities were very low.
- Given a small increase in channel hydraulic capacity from 1911 to 2008, our estimated 1909, 1917, and 1921 flood peak discharges of 179,000, 158,700 and 169,700 cfs, based on Stewart's surveyed HWMs at El. 185.82, 183.03, and 184.55 at the Wolfe/McDaniels

residences, correlate well with the 2003 flood peak discharge of 165,655 cfs observed at the Dalles gage and the 2003 flood stage El. 183.49 observed at the Baker River gage as shown in Figure 14.

- Figure 15 presents the flood profiles from the HEC-RAS model results for the 2003 flood calibration and the two estimated 1921 flood peak discharges of 169,700 cfs and 173,700 cfs based on the 1911 and 2008 channel sections, respectively. Modeled energy gradient lines for the two estimated 1921 flood peak discharges were also plotted. The selected estimate of the 1921 flood peak discharge is 169,700 cfs based on the Corps 1911 surveyed channel sections. The Stewart-surveyed 1921 HWM 182.58 at the old ferry crossing is also plotted on the figure, comparing well with the modeled flood profiles and confirming that the estimated 1921 flood peak discharge is 169,700 cfs, using this HWM and the modeling method.

- It is our opinion that the methodology using the backwater stage-discharge rating of the Baker River gage in conjunction with the HEC-RAS model and Stewart's HWMs at the Wolfe/McDaniels residences in Concrete is superior to the methodologies employed for the slope-area study below the Dalles and for the stage-discharge rating at the Dalles gage, due to the uncertainty of the data used in the slope sections and the Dalles, compared to the accurate backwater stage-discharge rating curve at the Baker River gage. This backwater stage-discharge rating methodology produced the estimates of three historical flood peak discharges (1909, 1917 and 1921) directly using Stewart's HWMs in this backwater area, instead of using HWMs transferred from other location for the other two methodologies.

- It is our opinion that the estimated 1921 flood peak discharge, 169,700 cfs, using this modeling method is more accurate than those either originally estimated by Stewart in 1923 or later revised by the USGS in 2007 (see Table 4). This estimated discharge is also consistent with 1921 HWMs or water depths observed, stated, or reported by all parties including Stewart, Skagit County, Great Northern Railroad (GNRR), and local newspaper articles in the Concrete–Hamilton area.

- In our opinion, the use of Stewart's surveyed 1909 and 1917 HWMs at the Wolfe/McDaniels residences, in conjunction with the use of the backwater stage-discharge rating curve at the Baker River gage for 1911 channel shown in Figure 14, is the best methodology to estimate the 1909 and 1917 flood peak discharge. The estimated 1909 and 1917 flood peak discharges of 179,000 and 158,700 cfs are more accurate than those either originally estimated by Stewart in 1923 or later revised by the USGS in 2007 (see Table 4).

Un-reliable Estimate of 1897 Peak Discharge Using Questionable Flood Marks Upstream of The Dalles

- There is 2-ft differential gage height between the 1897 and 1909 floods listed in Table 4 (either as published by USGS in 1961, 51.1-49.1=2.0, or estimated by Stewart in 1923, 38.4-36.4=2.0). This flood differential height was transferred by Stewart from Stewart's flood marks on a hotel located over 2 miles upstream on east bank of the Baker River.
- WSP 1527 (USGS, 1961) and Stewart's 1923 report (Stewart, 1923) referenced two 1897 HWMs transferred to a hotel footing. The first 1897 HWM was found "on a barn on the right bank about a mile upstream from Concrete, was transferred by levels to the footing of a hotel in Concrete on which the other flood mark had been made in 1909." Later, a second HWM was found on a stump that was reported by Magnus Miller to be "1.5 feet out of the water during the flood of 1897." For the 1909 flood, WSP 1527 states that Stewart measured a flood mark on "a hotel near the cement plant [that] was just reached by the water." No information is given on the exact location of the hotel. Exact locations are not given for either the barn or the stump. It is not known if, like the 1909 HWM on the hotel, these two 1897 HWMs represented flood peak elevations on the Baker River, or if they represented flood peaks on the Skagit River. For these two 1897 flood marks, WSP 1527 states at the end that "the flood elevations in Concrete probably were affected to a considerable extent by the flow of Baker River. The relationship between the two floods [1897 and 1909] at that point may have been quite different from the relationship at the [Concrete] gaging station site."
- Assuming this 2-ft differential height between the 1897 and 1909 flood marks was also representative of the Skagit River backwater flooding, we would obtain an 1897 backwater stage at El. 187.82 (=2.0+185.82, the 1909 HWM at the McDaniels residence). The estimate of the 1897 flood peak discharge would be 193,300 cfs, using the backwater stage-discharge rating curve for 1911 channel shown in Figure 14. Even this estimate, 193,300 cfs, is much lower than the current USGS published 265,000 cfs listed in Table 4 for the 1897 flood.
- It is our opinion that the estimate of the 1897 flood peak discharge based on the 1897 HWMs upstream of the Dalles is not reliable, giving the uncertainties associated with the 1897 HWMs that include transferring the HWMs from two questionable locations to an unknown hotel footing location, and Stewart's interview with Leonard Everett (Stewart's Notes, p. 23) stating that the "log jam in the Dalles raised water 10 ft in 2 hrs" during the 1897 flood.

Unsupported Relationship of 1909, 1917, and 1921 HWMs Used in Stewart and USGS Peak Discharge Estimates

- Stewart-surveyed 1909, 1917 and 1921 HWMs at the Wolfe and McDaniels residences have the differential heights of 1.27 ft between the 1909 and 1921 HWMs, 1.52 between the 1921 and 1917 HWMs, and 2.79 ft between the 1909 and 1917 HWMs. The estimated peak discharges are 179,000, 158,700 and 169,700 cfs for the 1909, 1917 and 1921 historical floods, as discussed above, using the backwater stage-discharge rating curve at the Baker River gage for 1911 channel sections shown in Figure 14 These provide the differential peak discharges of 9,300 cfs between the 1909 and 1921 estimates, 11,000 cfs between the 1921 and 1917 estimates, and 20,300 cfs between the 1909 and 1917 estimates. These differential peak discharges are supported by the same relationship of the estimated peak discharges using Stewart's HWMs at the Savage Ranch discussed next in Section 2.3.4.
- The differential heights calculated from the gage heights published by USGS (Table 4) and used in Stewart and USGS peak discharge estimates are 1.50 ft between the 1909 and 1921 HWMs, 1.90 between the 1921 and 1917 HWMs, and 3.40 ft between the 1909 and 1917 HWMs. The differential peak discharges are calculated, using the USGS 2007 revised estimates listed in Table 4, to be 17,000 cfs between the 1909 and 1921 estimates, 18,000 cfs between the 1921 and 1917 estimates, and 35,000 cfs between the 1909 and 1917 estimates. These differential peak discharges are higher by an average of 72 percent (ranging 64 to 83 percent) than those estimates above based on Stewart's HWMs at the Wolfe and McDaniels residences.
- The HWM relationship of these published gage heights is not supported by Stewart's surveyed HWMs at the Wolfe and McDaniels residences. This un-supported HWM relationship provides exaggerated differential peak discharges, and is not appropriate for use to estimate the historical flood discharges.

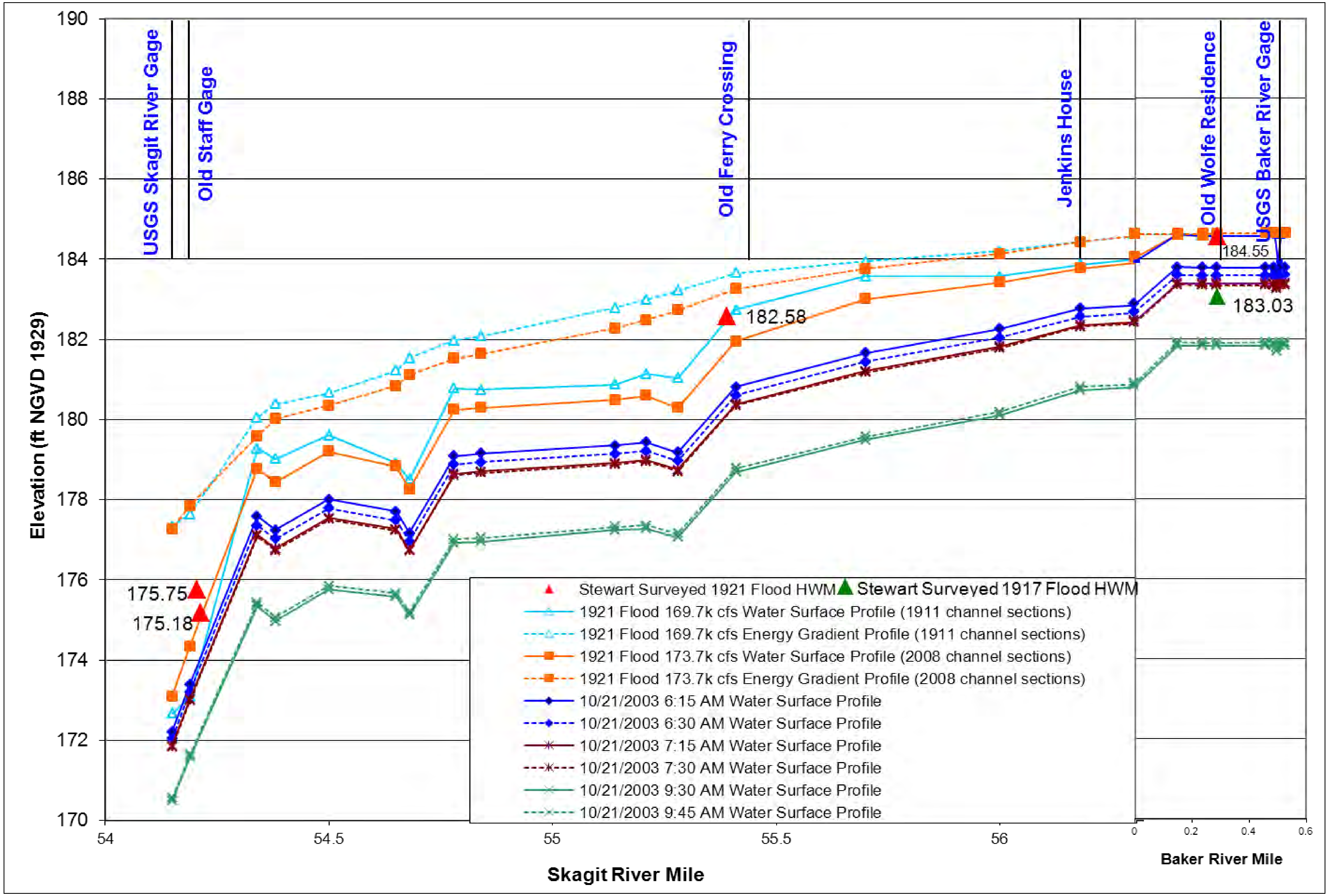

Figure 15. HEC-RAS modeled flood profiles in Concrete reach of the Skagit and Baker Rivers

2.3.4 Estimates of 1897, 1909 and 1917 Flood Peak Discharges Using HEC-RAS Modeling and Stewart's Flood Marks Downstream of Concrete

As an alternative methodology, PI Engineering used the HEC-RAS model and Stewart's surveyed HWMs downstream of the Dalles to estimate the 1897, 1909 and 1917 flood peak discharges, as summarized below.

- In the town of Hamilton, approximately at RM 39.9, Stewart found a 1917 HWM 0.55 feet below a 1909 HWM and 0.84 feet below a 1921 HWM (Stewart' notes, pp. 13-14, Stewart 1922-23). These HWMs surveyed by Stewart are at El. 96.17, 95.62, and 96.46, respectively. Stewart-surveyed HWMs were located at the A.J. Jacobin cigar store building which no longer exists today. The old Jacobin cigar store was located approximately 200 yards west of the Smith house on the same street, where Skagit County has recent 1995 and 2003 flood elevations surveyed (see Figure 2 and Section 2.1.3). A comparison of the 1995 and 2003 flood stages at the Smith house and the 1909, 1917, and 1921 flood stages at the old Jacobin cigar store building (Table 3) appear to indicate that the peak discharges of the recent two floods are greater than the peak discharges of the three historical floods. However, due to the complexity of flood hydraulics on the Hamilton floodplain and the historical migration of the Skagit River channel alignment in the vicinity, it is difficult to use Stewart-surveyed HWMs at the old Jacobin cigar store building to estimate the flood peak discharges with certainty.

- At the Kemmerick Ranch (about RM 44.5), Stewart found HWMs that showed the 1897 peak was about the same as the 1909 peak and 0.78 feet above the 1921 peak (Stewart's notes, pp. 26-27). At the Savage Ranch, across from the Old Birdsview School (about RM 45.2), Stewart's notes show the 1909 flood to be 0.51 and 0.67 feet higher than the 1921 flood and the 1917 flood to be 0.68 feet below the 1921 flood (Stewart's notes, pp. 26-27). Stewart did not survey to tie these HWMs to any known benchmark. However, the differential heights of these HWMs provide a reasonable basis to estimate the differential quantities of the 1897, 1909 and 1917 peak discharges in relation to the 1921 peak discharge estimated previously using Stewart surveyed HWMs at the Wolfe residence in Concrete.

- Figure 16 presents the flood stage-discharge rating curves at the Kemmerick and Savage Ranches near Birdsview, approximately at RM 44.5 and 45.2, respectively. These curves were plotted for the Skagit River flood peak discharges between 160,000 and 190,000 cfs from results of an unsteady HEC-RAS model originally developed by the USACE, Seattle District and improved by PI Engineering for the Skagit River Basin.

- Using Stewart-surveyed difference of 0.78 ft between the 1897 and 1921 HWMs at the Kemmerick Ranch, the 1897 flood peak discharge was estimated to be 181,200 cfs. It is noted that the 1897 flood may well have been a debris-blockage event as noted by Stewart (Stewart notes, p. 23), in which case this estimate of the 1897 flood peak discharge would be too high.
- Using Stewart-surveyed difference of 0.59 ft (the average of 0.51 and 0.67 ft) between the 1909 and 1921 HWMs at the Savage Ranch, the 1909 flood peak discharge was estimated to be 179,000 cfs. This estimate, 179,000 cfs, is identical to the estimate using Stewart's surveyed 1909 HWM at the McDaniels residence in conjunction with the use of the backwater stage-discharge rating curve at the Baker River gage for the 1911 channel sections as discussed in Section 2.3.3.
- Using Stewart-surveyed difference of -0.68 ft between the 1917 and 1921 HWMs at the Savage Ranch, the 1917 peak discharge was estimated to be 159,200 cfs. This estimate, 159,200 cfs, is very close (with a difference of 0.3 percent) to 158,700 cfs estimated using Stewart's surveyed 1917 HWM at the Wolfe residence in conjunction with the use of the backwater stage-discharge rating curve at the Baker River gage for the 1911 channel sections as discussed in Section 2.3.3.

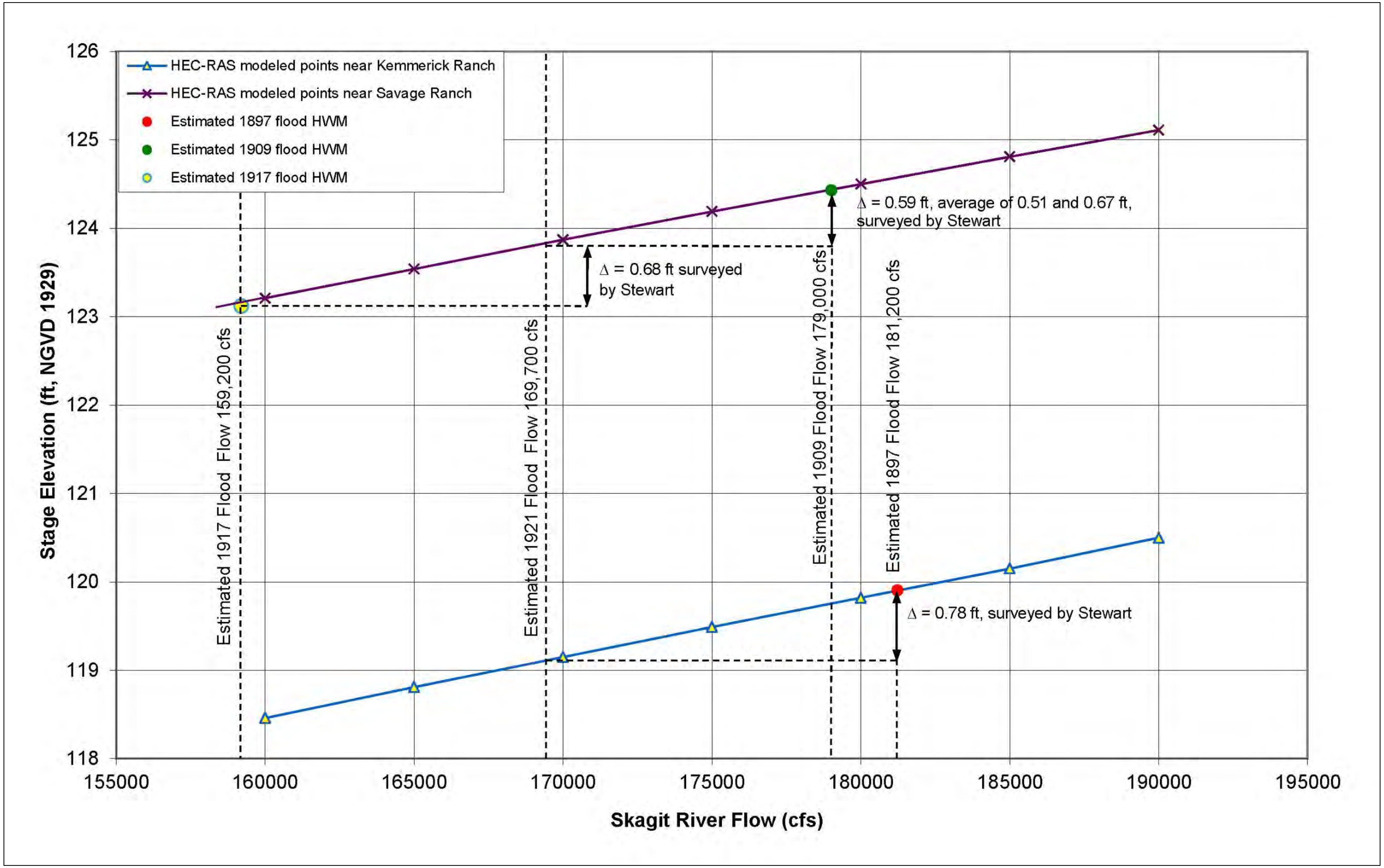

Figure 16. Skagit River flood stage-discharge curves at Kemmerick and Savage Ranches near Birdsview

2.3.5 Conclusion from Reevaluation of 1897, 1909, 1917 and 1921 Historical Flood Peak Discharges

It is our conclusion that the 1897, 1909, 1917, and 1921 historical flood peak discharges estimated by PI Engineering using Stewart's surveyed HWMs in the Concrete to Hamilton area in conjunction with the HEC-RAS model to provide more rigorous methodologies than the slope-area method, as discussed above and shown in Table 13, are most representative of the documented conditions at that time. We believe the methodologies and results summarized above are more accurate than those either originally estimated by Stewart in 1923 or later revised by the USGS in 2007, also listed in Table 13.

Table 13. Historical flood peak discharges of Skagit River near Concrete

	Peak Discharge (cfs)		
	1923 Estimated by Stewart	2007 Revised by USGS	2011 Re-evaluated by PI Engineering
Flood	Slope-Area Method	Slope-Area Method	Various HEC-RAS Modeling
1897	275,000	265,000	181,200
1909	260,000	245,000	179,000
1917	220,000	210,000	158,700
1921	240,000	228,000	169,700

2.4 Unregulated Peak Discharge for the 1932 Flood

The RFIS flood frequency analysis uses an unregulated peak discharge for the 1932 flood, based on an estimate made by USGS and published in the Water Supply Paper 1527 (USGS 1961). Figure 17 is the figure from Water Supply Paper 1527 (USGS, 1961, Figure 4) showing the USGS' estimate. The USGS estimated unregulated peak discharge is 182,000 cfs, which is used by the Corps of Engineers in the RFIS hydrology analysis (USACE 2008). We believe this estimate is incorrect, as discussed below.

As shown in Figure 17, USGS estimated the effects of Diablo and Lake Shannon storage to be 26,400 and 35,100 cfs, respectively, on reduction of the peak flows occurring at the same time, approximately 4 to 5 AM on February 27, 1932. Adding these to the regulated 120,500 cfs observed at the Skagit River gage near Concrete at that time, results in the USGS estimated unregulated peak discharge of 182,000 cfs. The regulated peak flow actually recorded and published by the USGS at the gage is 147,000 cfs at about 10 PM on February 27, 1932.

The USGS estimated effects on the peak flow reduction at each storage location are probably reasonable. It is also reasonable that the storage effects likely occurred approximately at the same time (4 to 5 AM on February 27, 1932) at Diablo and Lake Shannon sites. But it is incorrect to directly cumulate and transform these effects to the Skagit River flood hydrograph observed at the gage near Concrete, ignoring flow travel time difference between these two storage sites to Concrete. The travel time from Diablo reservoir to Concrete is about nine hours (an approximate distance of 46 stream miles), and from Lake Shannon to Concrete is less than one hour (an approximate distance of 3 stream miles). The travel time difference between these two reservoirs is significant, more than 8 hours, and should not be ignored in estimating the unregulated peak discharge. The USGS estimated unregulated flow apparently ignored this travel time difference.

In the figure below, taken from WSP 1527, the USGS unregulated peak discharge (182,000 cfs) at the Skagit River gage near Concrete occurred at the same time (4 to 5 AM on February 27, 1932) as the unregulated peak discharge (35,100 cfs) at Lake Shannon. This peak time coincidence is incorrect; none of the flood events in the past occurred this way. Floods similar to the 1932 event typically peak at Lake Shannon 8 to 10 hours before the Skagit River peaks at Concrete. Therefore, if the USGS unregulated peak time at Lake Shannon is accurate, the Skagit River unregulated flood peak time would be around 1 PM (not 4 to 5 AM on February 27, 1932 as shown by the USGS unregulated hydrograph plotted in Figure 17). The Skagit River peak as recorded by the Concrete gage, 2 miles downstream from the mouth of the Baker River, was at about 10:00 p.m. on February 27th.

PI Engineering revised the unregulated discharge with a 9-hour travel time adjustment to the flow unregulated by USGS for the Diablo storage effects. Our revised estimate is 165,000 cfs as annotated in Figure 15, and described below.

The USGS unregulated peak of 26,400 cfs from Diablo reservoir as shown in Figure 17, was delayed by 9 hours to arrive at the gage near Concrete approximately at 1 PM on February 27, 1932. Adding this flow to 138,600 cfs scaled from the figure, which is the sum of the USGS unregulated flow from Lake Shannon and the flow observed at the gage near Concrete at 1 PM on February 27, 1932, results in the unregulated peak flow estimate of 165,000 cfs. Lack of details and data used in the USGS unregulated flow estimate prevents us from performing a complete hydrograph unregulation.

It is our opinion that our revised unregulated peak discharge of 165,000 cfs is more accurate than the USGS estimated 182,000 cfs for the 1932 flood.

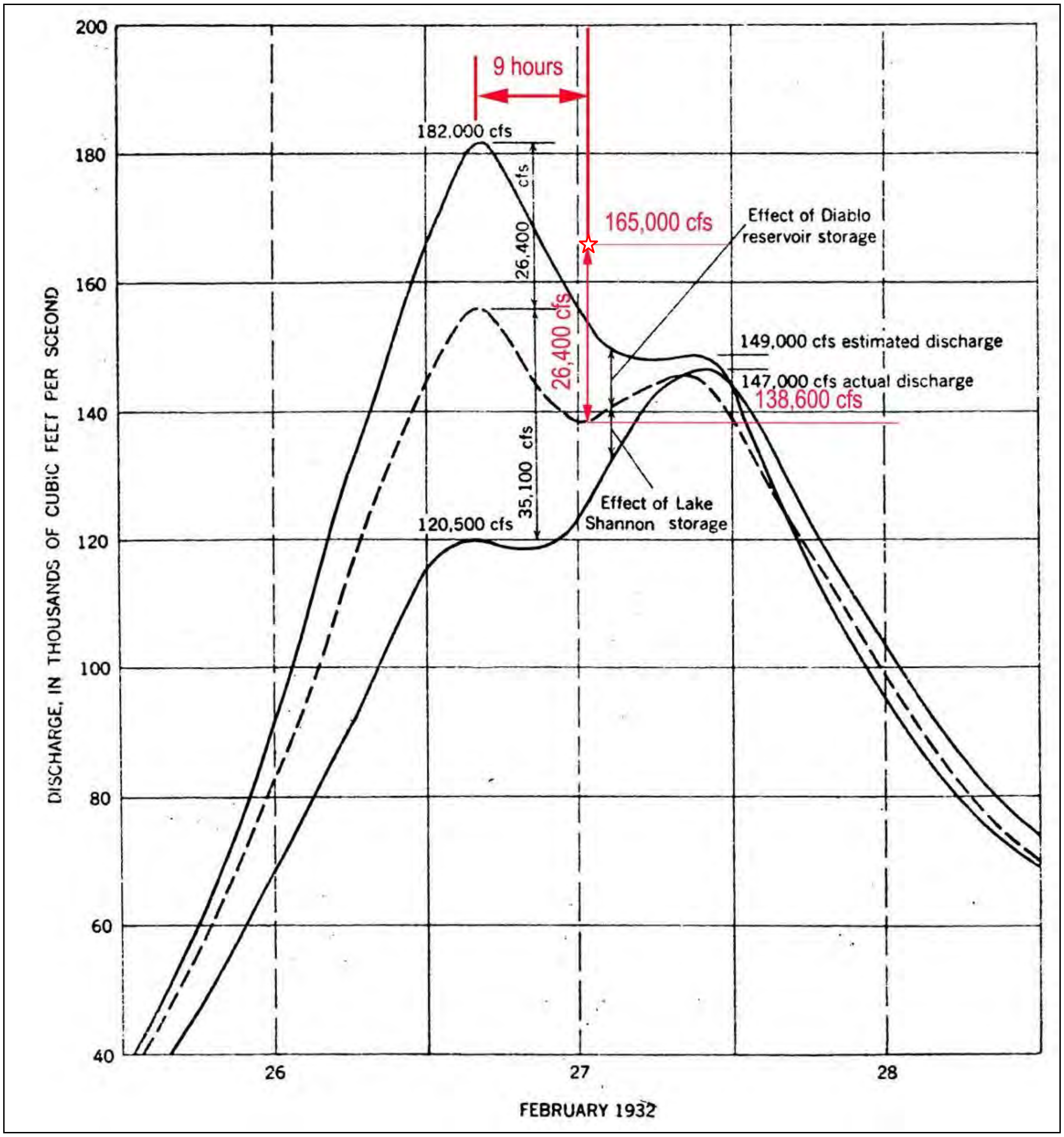

Figure 17. USGS published estimate of 1932 unregulated flood peak discharge at Concrete (source of data: Figure 4 – Hydrographs showing effect of dams on flood peak, USGS 1961)

2.5 Flood Frequency Analysis for Unregulated Flows at Concrete

This section presents the results of a flood frequency analysis for unregulated flows at Concrete, using our revised flood estimates discussed above in Sections 2.3 and 2.4. Details of this frequency analysis are summarized below.

The USGS-developed, FEMA-approved, computer program "PEAKFQ, Annual Flood Frequency Analysis following Bulletin 17B Guidelines" (version 5.0, May 6, 2005) was used for performing the Skagit River flood frequency analysis (USGS 1998). In accordance with the FEMA guidelines (Section c.2.1) (FEMA

2003), the Skagit River flood frequency curves for this analysis were developed for unregulated conditions, and subsequently converted to regulated conditions using the current reservoir operation criteria.

The unregulated flow data considered for the Skagit River flood frequency analysis include 84 systematic peaks for water years (WY)1925 through 2008, and 4 historical peaks for 1897, 1909, 1917, and 1921 (WY 1898, 1910, 1918, and 1922).

2.5.1 Unregulated Systematic Flow Data (WY 1925–2008) in the Skagit River at Concrete

Table 14 presents the systematic annual peak and one-day discharge data observed at the USGS gage 12194000 – Skagit River near Concrete for WY 1925–2008. Also included in the table are unregulated flows estimated mostly by the Corps with some estimated by PI Engineering (noted). Discussion of the source and any adjustments made to the data are provided below.

Table 14 Annual peak and one-day discharge data at the USGS Gage 12194000 - Skagit River near Concrete

Water Year	USGS Observed Annual Peak Flows (cfs)	Winter Unregulated Annual Peak Flows	USGS Observed Winter One-Day Flows	Winter Unregulated One-Day Flows
1925	92,500	100,721	85,400	85,400
1926	51,600	48,591	42,100	41,200
1927	88,900	66,754	56,700	56,600
1928	95,500	94,812	81,200	80,390
1929	74,300	83,631	62,200	70,910
1930	*32,200	41,937	29,200	35,558
1931	*60,600	58,770**	48,900	48,900**
1932	147,000	165,000**	129,000	151,945**
1933	116,000	115,519	97,800	97,947
1934	101,000	97,733	85,000	82,867
1935	131,000	143,702	120,000	121,843
1936	*60,000	18,000	14,300	14,480
1937	*68,300	25,767**	21,500	21,500**
1938	89,600	88,484	63,500	75,025
1939	*79,600	64,203	55,200	54,437
1940	48,200	45,280	38,900	38,392
1941	51,000	46,471	42,200	39,402

Water Year	USGS Observed Annual Peak Flows (cfs)	Winter Unregulated Annual Peak Flows	USGS Observed Winter One-Day Flows	Winter Unregulated One-Day Flows
1942	76,300	67,515	56,100	57,245
1943	54,000	55,529	45,000	47,082
1944	65,200	61,643	49,000	52,266
1945	70,800	64,412	61,200	54,614
1946	102,000	108,451	87,500	91,954
1947	82,200	77,377	62,000	65,607
1948	95,200	81,409	69,000	69,026
1949	*55,700	36,127	52,100	30,632
1950	154,000	170,342	123,000	144,431
1951	139,000	157,098	128,000	133,202
1952	*43,500	32,094	36,700	27,212
1953	66,000	75,243	60,700	63,798
1954	58,000	54,313	46,900	46,051
1955	*56,300	56,676	51,200	48,055
1956	106,000	125,871	94,100	106,725
1957	61,000	60,813	49,700	51,563
1958	41,400	40,293	34,600	34,164
1959	*90,700	79,089	58,200	67,059
1960	89,300	99,673	77,500	84,512
1961	79,000	89,468	60,300	75,859
1962	56,000	68,720	48,900	58,267
1963	114,000	106,674	81,700	90,448
1964	73,800	78,105	58,600	66,224
1965	52,600	58,788	49,500	49,846
1966	*36,800	35,738	29,000	30,302
1967	*72,300	78,247	53,900	66,345
1968	84,200	83,101	60,200	70,460
1969	49,500	59,240	44,100	50,229
1970	38,400	34,032	29,000	28,855
1971	62,200	79,312	54,700	67,248
1972	*91,900	57,099	40,400	48,414
1973	49,500	50,781	43,100	43,057
1974	79,900	123,434	73,400	104,658

Water Year	USGS Observed Annual Peak Flows (cfs)	Winter Unregulated Annual Peak Flows	USGS Observed Winter One-Day Flows	Winter Unregulated One-Day Flows
1975	57,500	57,427	42,500	48,692
1976	122,000	155,281	108,200	131,661
1977	58,400	65,441	45,800	55,487
1978	70,300	69,589	57,800	59,004
1979	46,000	52,015	35,300	44,103
1980	135,800	149,079	113,700	126,402
1981	148,700	170,470	104,900	144,540
1982	*51,700	61,885	49,000	52,472
1983	101,000	79,992	61,500	67,824
1984	109,000	111,556	79,600	94,587
1985	*46,100	32,515	23,900	27,569
1986	93,400	103,347	70,100	87,627
1987	83,500	74,104	60,300	62,832
1988	39,600	35,801	29,000	30,355
1989	74,100	86,250	55,900	73,130
1990	119,000	141,277	86,100	119,787
1991	149,000	199,017	135,000	172,979
1992	*53,300	47,389**	35,300	39,459**
1993	*39,300	31,490**	25,300	26,257**
1994	36,500	50,609	31,400	42,911
1995	59,800	74,313	51,800	63,009
1996	160,000	187,982	131,000	156,645
1997	*91,400	103,692	63,000	87,919
1998	76,700	70,049	61,400	59,394
1999	61,400	76,869	45,100	65,176
2000	103,000	138,206	86,000	117,183
2001	30,900	33,277	22,800	28,215
2002	94,300	127,137	79,700	107,798
2003	65,500	72,461	43,200	61,439
2004	166,000	205,651	131,000	171,364
2005	99,400	111,118	74,700	94,216
2006	56,300	66,893	47,700	56,718
2007	145,000	173,974	118,000	153,886

Water Year	USGS Observed Annual Peak Flows (cfs)	Winter Unregulated Annual Peak Flows	USGS Observed Winter One-Day Flows	Winter Unregulated One-Day Flows
2008	77,900	106,503	72,400	88,439**

* Non-winter event
** Estimated by PI Engineering

Unregulated Flow Data for WY 1944–2007 Estimated by the Corps

A synthetic record of the mean daily unregulated discharge in the Skagit River at the Concrete gaging site was constructed by the Corps for the period including water years 1944 through 2007 (excluding WY 1992 and 1993). The Corps constructed this record by adjusting the observed mean daily flows to include estimated effects of the regulation operations occurring at the three Seattle City Light (SCL) dams on the Upper Skagit and two Puget Sound Energy (PSE) dams on the Baker River. The unregulated annual winter peak one-day flows in the Skagit River at Concrete for these water years were selected from the mean daily unregulated discharges estimated by the Corps.

The Corps also developed the unregulated annual peak flows for this period based on a regression of the winter peak to one-day flows from water years 1925 through 1953 for the Skagit River near Concrete. The Corps assumed that the regression closely mimicked unregulated basin conditions, as no storage occurred at any of the dams for flood control during this time period. Details of the Corps-developed unregulated annual peak and one-day discharges are documented in the Corps' "Draft Report – Skagit River Basin, Washington, Revised Flood Insurance Study, Hydrology Summary" (USACE 2008).

Unregulated Flow Data for WY 1925–1943 Estimated by the Corps

The period of record of stream flow data at the USGS gage 12194000 – Skagit River near Concrete – includes the period 1924 to present. Data collected at this gage includes the effects of regulation at upstream reservoirs. Flow data measured by USGS at the Concrete gage during the period between 1924 and 1943 comprised lower annual flood peaks, in general, than the flood peaks measured outside of this period. Prior to 1943, two dams were in operation in the Skagit watershed, Lower Baker Dam and Diablo Dam. (Construction of Ross Dam was completed in 1949, and regulation of Ross Dam for winter flood control storage was initiated in 1954). Prior to 1943, construction and operation of Lower Baker Dam and Diablo Dam had only incidental regulation effects on the flood flows in the Skagit River.

Diablo Dam – Construction of the dam was completed in 1930, and the power plant began operation in 1936. The dam has never been operated for flood control purposes. During construction, all flows were routed through construction bypass tunnels with no provision for storage during the fall and winter periods.

Lower Baker Dam – Construction of Lower Baker Dam was completed in 1925. Operation of Lake Shannon, the reservoir created by Lower Baker Dam, for flood control has never been part of the purpose of the dam. Hydrologically, storms arrive at the Baker system early in the event and the peak flood outflow from the Baker River passes the Concrete gage about 8 to 10 hours in advance of the peak flow coming from the Skagit River upstream of the Concrete gage.

The USACE in 1965 performed calculations of the one-day maximum flows and reservoir storage changes to unregulate the observed annual winter maximum one-day discharges at the Skagit River gage near Concrete. These unregulated one-day flow discharges estimated by the USACE include data for WY 1925 through 1943 (excluding WY 1931 and 1937). By applying the unregulated peak to one-day flow correlation, the corresponding annual winter peak discharges for this period were estimated by the USACE.

<u>*Unregulated Flow Data Estimated by PI Engineering*</u>

PI Engineering estimated the winter unregulated one-day flows for WY 1931, 1937, 1992, 1993, and 2008 by adjusting the USGS observed one-day flows with the regression of regulated and unregulated flows developed by the USACE. The annual peak discharges for these five water years were estimated by using the peak to one-day flow regression developed by the USACE. For those water years when the annual peak flows observed by USGS were non-winter events, USGS-observed one-day flow data were used and the corresponding winter peak flows were estimated by using the same peak to one-day flow regression discussed above.

PI Engineering also modified the unregulated peak flow for WY 1932 as discussed above in Section 2.4.

2.5.2 Unregulated Four Historical Flood Data

The four historical flood peak discharges used for the frequency analysis are based on PI Engineering reevaluated estimates listed in Table 15 (same as listed in Table 13). PI Engineering estimated the one-day flows for these four historical events, also listed in Table 15. Discussion of our one-day flow estimates follows.

Table 15. Estimated unregulated peak and one-day discharges for four historical floods in the Skagit River near Concrete

Water Year	Date	Peak Discharges (cfs) Estimated by PI Engineering	One-Day Discharges (cfs) Estimated by Regression
1898	Nov. 19, 1897	181,200	148,300
1910	Nov. 30, 1909	179,000	146,500
1918	Dec. 30, 1917	158,700	130,200
1922	Dec. 13, 1921	169,700	139,100

The one-day flows represent the most critical flood volumes determining the lower Skagit River floodplain flooding conditions after routing through dams and floodplain storages in the Skagit River system. The winter unregulated one-day flow data for water years 1925 through 2008 are provided in Table 14.

The four historical floods estimated by Stewart and the USGS, as well as by PI Engineering, have only the unregulated peak discharges estimated. To estimate the corresponding unregulated one-day discharges for these four events, a regression of selected flood events developed by the USACE was applied. Figure 18 below shows the USACE' unregulated floods and the regression curve.

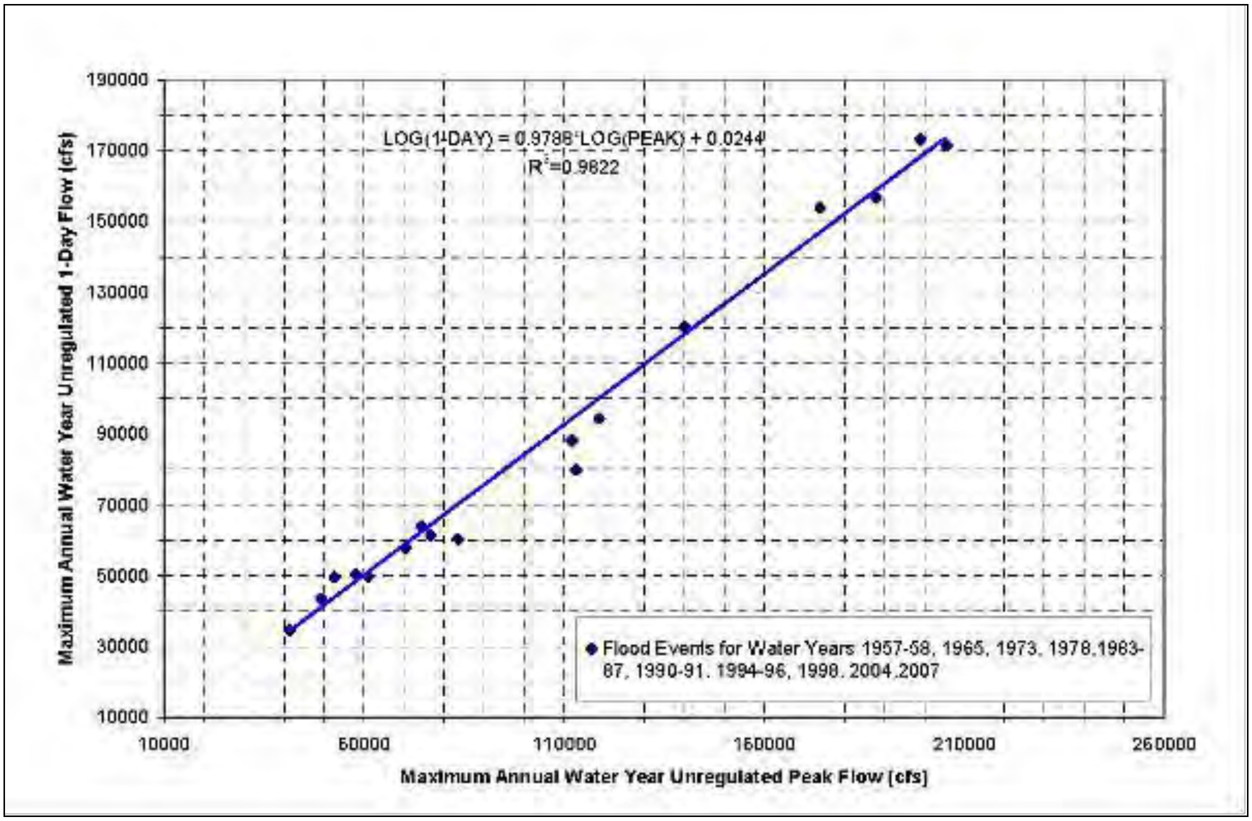

Figure 18. Regression curve of peak to one-day flow for the flood events unregulated by the USACE

2.5.3 Flood Frequency analysis for Unregulated Flows in the Skagit River near Concrete

A flood frequency analysis for unregulated peak and one-day flows in the Skagit River near Concrete was performed, using PEAKFQ software (USGS 2005). The unregulated peak flow frequency curve and the confidence band from the result of the PEAKFQ run using 84 water years of systematic data (Table 14) and our estimated four historical events (Table 15) are shown on Figure 19. The unregulated peak flows at Concrete would have values of 146,800, 212,100, 240,800, and 309,500 cfs, for the 10-, 50-, 100- , and 500-year floods, respectively. For a full description of the calculated statistics see the PEAKFQ model runs on the attached CD (see Section 2.7).

The unregulated one-day flow frequency curve and the confidence band, together with all data used in the frequency analysis, are plotted in Figure 20. The unregulated one-day flows at Concrete would have values of 123,700, 177,900, 201,400, and 257,500 cfs, for the 10-, 50-, 100-, and 500-year floods, respectively. For a full description of the calculated statistics see the PEAKFQ model runs on the attached CD (see Section 2.7).

The appropriateness of using the four historical floods in the frequency calculation was checked in accordance with federal procedures described in Bulletin 17B (IACWD 1982 pg. 19). The historical floods as calculated by PI Engineering were shown to be consistent with the systematic record and therefore should be used in establishing the Flow Frequency Curve (Countryman 2011). A similar check of the USGS estimated historical flows was made. It was determined that the historical floods estimated by USGS were not indicative of the extended record (too extreme) when compared to the systematic record and that the USGS-estimated historical floods should not be used in the establishment of the frequency curves (Countryman 2011).

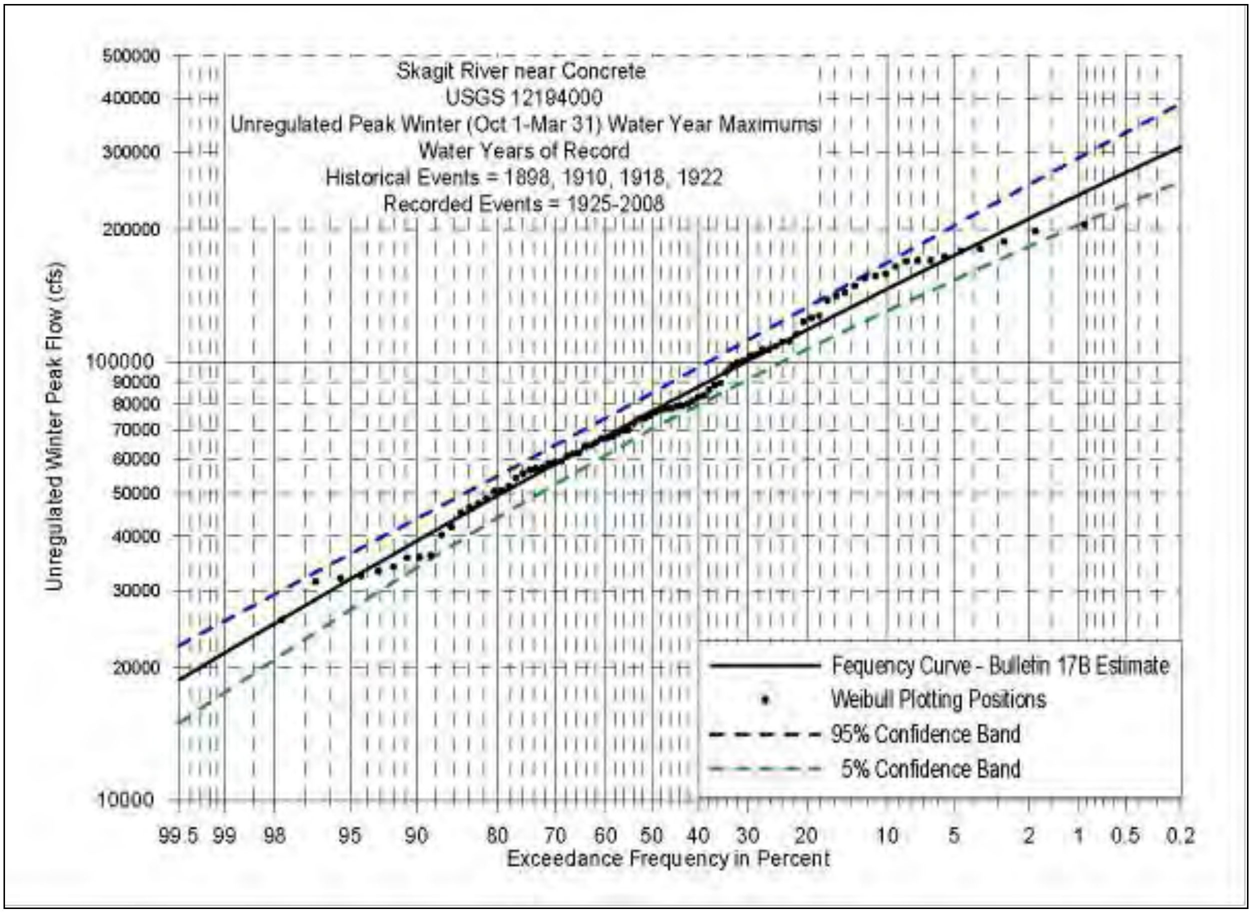

Figure 19. The unregulated peak flow frequency curve for the Skagit River near Concrete, and the confidence band together with all data used in the frequency analysis at Concrete

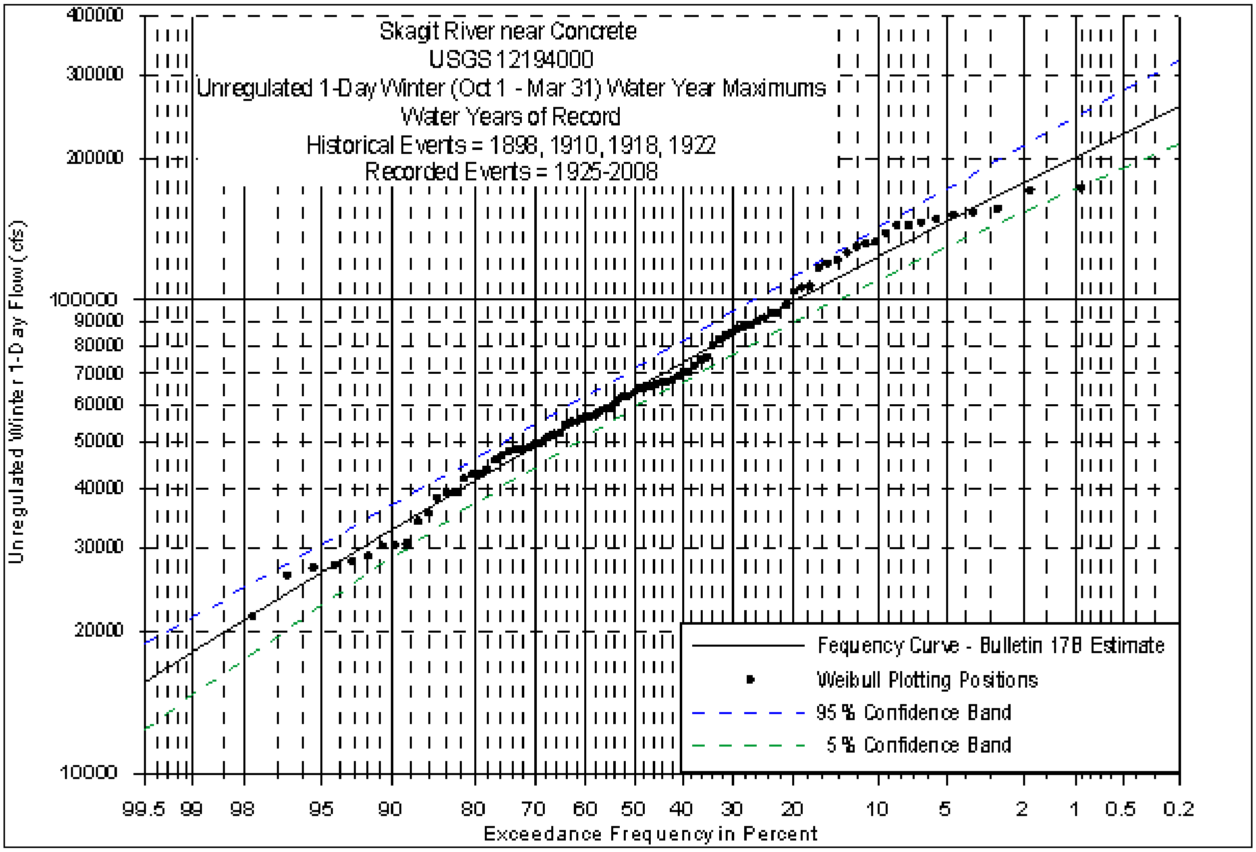

Figure 20. The unregulated one-day flow frequency curve and the confidence band together with all data used in the frequency analysis at Concrete

2.6 Summary of Revised Hydrology Analysis and Estimates of 10-, 50-, 100-, and 500-Year Flood Peak Discharges

Development and routing of unregulated and regulated synthetic flood hydrographs for the 10-year, 50-year, 100-year, and 500-year (or 10, 2, 1, and 0.2 percent annual chance) were performed, using our revised flood estimates and frequency curves in conjunction with the use of the USACE developed HEC-RAS and HEC-5 models for the Skagit River Basin. Details of the unregulated and regulated synthetic flood hydrograph development and routing are presented in Appendix A to this Technical Report.

Figure 21 show the plots of unregulated and regulated discharges for a comparison, indicating similar regulation results of the upstream flood control storage operation in the basin, between the USACE and PI Engineering analyses. Table 16 provides a summary of the regulated and unregulated peak discharges of the 10-year, 50-year, 100-year, and 500-year synthetic floods analyzed by the USACE and PI Engineering (PIE).

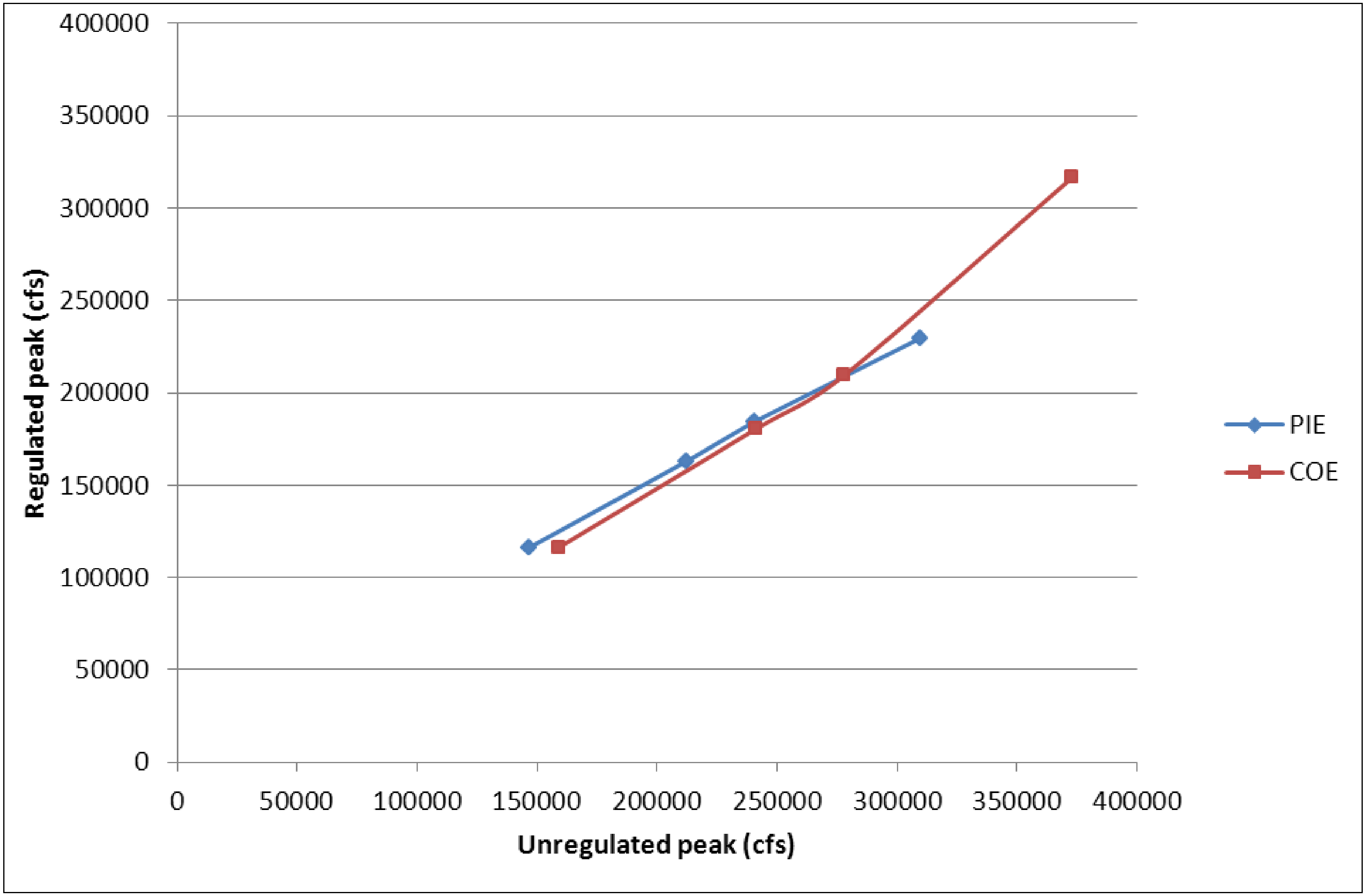

Figure 21. Regulated and unregulated peak discharges at Concrete resulting from the Corps (COE) and PI Engineering (PIE) analyses

Table 16. Summary of regulated and unregulated peak discharges (cfs) at Concrete gage

Flood Event	USACE Regulated	USACE unregulated	PIE regulated	PIE unregulated
10-year	116,300	159,000	116,100	146,800
50-year	180,260	241,000	162,600	212,100
100-year	209,490	278,000	184,400	240,800
500-year	316,530	373,000	229,400	309,500

The USGS-published data for the 1897, 1909, 1917, and 1921 historical floods and the 1932 flood are currently used in the RFIS hydrology analysis for estimates of the 10-year, 50-year, 100-year, and 500-year (or 10, 2, 1, and 0.2 percent annual chance) synthetic flood peak discharges, which are listed in Table 17. Use of the new data estimated by PI Engineering will improve the accuracy of the synthetic flood estimates, which are also listed in Table 17. Details of the PI Engineering's downstream flood routing analysis are presented in Appendix A to this Technical Report.

Table 17. Comparison of regulated 10-, 50-, 100-, and 500-year flood peak discharges at Concrete and Sedro-Woolley for existing basin conditions with upstream dam storage regulation

	Concrete (RM 54.15)		Sedro-Woolley (RM 22.40)	
Flood	USACE Developed Peak Discharge (cfs)	PI Eng. Revised Peak Discharge (cfs)	USACE Developed Peak Discharge (cfs)	PI Eng. Revised Peak Discharge (cfs)
10-year	116,300	116,100	123,610	117,200
50-year	180,260	162,600	183,780	161,900
100-year	209,490	184,400	215,270	184,700
500-year	316,530	229,400	322,900	231,700

2.7 Sources of Hydrology Supporting Data

Sources of data, including Stewart's 1922–23 field survey notes, Stewart's unpublished 1923 report (including Exhibit B), and Stewart's 1950 letter to the USGS District Engineer (including his memorandum), are provided on a CD which is entitled "Hydrology Supporting Data and Model Run Files for Skagit

River RFIS Appeal" and attached to this Technical Report. The USGS 2007 report for reevaluation of the 1921 flood peak discharge; PI Engineering 2007 Technical Memorandum – Hydraulic Analysis, Smith House Flood Stages; and HEC-RAS, HEC-5 and PEAKFQ model run files are also included on the CD.

3.0 Inconsistent Levee Methodology and Poor-Quality Topographic Data Used in the Hydraulic Analysis

3.1 Inconsistent Levee Methodology Used in the RFIS Hydraulic Analysis

A FLO-2D model developed by the Corps of Engineers (COE, or COE) was used in the RFIS hydraulic analysis to model the lower Skagit River valley below RM 22.4, which is comprised of flows that travel out of stream channels and across the topography of the floodplain. A levee methodology involving seven levee removal scenarios was applied to the modeling to appropriately depict the base flood elevations. The levee methodology used for developing the base flood elevations from the different levee condition scenarios was derived from Appendix H of the Flood Insurance Study Guidelines and Specifications for Study Contractors (FEMA, 2002). These seven levee removal scenarios are described in detail in the report titled "Skagit River Basin, Washington, Revised Flood Insurance Study, Hydraulics Summary" (COE, 2009).

PI Engineering reviewed the model runs for these seven levee removal scenarios, and found that the levee methodology was inconsistently applied to the scenario shown in Figure 22 - Right Bank Levees on Mainstem and North Fork Skagit River Removed While All Other Levees Remain Intact. For this scenario, the removed right-bank levee starts upstream approximately at RM 21 in the current model. It is our opinion that an additional 1.2-mile levee (RM 21 – 22.2) upstream also needs to be removed, as annotated in the figure.

This 1.2-mile levee reach consists of earth-fill embankments along SR-20 and BNSF railway, between Collins Road in Sterling and Rhodes Road in Sedro-Woolley, including a 20-ft high dam that plugs the original upstream inlet opening of Gages Slough near Holtcamp Road (in the model grid cells 21411 and 21502). These roadway and railway fills and the dam effectively act like an upstream extension of the right-bank levee system during floods. During flood events, the community routinely undertakes flood fights along this reach. None of the fills in this reach, similar to the downstream levee system, has been certified to meet the minimum levee requirements of 44 CFR Section 65.10 under the NFIP regulations.

The flood mapping and modeling for the preliminary DFIRM and RFIS are not consistent in treating this fill reach. In mapping the floodplain, this reach was considered as a part of the right bank uncertified levee, as shown in Figure 23 (which is the Figure 11 of the COE hydraulics summary report, COE 2009). But when modeling the base flood elevations for the right bank floodplain, it was not included as a part of the right bank uncertified levee in the model. To correct this inconsistency, the right-bank levee removal scenario needs to extend upstream to include the removal of this 1.2-mile fill as an uncertified levee.

Figure 24 shows the FLO-2D model levee alignments revised by PI Engineering to include the extension of this 1.2-mile right-bank uncertified levee, in comparison with the model levee alignments originally developed by the COE. The figure also shows other PI Engineering revised levee segments representing high ground or road fill in the City of Burlington and Gages Slough floodplain area. These levee revisions are based on recent and more accurate topographic data collected by the City of Burlington in this area, which are discussed in the next section.

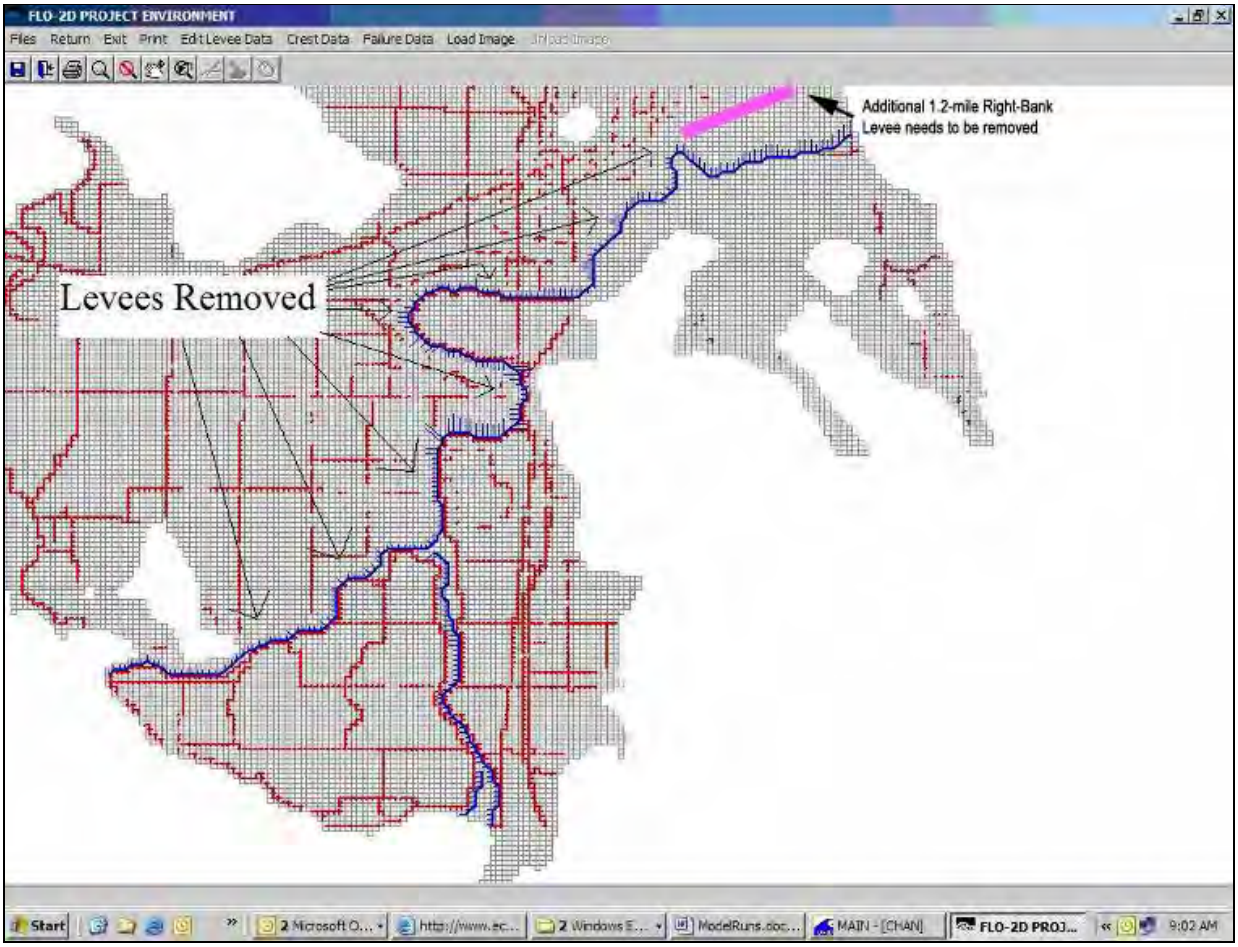

Figure 22. Right Bank Levees on Mainstem and North Fork Skagit River Removed While All Other Levees Remain Intact (original source: Figure 9, COE 2009)

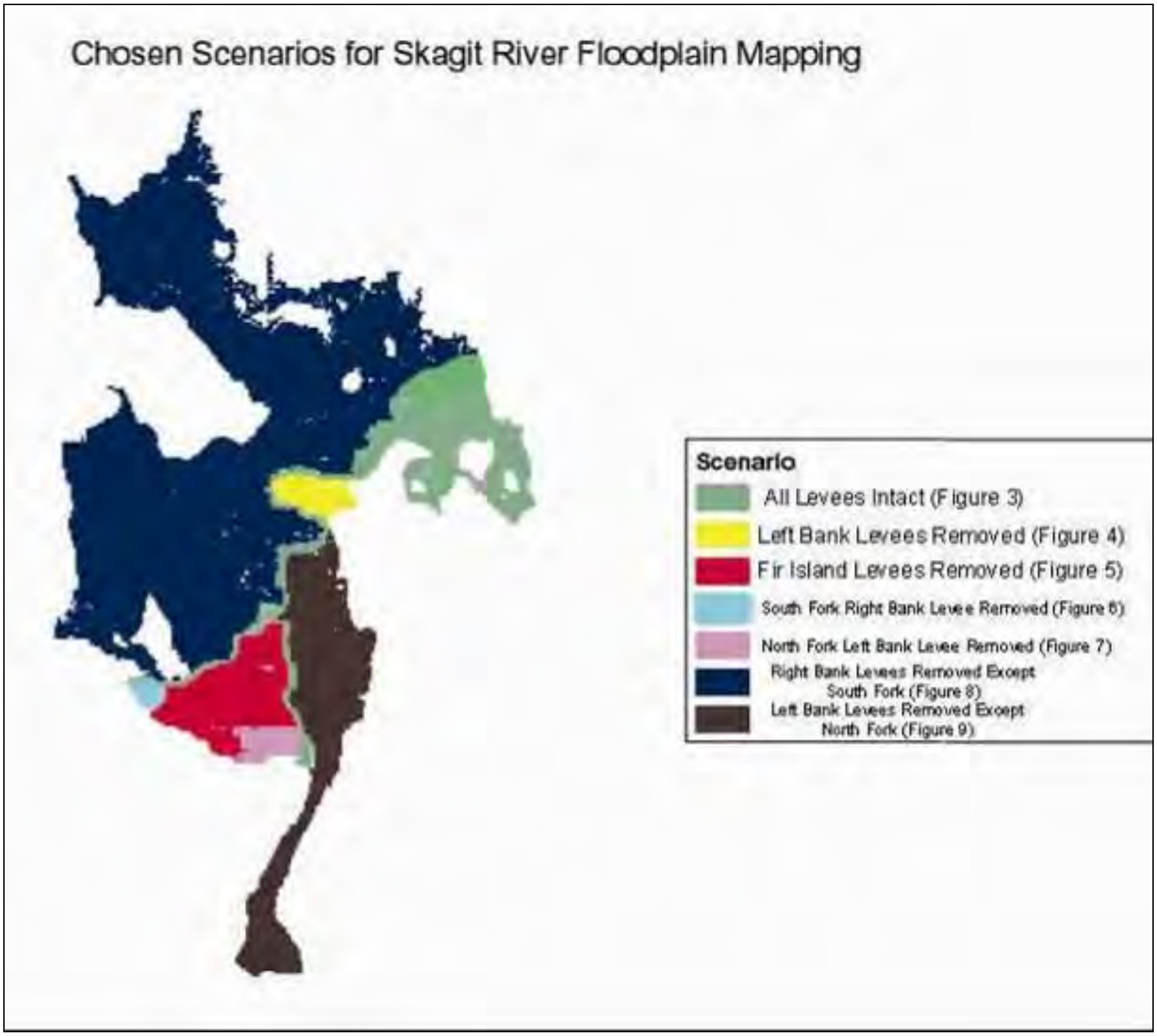

Figure 23. Depiction of the Floodplain Scenario Location of Implementation (original source: Figure 11, COE 2009)

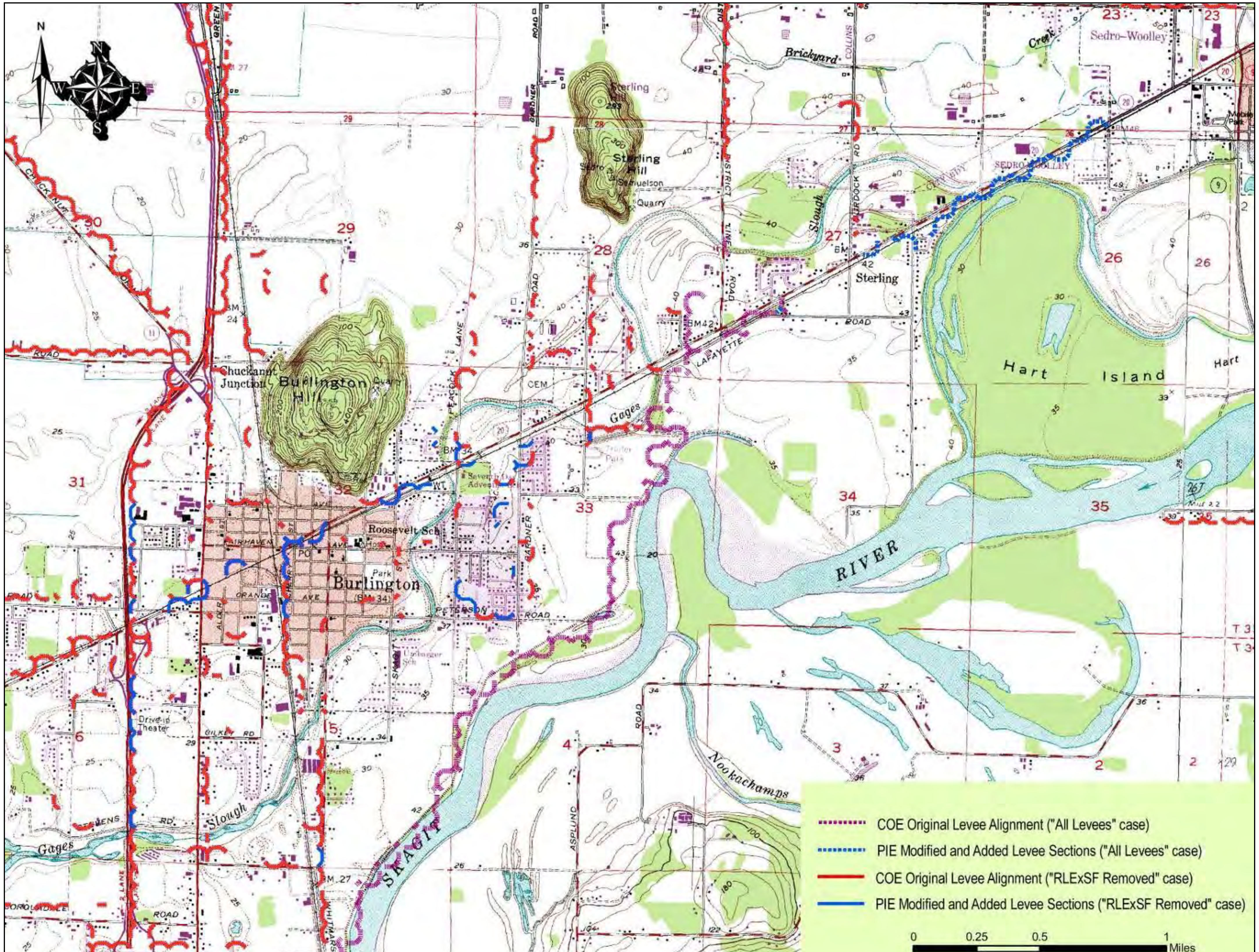

Figure 24. Comparison of COE Original and PIE Revised FLO-2D Model Levee Alignments

3.2 Poor-Quality Topographic Data Used in the RFIS Hydraulic Analysis

A FLO-2D model with a 400-by-400 foot grid system was used in the RFIS hydraulic analysis to route the overbank flows in the lower Skagit River valley below RM 22.4. The entire floodplain for the lower Skagit Valley was aerial surveyed in 1999 by the Corps of Engineers (COE) at a scale of 1 inch = 400 feet, based on the NGVD 1929 vertical datum. The FLO-2D grid of the floodplain developed by the COE uses the mapping information from this aerial flight. This mapping was done to an accuracy that meets ASPRS standards for Class 2 accuracy for two-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 COE 1999 topographic data are not the best topographic data available for the modeling to depict the base flood elevations in the Burlington and Mount Vernon urban areas.

A comparison of this topographic data with recent and more accurate topographic data collected by the City of Burlington shows that the COE 1999 mapping elevations are higher along the I-5 and the SR-20 corridor within the City and in the Sterling area. The topographic data provided by the City shows the roadway is 2 to 3 feet lower than modeled along about 2000 feet of I-5, and the ground elevations are from a few inches to over one foot lower than modeled along the SR-20 corridor east of I-5. The roadway elevations of I-5 and SR-20 coded as floodplain levees are critical elevations in the model to determine the flood elevations in the Burlington core area. The topographic data used by the COE tend to be biased upwards, resulting in higher flood elevations in this area.

A review of the newer topographic data collected by the City of Burlington indicates there are grid cells with higher ground elevations than modeled. Some are because only sparse elevation points were available from the 1999 mapping to develop the COE original model, resulting in the 400x400 foot grid cell elevations different from the true average grid cell elevations. Others are because extensive fill and development in the City of Burlington have taken place since 1999, and are not shown on the COE maps.

3.3 Better-Quality Topographic Data Collected by the City of Burlington

Topographic data at a scale of 1” = 100 ft or better were recently collected by the City of Burlington. Compared to the COE 1999 data at a scale of 1”=400 ft, these more refined topographic data are better quality. The newer data include the 2004 topographic data for the entire City of Burlington area and the 2009 topographic data for the Dike District 12 (DD 12) levee within the City and the SR-20 corridor in the Sterling area.

3.3.1 2004 Topographic Data for City of Burlington

The entire City of Burlington was aerial mapped in 2004. A digital elevation model (DEM) was created by mapping breaklines along significant topographic features and collecting mass points at regular intervals throughout the stereomodel. Standard density of spot heights included 100-foot spacing between spot heights in open areas, with additional spot heights collected at road intersections and road ends. A set of 2-foot contour maps at a scale of 1 inch = 100 feet was produced for the entire City of Burlington. The mapping was produced in compliance with the National Standard for Spatial Data Accuracy (NSSDA) for horizontal and vertical accuracy. The horizontal datum is NAD 83/91 Washington State Plane Coordinates, North Zone and the vertical datum is NGVD 1929.

The elevation differences between the City of Burlington 2004 topographic data and the 1999 topographic data used by the COE in the FLO-2D model are generally within a few inches (+/-), but over 2 to 3 feet on spots of I-5 roadway north of the Gages Slough crossing and about one-half to over one foot along the SR-20 corridor. Also affecting the accuracy of the grid cell and levee elevations coded into the model, is that the COE 1999 mapping provides only very sparse elevation points, mostly spacing at 400 feet, while there are plentiful spot elevation points provided on the City of Burlington 2004 maps.

3.3.2 2009 Topographic Data for DD 12 Levee and SR-20 Corridor

Aerial mapping for the DD 12 levee within the City of Burlington and the SR-20 corridor in the Sterling area, at a scale of 1 inch = 50 feet for one-foot contours based on the NGVD 1929 datum, was undertaken in 2009 by the City in partnership with DD 12. The purpose of this mapping was to provide base maps for preparation of the engineering design and construction plans for the on-going DD 12 levee improvement and certification project. This new set of topographic data has contour accuracy +/- 0.5 feet and spot elevation accuracy +/- 0.25 feet. Confidence level is 90 percent of absolute (or 90 points out of 100 are within the above accuracies). This newest set of topographic data has the best horizontal and vertical accuracies, but covers only a narrow strip along the existing levee and the SR-20 corridor where the potentially improved and certified levee system is located.

The elevation differences at the levee project site compared with the 1999 topographic data used by the COE in the FLO-2D model are generally within a few inches (+/-) to over one foot in open areas. Again, affecting the accuracy of the data of the grid cell and levee elevations coded into the model, is that the COE 1999 mapping provides only very sparse elevation points, mostly spacing at 400 feet, while there are numerous spot elevation points provided on the City of Burlington 2009 maps.

3.3.3 Sources of Topographic Data

The documentation and topographic data collected by the City of Burlington in 2004 and 2009 are provided on a CD which is entitled “City of Burlington 2004 and 2009 Topographic Data for Skagit River RFIS Appeal".

3.4 Revised FLO-2D Model Using Consistent Levee Methodology and Better-Quality Topographic Data

PI Engineering revised the model, using the consistent levee methodology and the more current and more accurate topographic data collected by the City of Burlington in 2004 and 2009. The model revisions include:

(1) Extending the right-bank uncertified levee upstream for 1.2 miles along the SR-20 roadway and BNSF railway fill corridor including the 20-ft high dam at the Gages Slough inlet in the Sterling area as shown in Figure 22.

(2) Adding or deleting other segmental levees representing high ground or road fill in the City of Burlington and Gages Slough floodplain area as shown in Figure 24.

(3) Lowering I-5 roadway elevations by 2 to 3 feet for about 2000 feet north of the Gages Slough crossing.

(4) Lowering SR-20 roadway elevations by one-half to one foot for about one mile east of I-5.

(5) Revising the grid cell ground elevations as indicated in Figure 25 which shows the cell ground elevation difference between our revised model and the COE original model.

(6) Including a representation of Gages Slough crossing underneath I-5, which was not originally included in the COE model, i.e. it was modeled as if the Slough is completely blocked.

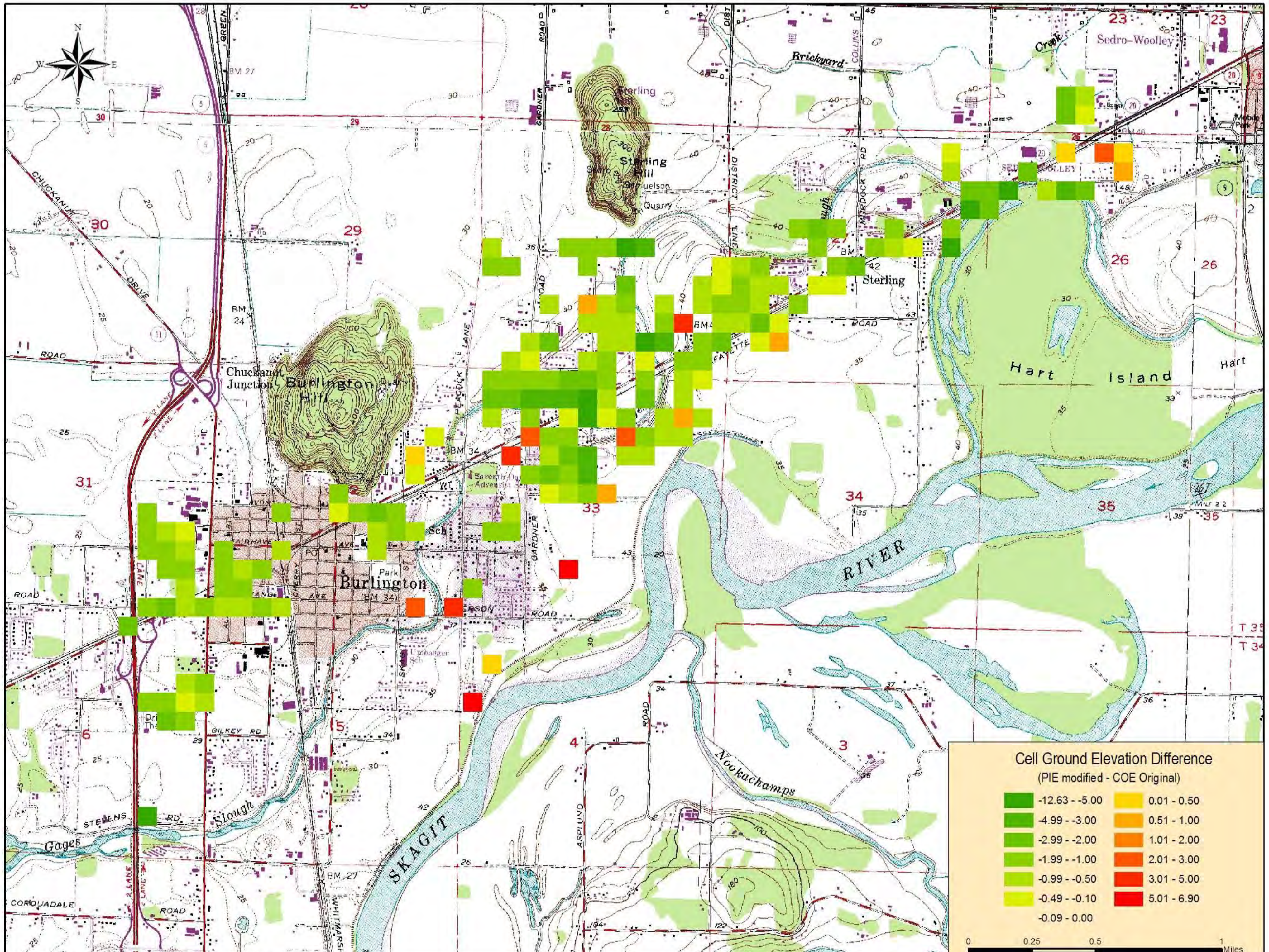

Figure 25. Cell Ground Elevation Difference between PIE Revised and COE Original FLO-2D Model

4.0 Revised BFEs and Flood Boundaries Using Revised Hydrology and Revised FLO-2D Model

PI Engineering applied our revised hydrology to our revised FLO-2D model, and performed new model runs for the levee removal scenarios, as determined by FEMA for the RFIS, to depict the revised base flood elevations (BFEs) in the Burlington and Mount Vernon urban and vicinity rural areas. Figure 26 shows the BFE delineation based on our revised model runs in these areas. The figure also shows the BFE delineation based on the COE original model runs for a comparison. Figure 27 shows the differences between the revised BFEs resulting from our revised model runs and the original BFEs resulting from the COE model runs for the FEMA released preliminary DFIRM and RFIS. The revised BFEs are substantially lower in the Burlington and Mount Vernon urban areas and in most of the vicinity rural areas with exception of the area around Sterling Hill and the area along Gages Slough east of Sterling Hill. Table 18 provides a comparison between our revised BFEs and the COE originally developed BFEs at selected locations in Burlington and Mount Vernon. Figure 28 shows the difference of the base flood boundaries between our revised BFE and the COE developed BFEs.

It is our opinion that the revised BFEs are more accurate than the COE modeled BFEs. Details of the model results are provided on a DVD which is entitled "Revised FLO-2D Model Run Files for Skagit River RFIS Appeal". Our revised flood profiles resulting from the revised FLO-2D model runs are included in Appendix B, 20P-33P. Our revised flood zone boundaries are provided on a DVD which is entitled "Revised DFIRM Database Files for Skagit River RFIS Appeal".

Table 18. Comparison of BFE in Burlington and Mount Vernon

Model Cell ID*	Location	PIE Revised BFE (ft, NGVD-29)	COE Developed BFE (ft, NGVD-29)	Difference (ft)
17028	Haggen Dr. @ Haggen Food & Pharmacy	32.75	34.44	-1.69
16989	South of Cascade Mall	33.17	34.57	-1.40
16967	College Way @ Skagit Valley Mall	34.36	36.09	-1.73
17160	Kincaid St. @ Court House	24.39	25.71	-1.32
18380	BNSF Bridge	39.99	42.70	-2.71
17392	Riverside Rd Bridge	38.89	41.77	-2.88
16558	I-5 Bridge	38.62	41.28	-2.66
16954	Division St. Bridge	30.55	32.33	-1.78

*NOTE: See Figures 24 and 25 for cell location

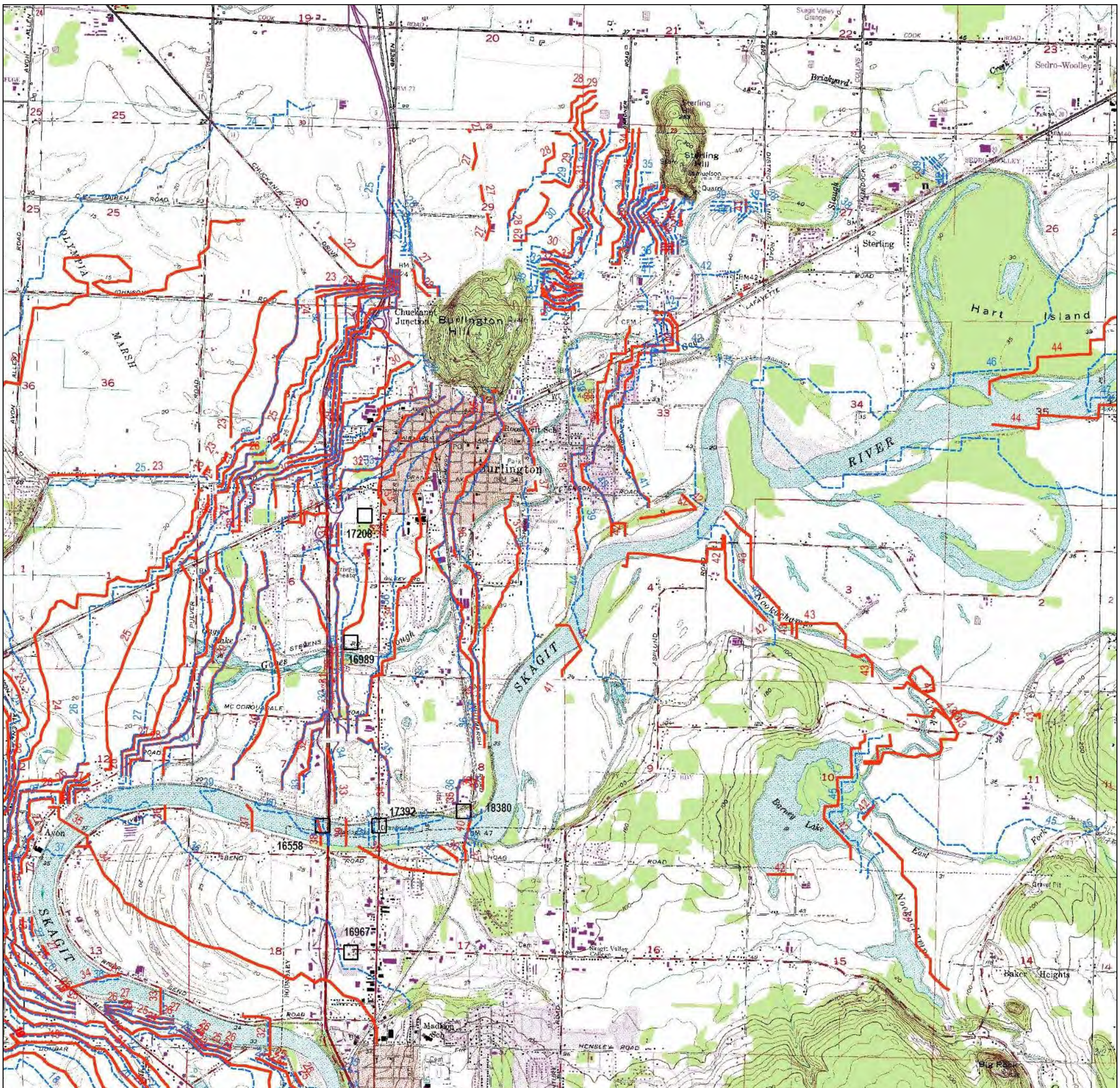

Figure 26. Comparison of Base Flood Elevation Delineation between PIE Revised and COE Original Model Runs

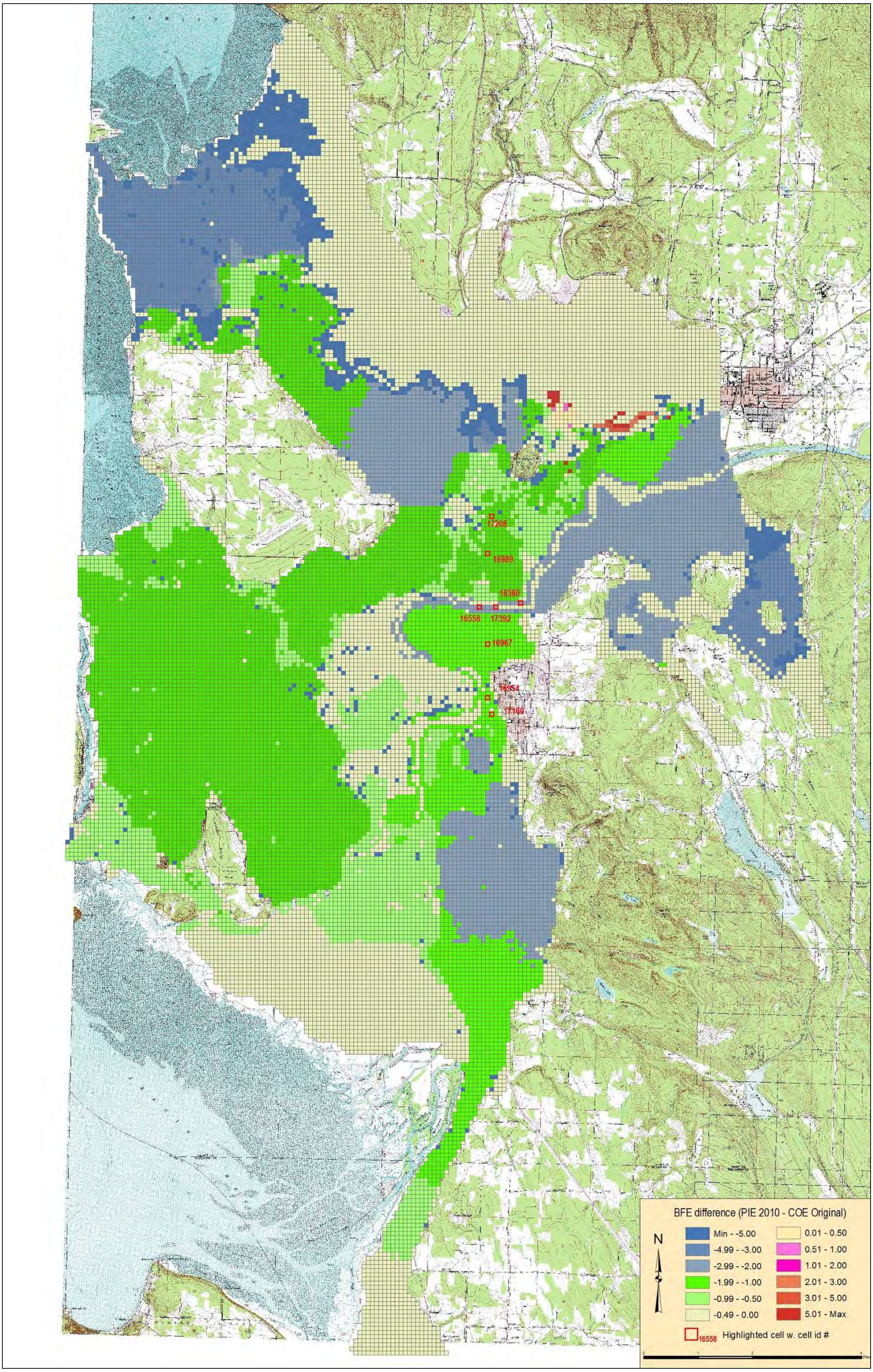

Figure 27. Base Flood Elevation Difference between PIE Revised and COE Original Model

Figure 28. Base flood boundary difference map

5.0 Revised Upstream Skagit River BFEs, Flooway Data, and Flood/Floodway Boundaries using Revised Hydrology and RFIS-used HEC-RAS Model

To tie the new flood elevations, flood zone boundaries, and regulatory floodway boundaries into those shown for upstream area not affected by the appeal, PI Engineering applied our revised hydrology to the unsteady flow HEC-RAS model used by the COE in the RFIS for the portion of the Skagit River between RM 22.4 and 56.61. As shown in Figure 28, PI Engineering revised BFEs reduce more than 2 ft from the COE original BFEs at the upstream limit of the FLO-2D model area, or at the Highway 9 Bridge in Sedro-Woolley (RM 22.4). Water surface elevations of the 10-, 50-, 100-, and 500-year floods, and the floodways for the upstream Skagit River above the FLO-2D model area were determined by the COE for the RFIS using the unsteady flow HEC-RAS model. [3]

The Skagit River flood stages at the Baker River confluence shown on Flood Profiles - 52P of the FEMA released preliminary RFIS report appear significantly under-estimated, when compared to the observed 2003 flood stages (see Table 12). The Skagit River flood stage at the Baker River confluence for the 50-year flood discharge of 180,260 cfs used in the RFIS and modeled by the COE is plotted at El. 187 (NAVD-88), corresponding to El. 183.2 (NGVD-29). This is only 0.45 ft above the observed 2003 flood stage El. 182.75 for 165,655 cfs at the Jenkins house located at RM 56.18 (see Figure 12), between the RFIS cross sections AR (RM 55.75) and AS (RM 56.70). Based on the stage-discharge rating curve for the 2008 channel sections shown in Figure 14, adjusted for the water surface drop of 0.7 ft estimated from the Baker River gage to the Jenkins house (see Figure 15), the flood stage for 180,260 cfs is at El. 184.7, or 1.5 ft higher than that plotted on Flood Profiles - 52P. The HEC-RAS model used by the COE for the Skagit River flood profiles was not calibrated for the 2003 flood stage observed at the Jenkins house, and is therefore not accurate for use to predict the Skagit River flood stages at the Baker River confluence.

In revising the upstream Skagit River flood elevations, we used the observed 2003 flood stage at the Jenkins house and calibrated the COE originally used HEC-RAS model. The calibration of the model increases the channel Manning's "n" values from 0.045 to 0.060 at the model cross sections between RM 54.1 and RM 54.65, where the flow energy losses are high during floods due to two 90-degree turns of the Skagit River channel through the Dalles. The calibration results indicate that the difference between the modeled 2003 flood elevation of 182.94 at RM 56.70 and the observed 2003 flood elevation of 182.75 at the Jenkins house is 0.19 ft, which is considered satisfactory. This calibrated HEC-RAS model was then used to model the upstream Skagit River flood elevations and the floodways for our revised hydrology.

[3] Note: The HEC-RAS zip file, Skagit River model us Sedro Woolley.zip, was provided to us on March 1, 2011; and the DFIRM database zip file, Skagit County DFIRM_DB.zip, was provided to us on March 7, 2011. Both zip files were provided by Ana Paula Simões, Ph.D., CFM, Water Resources Engineer, Flood Hazard Management, STARR – FEMA Region X Support Center. We would like to express our appreciation for the help provided by FEMA Region X Support Center.

Details of the model results are provided on a DVD which is entitled “Revised HEC-RAS Model Run Files for Skagit River RFIS Appeal”. Our revised flood profiles resulting from the revised HEC-RAS model runs are included in Appendix B, Skagit River 34P-53P and Baker River 01P. Our revised floodway data resulting from the revised HEC-RAS model runs are included in Appendix C, Skagit River Cross Sections A through AT. Our revised flood zone boundaries and floodway boundaries are provided on a DVD which is entitled “Revised DFIRM Database Files for Skagit River RFIS Appeal.

6.0 Revised Flood Profiles

Based on our revised hydrology and hydraulic analysis, the revised flood profiles listed below are presented in the appendix to this Technical Report.

- Skagit River Delta Overbank Flowpath 1 Panels 20P – 21P
- Skagit River Delta Overbank Flowpath 2 Panels 22P – 25P
- Skagit River Delta Overbank Flowpath 3 Panels 26P – 28P
- North Fork Skagit River Panels 29P – 31P
- South Fork Skagit River Panels 32P – 33P
- Main Stem Skagit River Panels 34P – 53P
- Baker River Panel 01P

7.0 Certification by Registered Professional Engineer

CERTIFICATION BY REGISTERED PROFESSIONAL ENGINEER

The supporting data and analyses presented in this Technical Report are correct to the best of my knowledge.

Certifier's Name: Albert Liou, P.E.	**License No.:** WA State P.E. #19768	**Expiration Date:** 08/23/2011
Company Name: Pacific International Engineering, PLLC	**Telephone No.:** 425.744-7700	**Fax No.:** 425-921-1740
Signature:		**Date:** 3/28/2011

ALBERT LIOU
STATE OF WASHINGTON
19768
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 8/23/2011

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APPENDIX A

Development and Routing of Synthetic Flood Hydrographs

1.0 Synthetic Flood Hydrographs at Concrete

This section presents information on development of the synthetic flood hydrographs for the Skagit River at Concrete. The HEC-5 and HEC-RAS models originally developed by the Corps and subsequently improved by PI Engineering were used to route the coincident synthetic flood hydrographs. The hydrograph routing was performed for the area of the Skagit River above Concrete (see Figure A-1), first for

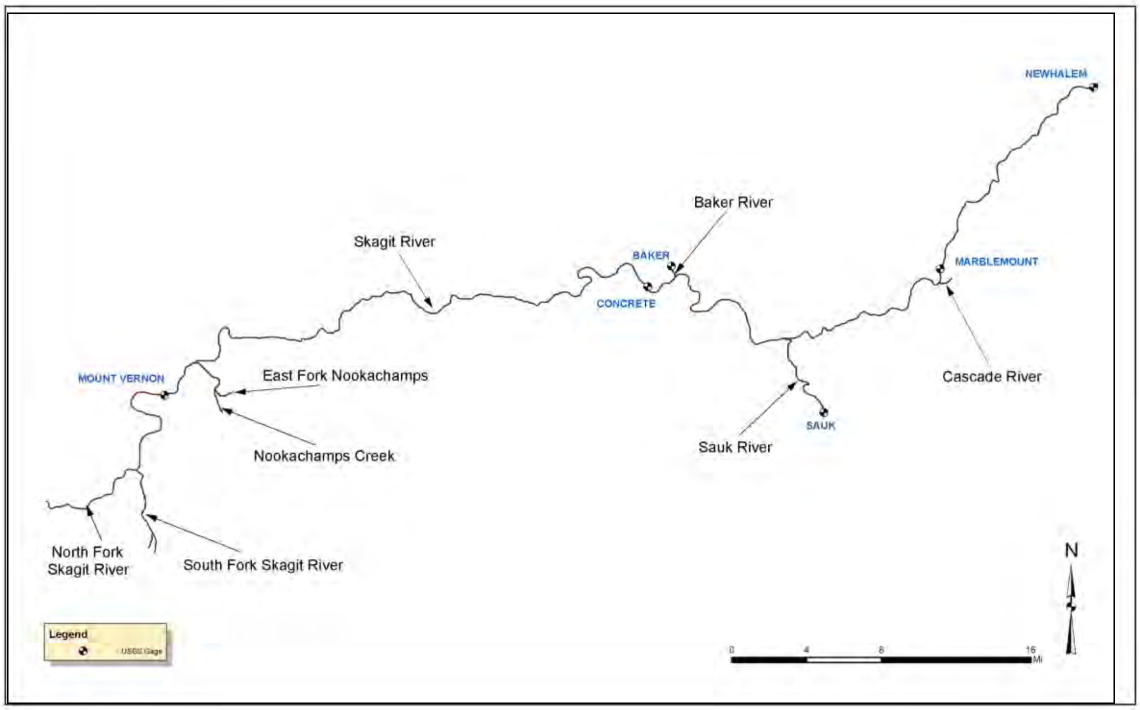

Figure A-1. Skagit River HEC-RAS model routing reaches

1.1 Development of Unregulated Synthetic Flood Hydrographs

Based primarily on the unregulated peak one-day flow data and various regressions, the Corps developed coincident flood hydrographs for nine upper Skagit River subbasins above Concrete. A total of nine synthetic flood hydrographs for each subbasin was constructed by the Corps. Details of the Corps-developed synthetic flood hydrographs for these subbasins are presented in the Corps' Draft Report – Skagit River Basin, Washington, Revised Flood Insurance Study, Hydrology Summary (Corps 2005 and 2008).

PI Engineering applied the improved HEC-5 and HEC-RAS models to route the unregulated flood hydrographs for the FEMA FIS-required 10-, 50-, 100-, and 500-year synthetic flood events along the Skagit River from Ross Dam to Concrete including Cascade, Sauk and Baker River tributaries. Details of the HEC-5 (without flood control storage operation) and HEC-RAS models are provided in the PI Engineering April 1, 2005 Draft Technical Memorandum –

Skagit River Basin Historical Flood Modeling – Hydrology (PI Engineering, 2005) and the November 29, 2004 Draft Technical Memorandum – Skagit River Basin Historical Flood Modeling – Hydraulics (PI Engineering, 2004).

The peak and one-day flows of the synthetic flood hydrographs routed to Concrete were compared with the corresponding unregulated events statistically developed for Concrete. These flows and subbasin hydrographs were then scaled and routed again as necessary until the routed flows matched the unregulated peak and one-day flows that were derived by the flood frequency analysis using the PEAKFQ model. The one-day scaled flows are listed in Table A-1.

Figure A-2 and Figure A-3 show the plots of the peak and the one-day flows, respectively, at Concrete for the four HEC-RAS simulated unregulated synthetic flood events, in comparison with the corresponding flood frequency curves. This comparison indicates that the unregulated peak and one-day flows resulting from the HEC-5 and HEC-RAS routing of the constructed synthetic flood hydrographs for each of the 10-, 50-, 100-, and 500-year events match very well with the statistically-derived unregulated peak and one-day flows at Concrete.

Table A-1 Unregulated synthetic flood one-day coincident flows (cfs) for upper Skagit River subbasins

	Flood Event			
Location	**10-year**	**50-year**	**100-year**	**500-year**
Unregulated Skagit River Near Concrete	123,700	177,900	201,400	257,500
Ross Dam Inflow	23,700	34,100	39,100	49,300
Thunder Creek and Ross Dam to Newhalem Local	8,500	12,300	14,000	17,700
Newhalem to Marblemount Local	17,600	25,400	29,000	36,700
Cascade River at Marblemount	8,100	11,600	13,300	16,800
Marblemount to Sauk Local	4,800	6,900	7,900	10,000
Sauk to Concrete Local	3,300	4,800	5,500	6,900
Sauk River at Sauk	39,800	57,300	65,600	82,800
Upper Baker Dam Inflow	17,000	24,500	28,100	35,400
Lower Baker Dam Inflow	4,800	7,000	8,000	10,100

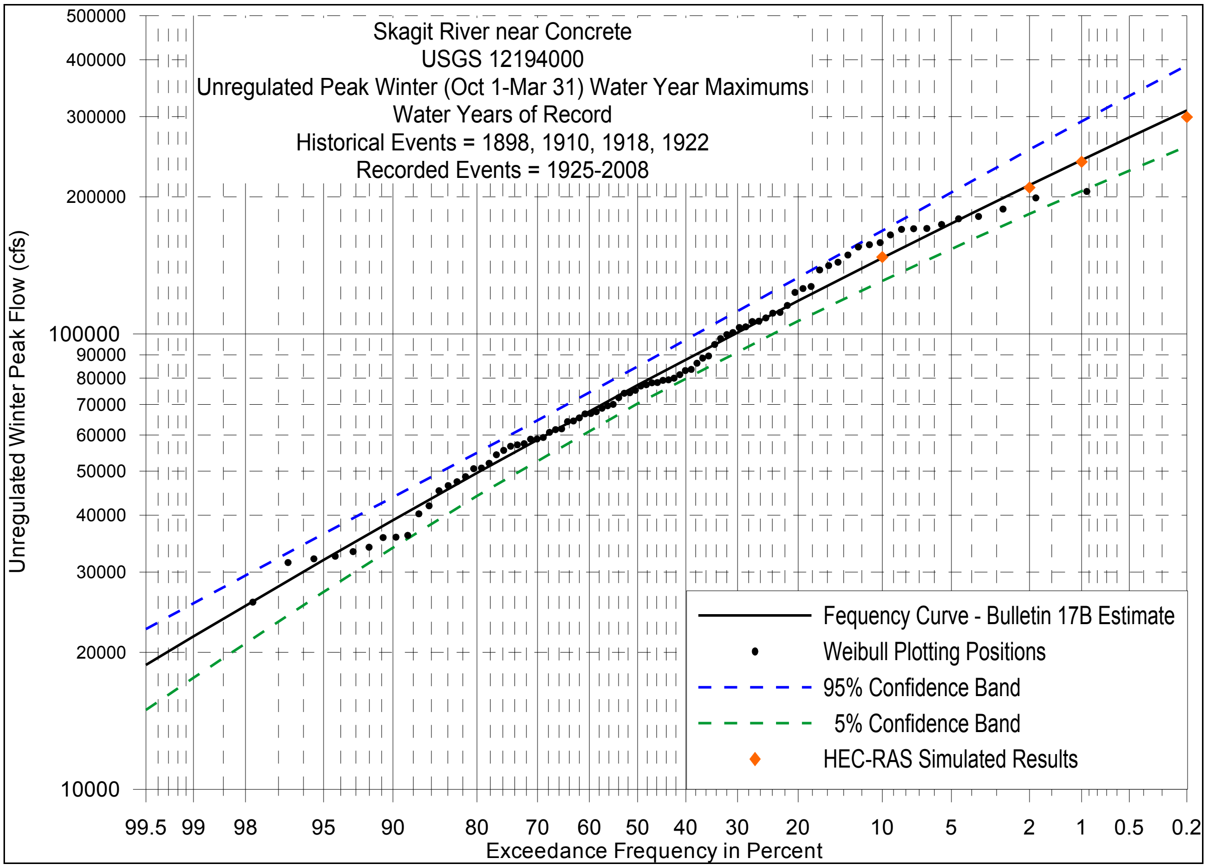

Figure A-2. Flood frequency curve for unregulated peak discharges at Concrete, compared with the HEC-RAS simulated peak flows at Concrete for the 10-, 50-, 100-, and 500-year synthetic events

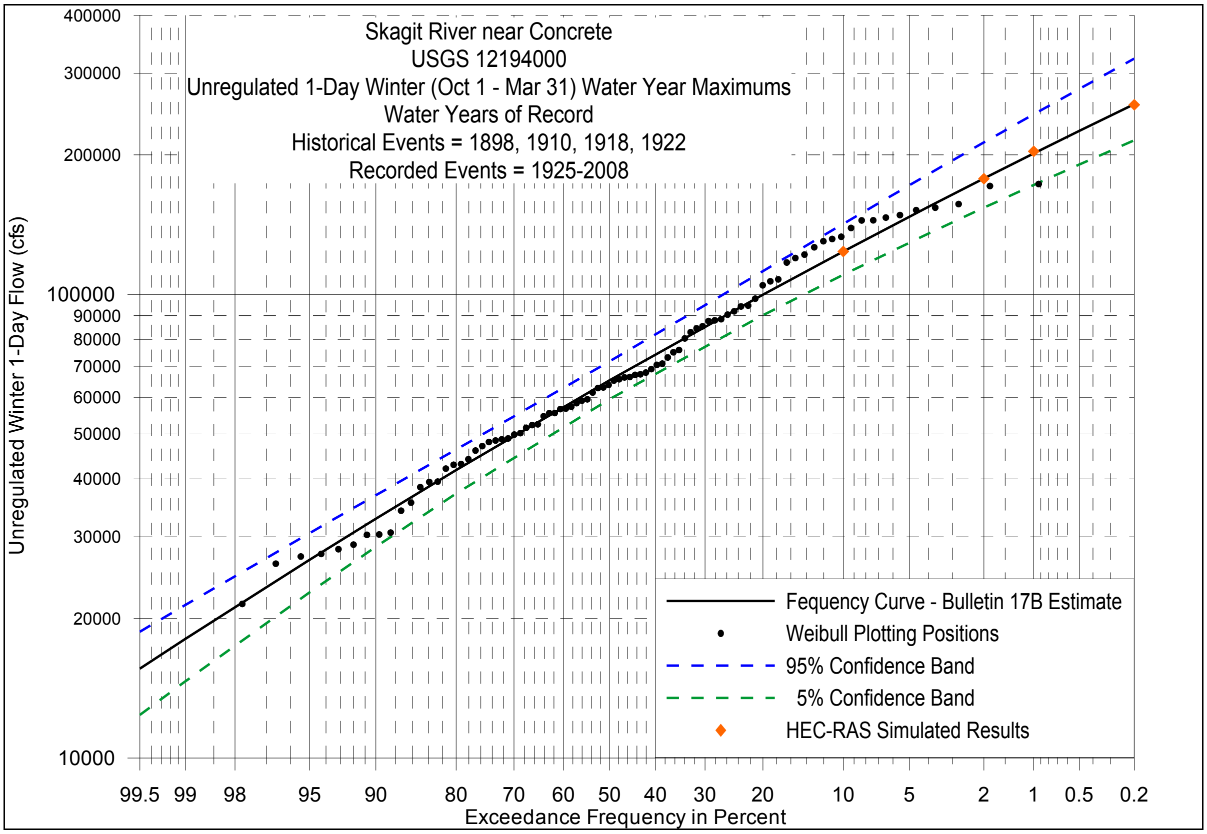

Figure A-3. Flood frequency curve for unregulated one-day discharges at Concrete, compared with the HEC-RAS simulated one-day flows at Concrete for the 10-, 50-, 100-, and 500-year synthetic events

1.2 Development of Regulated Synthetic Flood Hydrographs

The coincident unregulated hydrographs of all subbasins above Concrete for each of the 10-, 50-, 100-, and 500-year synthetic flood events derived as discussed above were then routed by the HEC-5 model with the existing flood control storage of 120,000 and 74,000 acre-feet provided at Ross Dam and Upper Baker Dam, respectively. The regulated outflow hydrographs at these two dams and local inflow hydrographs representing subsequent flow contribution from subbasins were routed by the HEC-RAS model along the Skagit River and main tributary routing reaches to Concrete. Development and details of the HEC-5 and HEC-RAS routing models are discussed in the Draft Technical Memorandum – Skagit River Basin Historical Flood Modeling – Hydrology (PI Engineering, 2005) and the Draft Technical Memorandum – Skagit River Basin Historical Flood Modeling –Hydraulics (PI Engineering, 2004).

Figure A-4 and Figure A-5 show the plots of the annual peak and one-day flows, respectively, at Concrete for the four routed regulated synthetic flood events, in comparison with the corresponding flood frequency curves based on PEAKFQ modeling of the USGS observed regulated flow data at Concrete for the time period from 1955 through 2006 (water years 1956-2007).

The comparison shown in Figure A-4 and Figure A-5 indicates that the regulated annual peak and one-day flows resulting from the HEC-5 and HEC-RAS routing of the synthetic flood hydrographs for each of the 10-, 50-, 100-, and 500-year events match reasonably well with the projection and within the confidence band of the frequency curves based on USGS observed regulated data at Concrete. It is noted that since the actually observed regulated data do not include the low-flow hydrological years preceding 1956, it is reasonable to expect that the frequency curves plotted from these observed regulated data are shown in the figures above the plotted points of the modeled four synthetic floods.

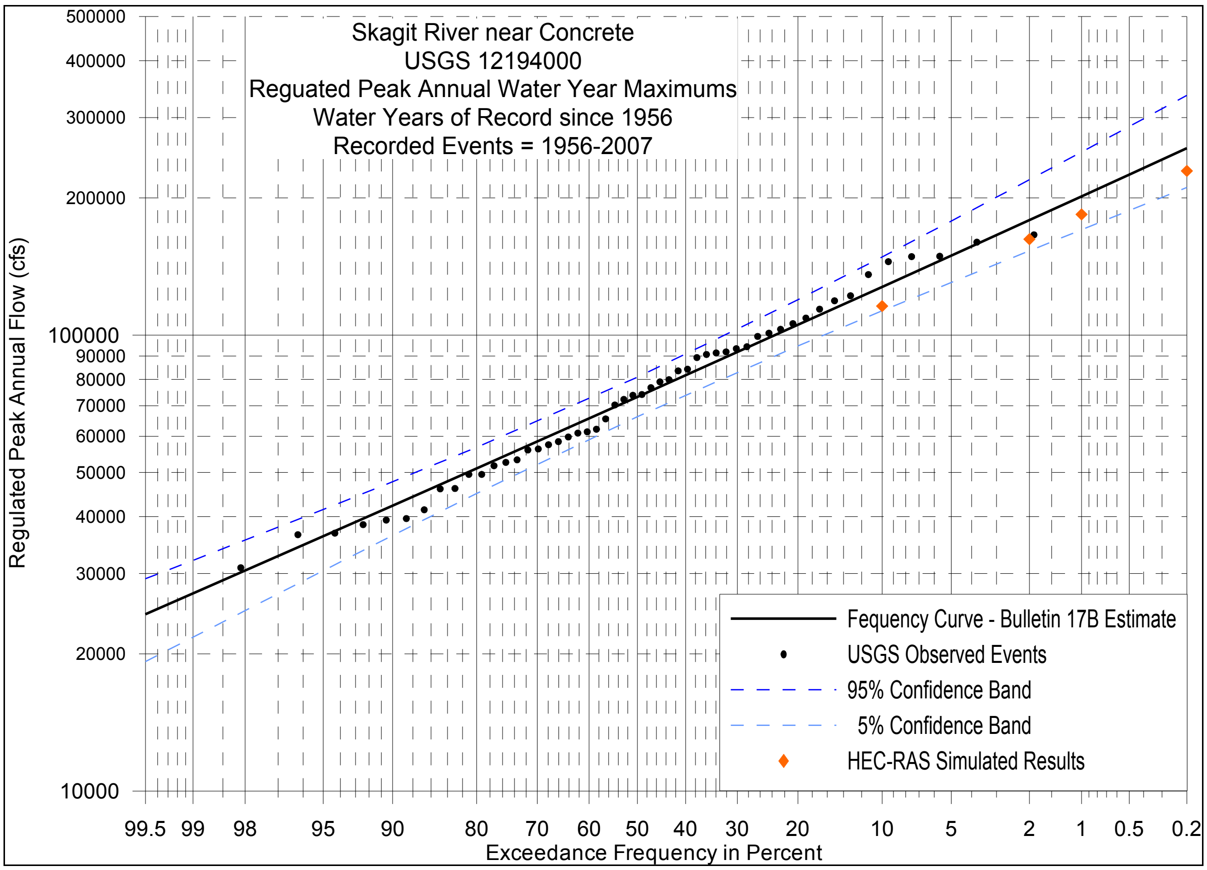

Figure A-4. Flood frequency curve for regulated peak discharges observed by USGS at Concrete, compared with the HEC-RAS simulated regulated peak flows at Concrete for the 10-, 50-, 100-, and 500-year synthetic events

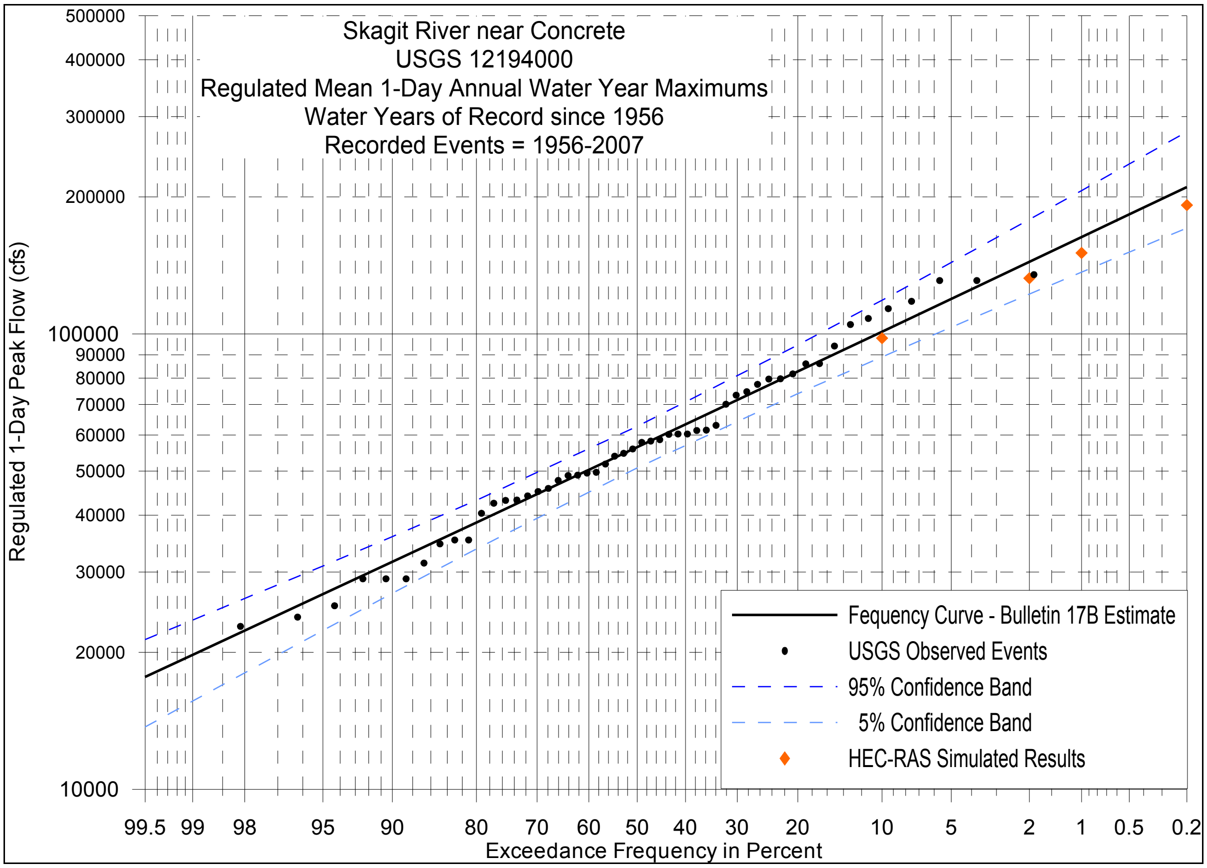

Figure A-5. Flood frequency curve for regulated one-day discharges observed by USGS at Concrete, compared with the HEC-RAS simulated regulated one-day flows at Concrete for the 10-, 50-, 100-, and 500-year synthetic events

2.0 Synthetic Flood Hydrographs at Mount Vernon

This section presents information on development of the regulated synthetic flood hydrographs routed by the HEC-RAS model originally developed by the Corps and later improved by PI Engineering along the Skagit River system from Concrete to Mount Vernon. Local coincident inflow hydrographs developed by the Corps were adjusted and used in the flood routing. A flood frequency based on USGS observed regulated events at Mount Vernon was developed and compared with HEC-RAS modeled results.

The majority of flood damages in the Skagit River floodplain occur below Concrete, primarily from Sedro-Woolley to the mouths of the North and South Forks of the Skagit River. It is, therefore, important that the flood modeled results match reasonably well with flood projections based on observed flood records available from USGS at the Mount Vernon gage. The Mount Vernon gage, USGS Station No. 12200500, provides the longest systematic flow record below Concrete (1941 to present).

2.1 Local Inflows below Concrete

The coincident local inflow hydrographs developed by the Corps for synthetic flood events from Concrete to Sedro-Woolley [see Section 5.1 of the Corps' Draft Report – Skagit River Basin, Washington, Revised Flood Insurance Study, Hydrology Summary (Corps 2005 and 2008)] were used in development of the synthetic flood hydrographs at Mount Vernon. This data represents flow contribution from the intermediate drainage area of 278 square miles between Concrete and Sedro-Woolley.

The coincident local inflow hydrographs developed by the Corps for the 71.6-square-mile Nookachamps Creek [see Section 5.2 of the Corps' Draft Report – Skagit River Basin, Washington, Revised Flood Insurance Study, Hydrology Summary (Corps 2005 and 2008)] were not used. Instead, the coincident local inflow hydrographs developed by the Corps for the 51.6-square-mile Finney Creek [see Section 5.1 of the Corps' Draft Report – Skagit River Basin, Washington, Revised Flood Insurance Study, Hydrology Summary (Corps 2005 and 2008)] were used with a direct proportional adjustment of the drainage area to represent the flow contribution from Nookachamps Creek. The Corps-developed regression for the Nookachamps Creek drainage area is a weak correlation, while the Corps-developed flow regression for Finney Creek is a better correlation. Finney Creek is located on the left bank of the Skagit River, the same side as Nookachamps Creek; and, the size and location of Finney and Nookachamps Creeks are similar.

Table A-2 lists the one-day coincident flows for the local drainage areas below Concrete, and the unregulated one-day flows at Concrete for the 10-, 50-, 100-, and 500-year synthetic floods analyzed.

Table A-2. Unregulated synthetic flood one-day coincident flows (cfs) for lower Skagit River subbasins

Location	Flood Event			
	10-year	50-year	100-year	500-year
Unregulated Skagit River Near Concrete	123,700	177,900	201,400	257,500
Concrete to Sedro-Woolley Local	11,700	16,800	19,200	24,300
Nookachamps Creek	2,800	4,000	4,600	5,800

2.2 Routing of Regulated Flood Hydrographs below Concrete

The regulated flood hydrographs at Concrete for the 10-, 50-, 100-, and 500-year synthetic events, derived as described above, were routed downstream along the Skagit River to the mouths of the North and South Forks of the Skagit River, using the PI Engineering improved HEC-RAS model. Local inflows as discussed above were added to the routing as necessary. It was assumed that there was no levee failure below Concrete, and no levee overtopping below Sedro-Woolley. Details of the HEC-RAS improvements are discussed in the Draft Technical Memorandum – Skagit River Basin Historical Flood Modeling – Hydraulics (PI Engineering, 2004).

The HEC-RAS routed peak and one-day flows for the 10-, 50-, 100-, and 500-year floods at Sedro-Woolley (RM 22.40) and Mount Vernon (RM 17.05) are listed in Table A-3. The regulated peak and one-day values at Concrete (RM 54.15) are also listed in Table A-3 for a comparison.

Table A-3. Peak and one-day flows (cfs) at Concrete, Sedro-Woolley and Mount Vernon for regulated synthetic floods

Flood	Concrete (RM 54.15)		Sedro-Woolley (RM 22.40)		Mount Vernon (RM 17.05)	
	Peak	One-Day	Peak	One-Day	Peak	One-Day
10-year	116,100	98,200	117,200	105,500	108,200	103,400
50-year	162,600	133,000	161,900	141,400	143,500	135,100
100-year	184,400	151,000	184,700	160,000	162,200	152,400
500-year	229,400	192,500	231,700	203,200	195,700	184,500

Figure A-6 and Figure A-7 present regressions of the USGS observed peak and one-day flows, respectively, at Concrete and Mount Vernon for the time period from 1955 through 2006 (water years 1956-2007), representing regulated conditions of the Skagit River. The HEC-RAS modeled peak and one-day values for the 10-, 50-, 100-, and 500-year events are also shown in these two figures, indicating a reasonable match of the HEC-RAS modeled results and the USGS observed data. The modeled values appear to be slightly conservative.

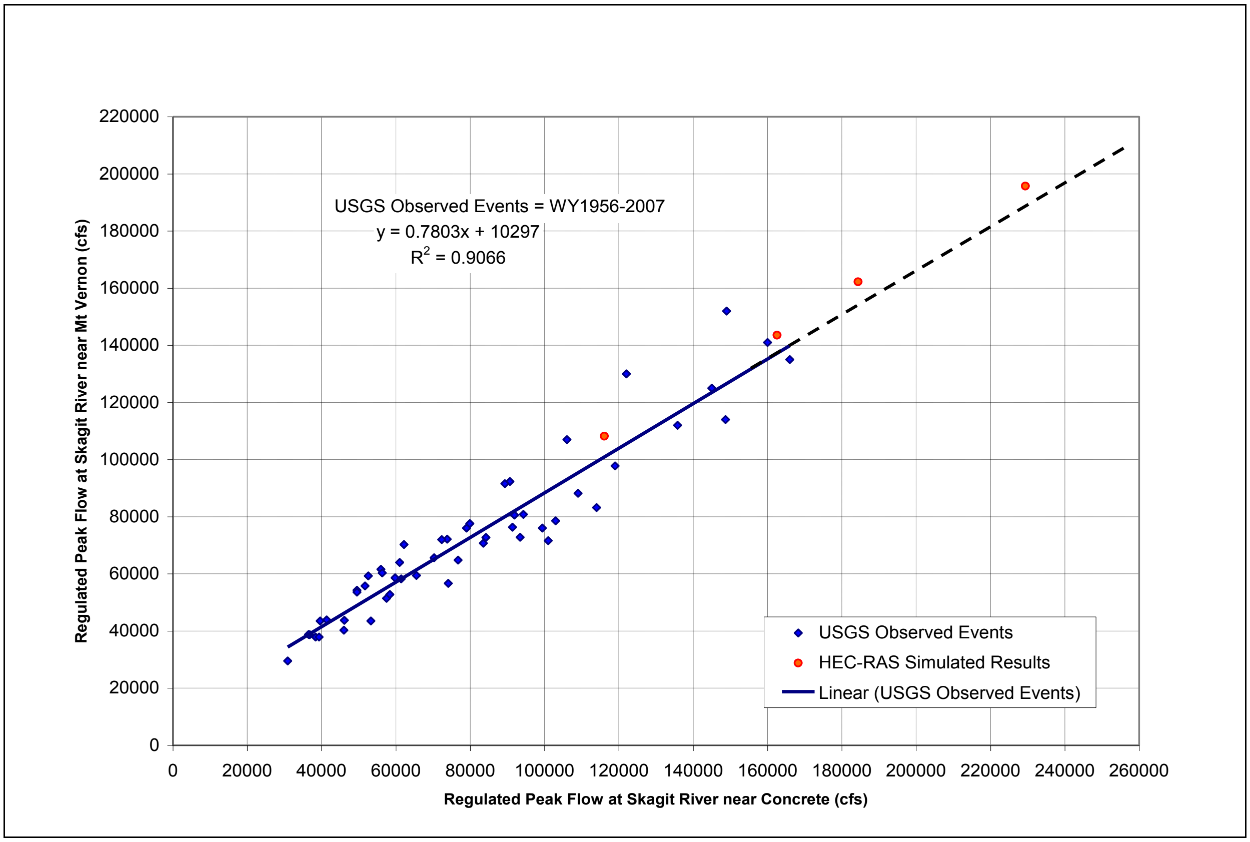

Figure A-6. Regression of regulated peak flows observed by USGS at Concrete and Mount Vernon, compared with the HEC-RAS simulated peak values for the 10-, 50-, 100-, and 500-year synthetic events

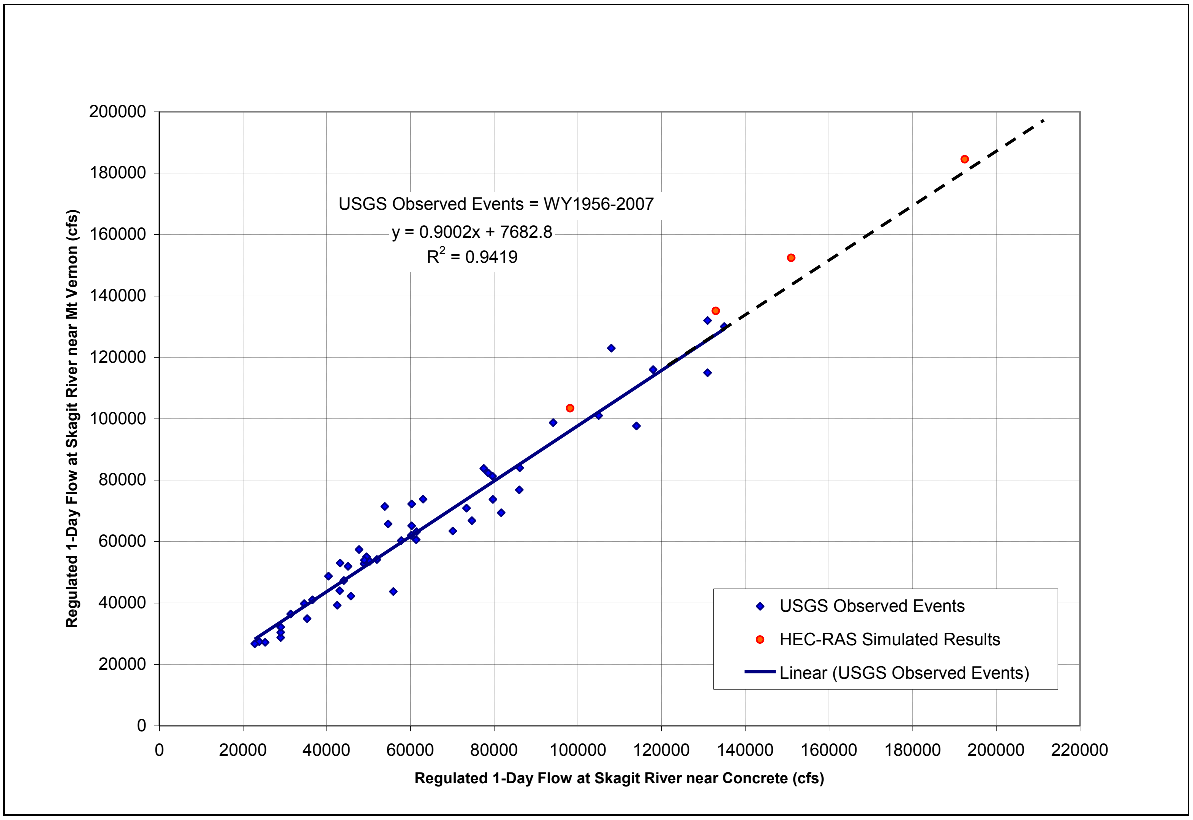

Figure A-7. **Regression of the regulated one-day flows observed by USGS at Concrete and Mount Vernon, compared with the HEC-RAS simulated one-day values for the 10-, 50-, 100-, and 500-year synthetic events**

2.3 Flood Frequency Curves at Mount Vernon

Figure A-8 and Figure A-9 show the annual peak and one-day flood frequency curves, respectively, at Mount Vernon. These frequency curves were based on the USGS observed flow data at the Mount Vernon gage for the time period from 1955 through 2006 (water years 1956-2007), representing regulated conditions of the Skagit River system. The HEC-RAS modeled peak and one-day flows at Mount Vernon for the 10-, 50-, 100-, and 500-year events were also plotted in Figure A-8 and Figure A-9 for a comparison with the USGS observed annual flood data and the calculated flood frequency curves. The comparison indicates that the modeled synthetic floods compare well with projection of the frequency curves based on the observed events at Mount Vernon.

2.4 Regulated Synthetic Flood Hydrographs

The HEC-RAS modeled flood hydrographs for the regulated 10-, 50-, 100-, and 500-year synthetic floods at Concrete, Sedro-Woolley and Mount Vernon are presented in Figure A-10.

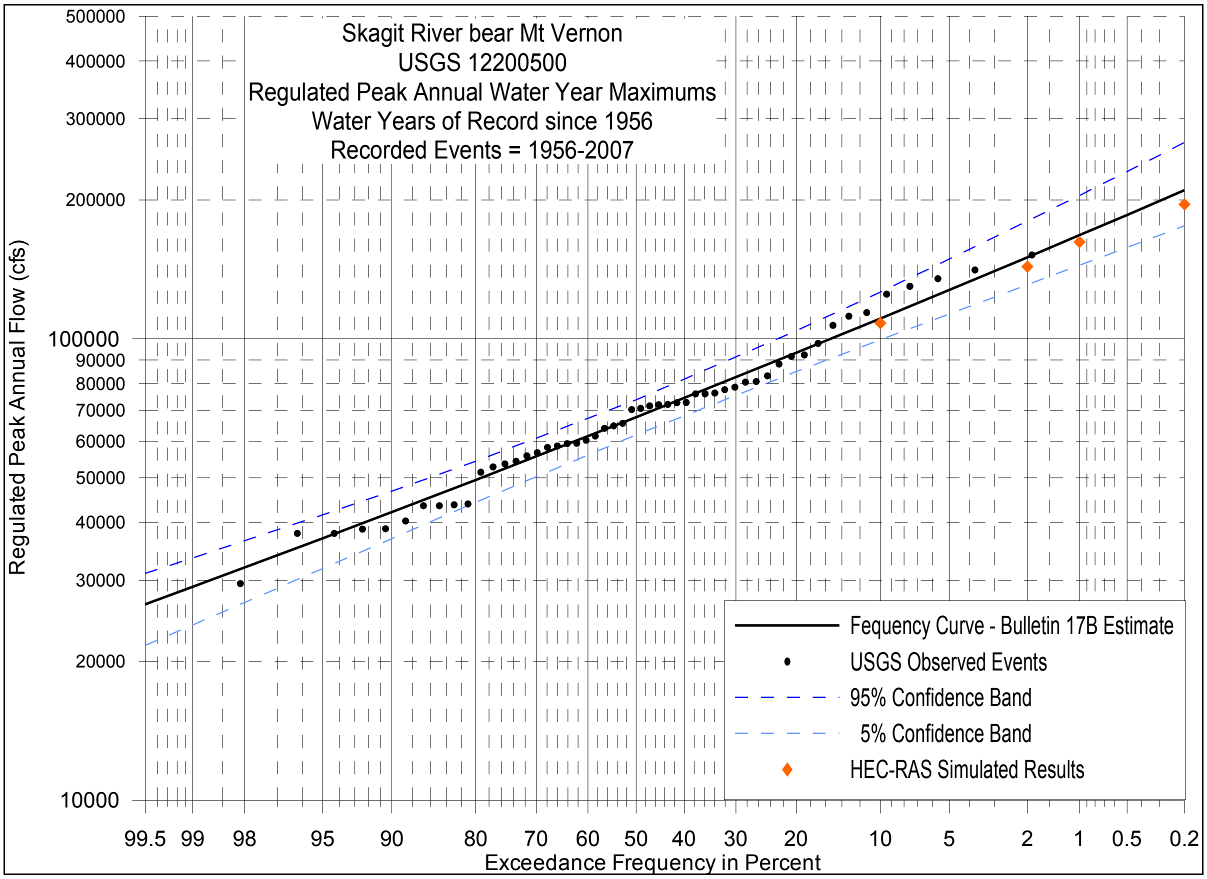

Figure A-8. Flood frequency curve for regulated peak discharges observed by USGS at Mount Vernon, compared with the HEC-RAS simulated peak flows at Mount Vernon for the 10-, 50-, 100-, and 500-year synthetic events

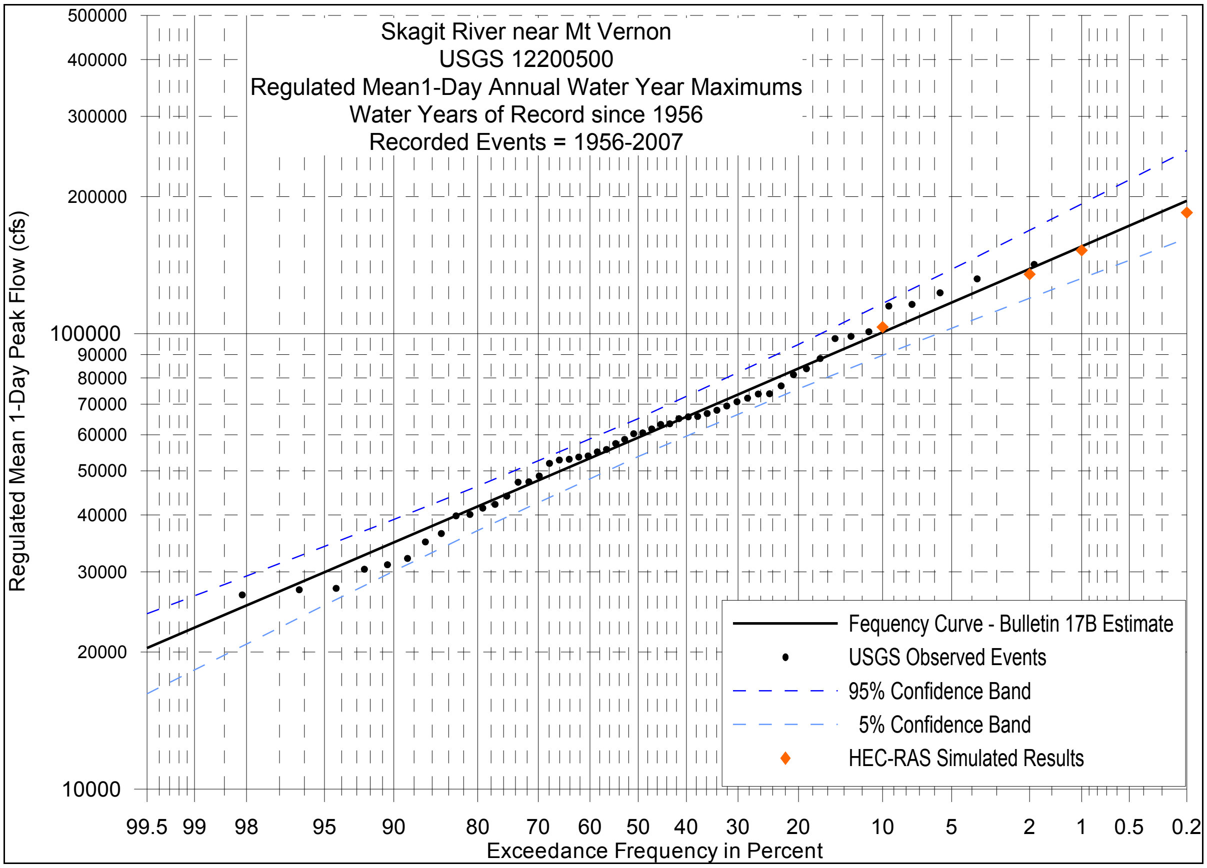

Figure A-9. Flood frequency curves for regulated one-day discharges observed by USGS at Mount Vernon, compared with the HEC-RAS simulated one-day flows at Mount Vernon for the 10-, 50-, 100-, and 500-year synthetic events

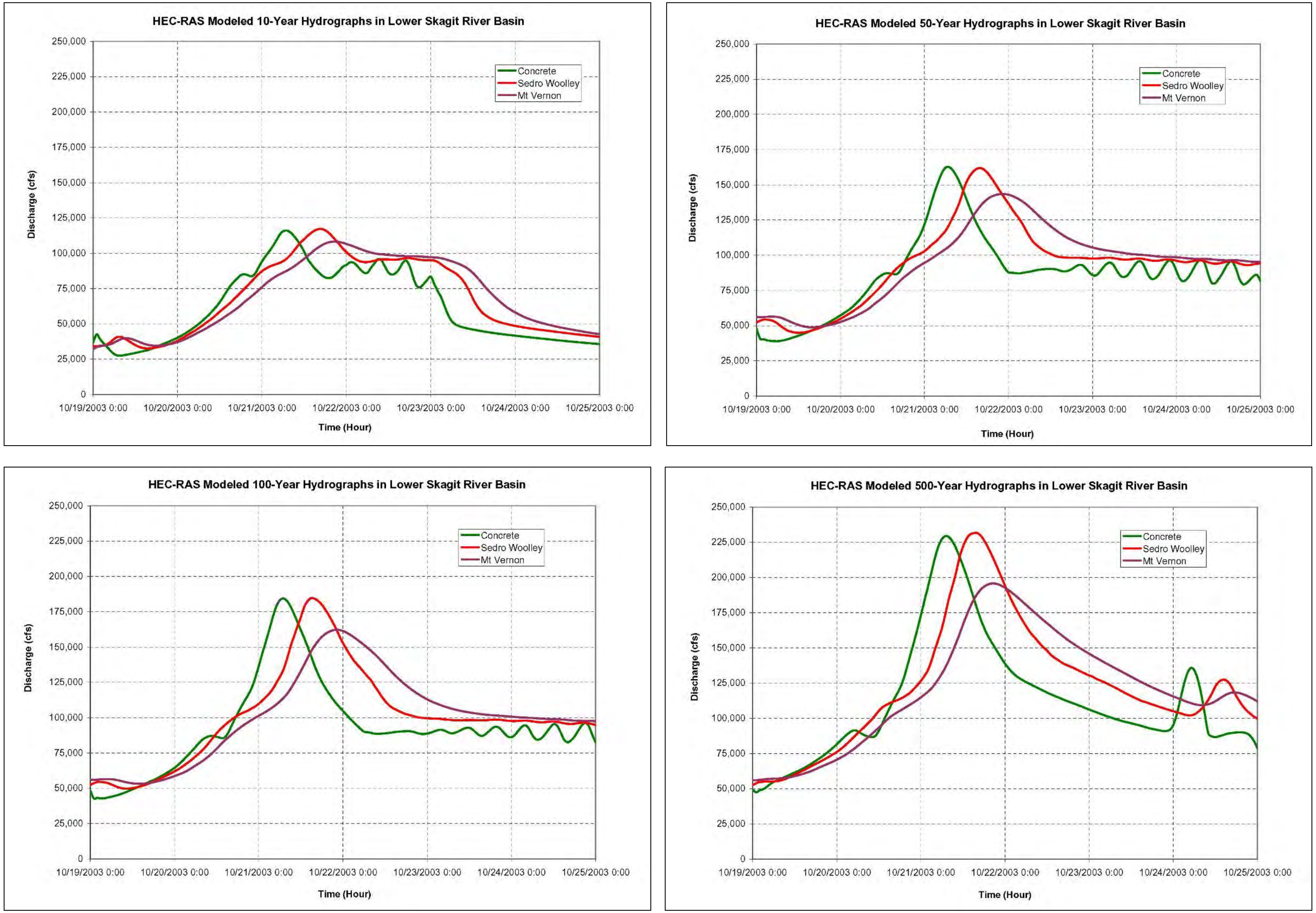

Figure A-10. Regulated flood hydrographs at Concrete, Sedro-Woolley, and Mount Vernon for 10-, 50-, 100-, and 500-year synthetic floods

3.0 List of References

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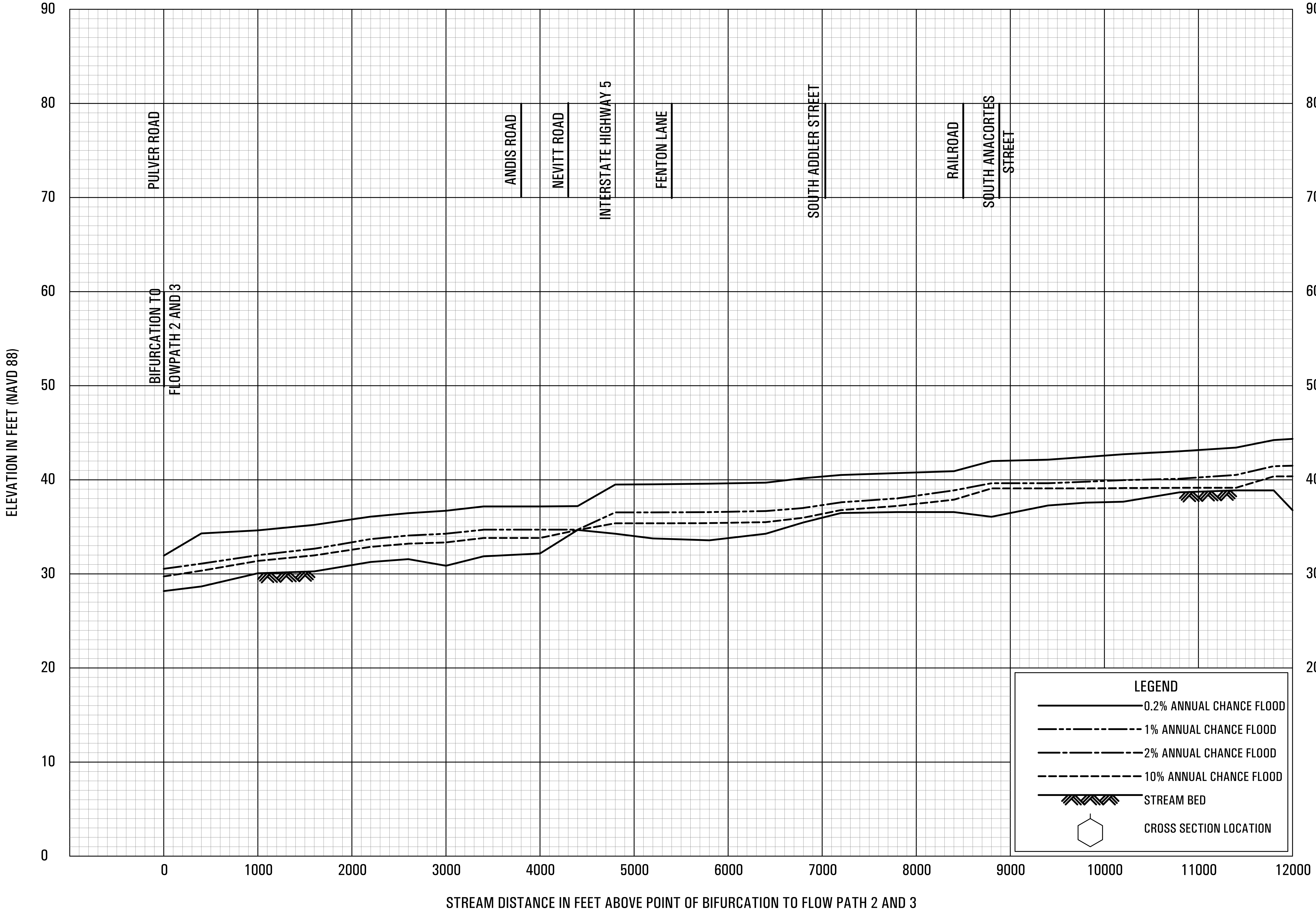
APPENDIX B

Revised Flood Profiles

Skagit River Delta Overbank Flowpath 1 Panels 20P – 21P
Skagit River Delta Overbank Flowpath 2 Panels 22P – 25P
Skagit River Delta Overbank Flowpath 3 Panels 26P – 28P
North Fork Skagit River Panels 29P – 31P
South Fork Skagit River Panels 32P – 33P
Main Stem Skagit River Panels 34P – 53P
Baker River Panel 01P

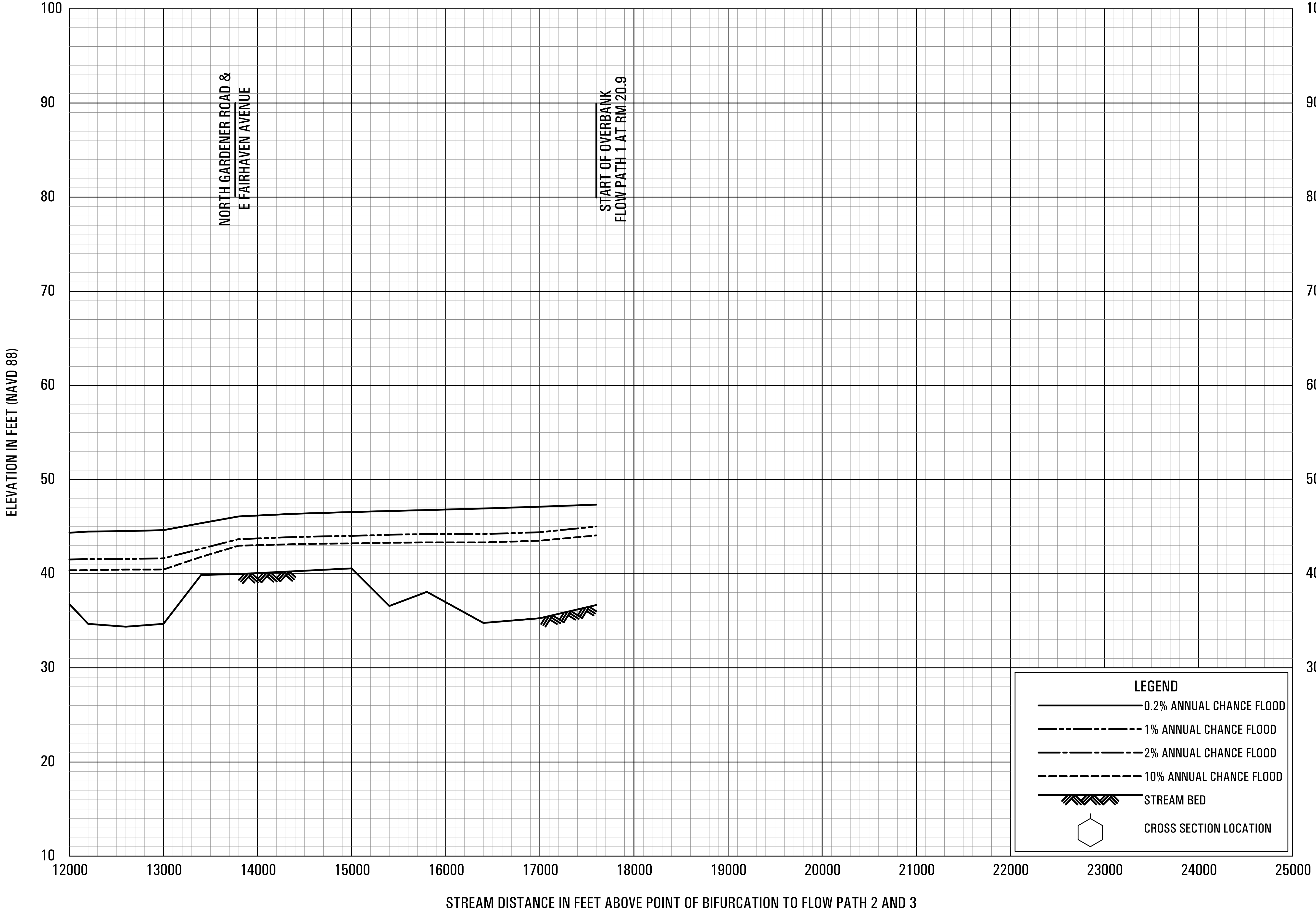


FLOOD PROFILES

SKAGIT RIVER DELTA, OVERBANK FLOW PATH 1

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA
AND INCORPORATED AREAS



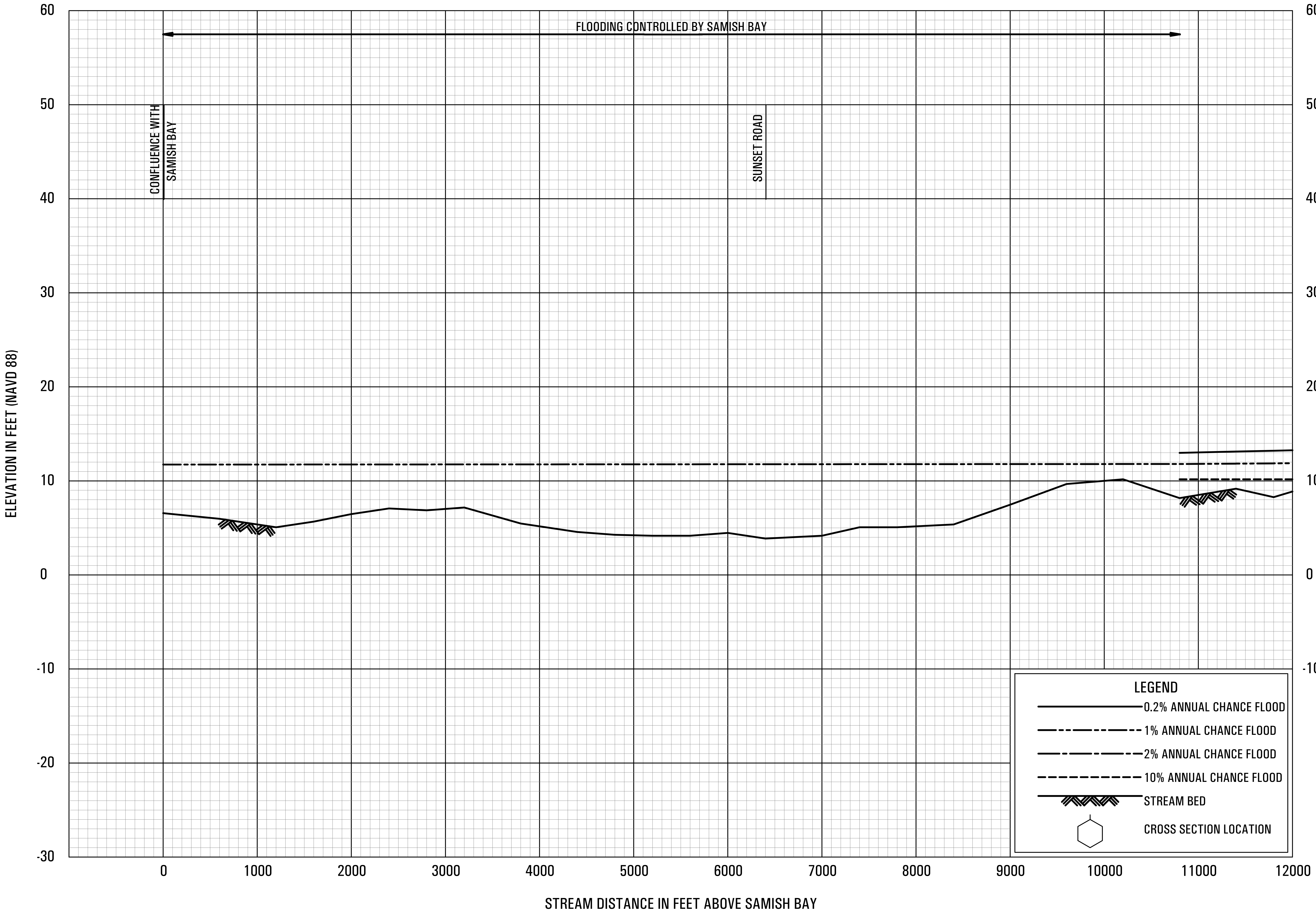
FLOOD PROFILES

SKAGIT RIVER DELTA, OVERBANK FLOW PATH 1

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



FLOOD PROFILES

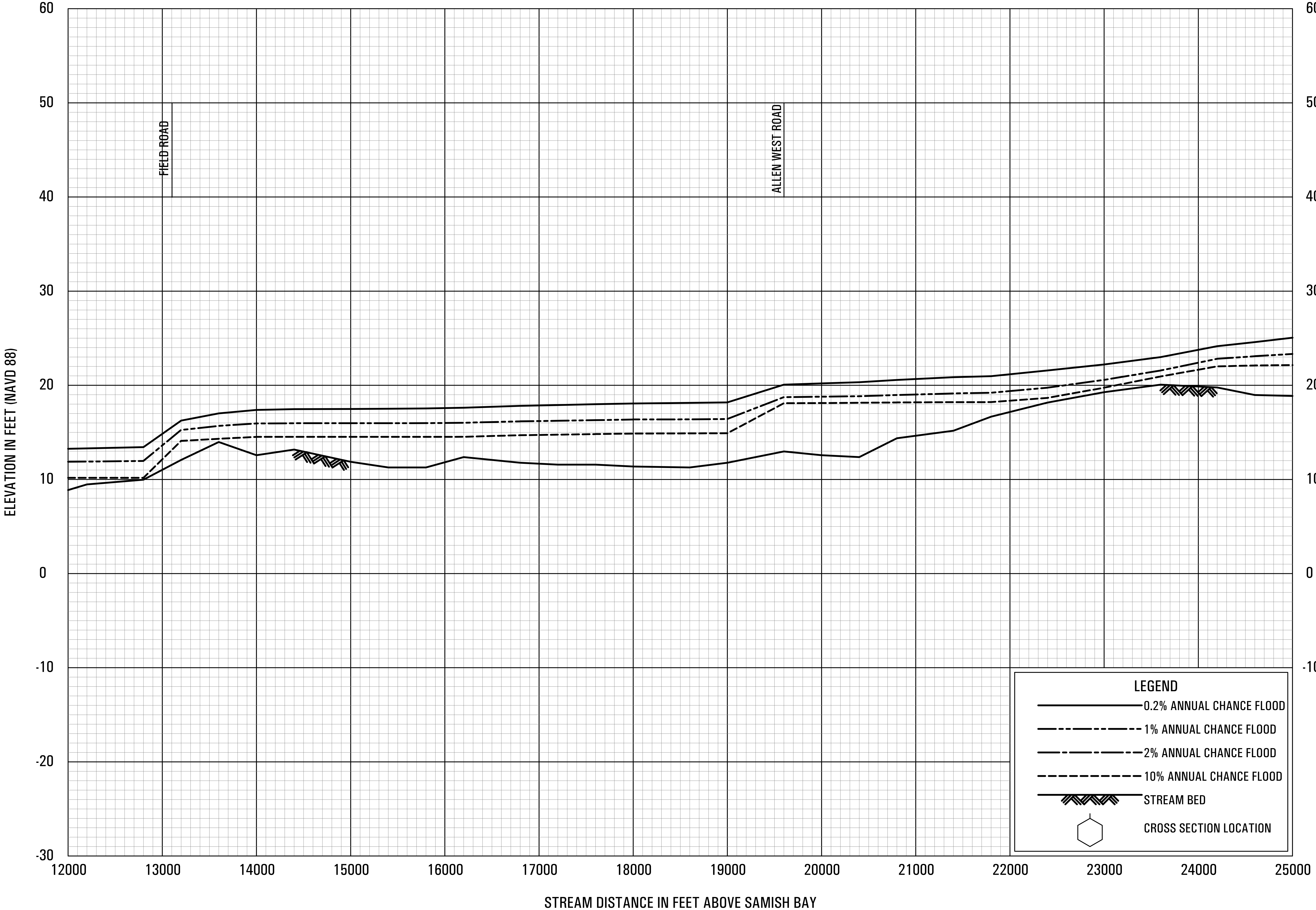
SKAGIT RIVER DELTA, OVERBANK FLOW PATH 2

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

22P



FLOOD PROFILES

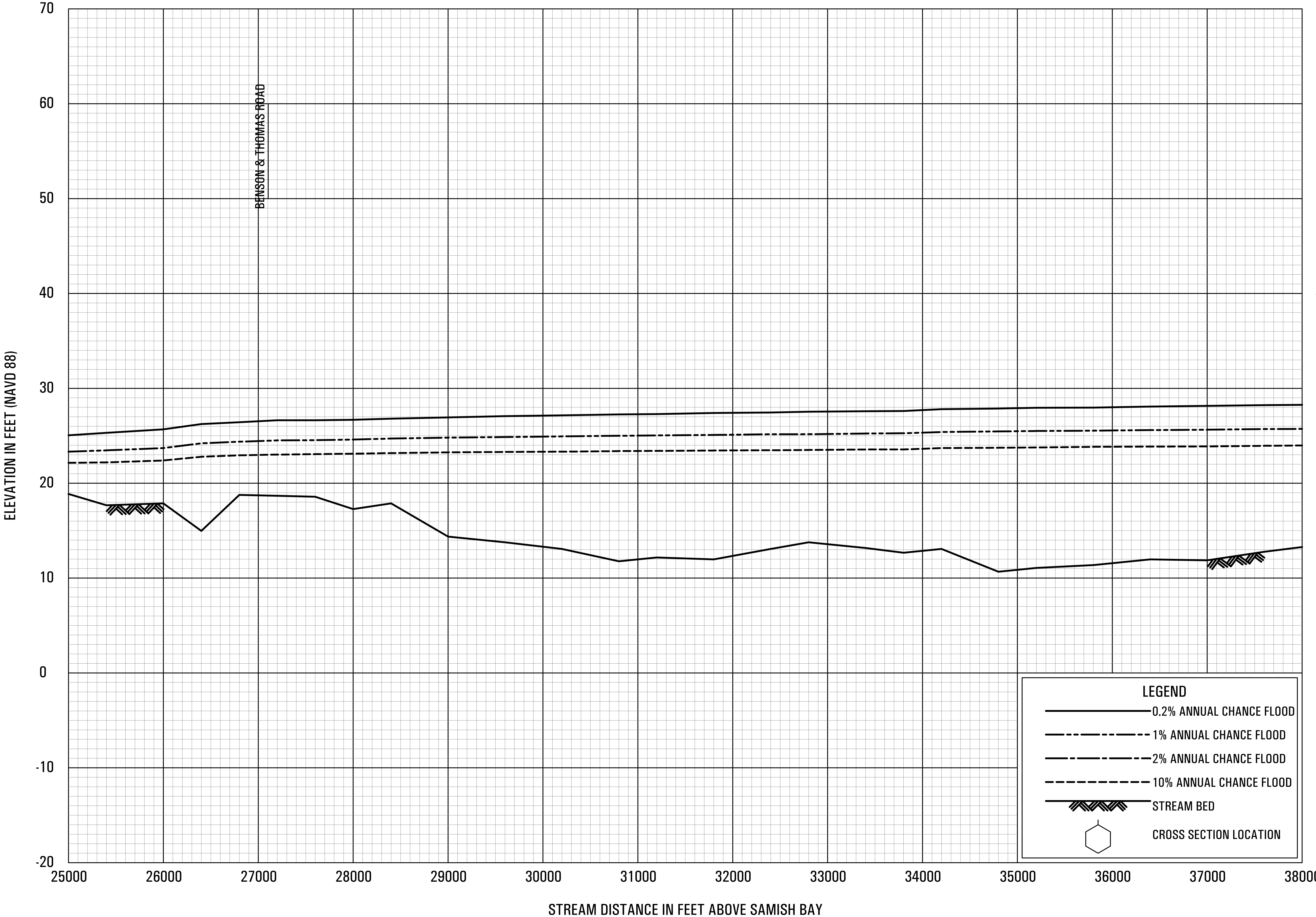
SKAGIT RIVER DELTA, OVERBANK FLOW PATH 2

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

23P



FLOOD PROFILES

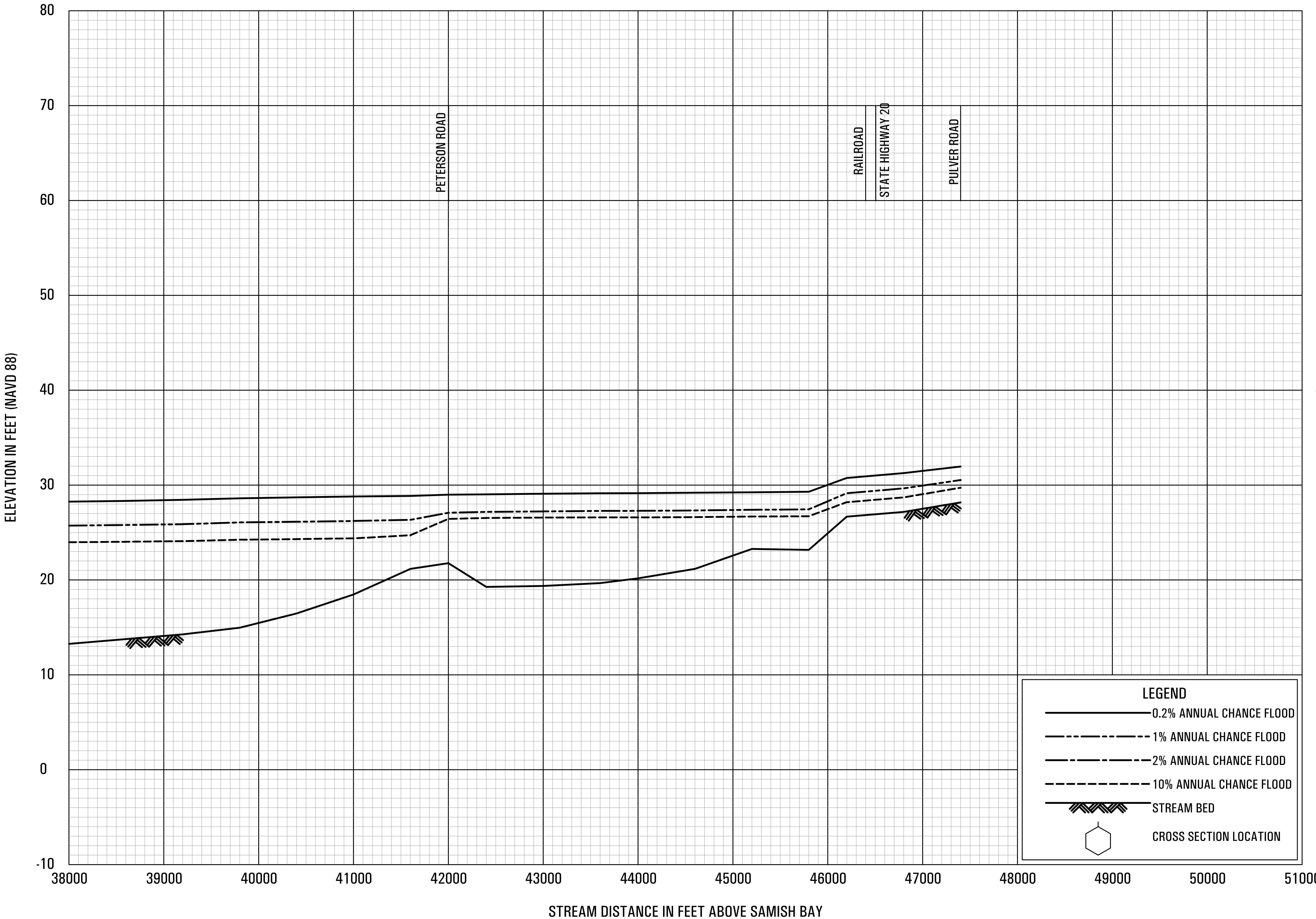
SKAGIT RIVER DELTA, OVERBANK FLOW PATH 2

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

24P

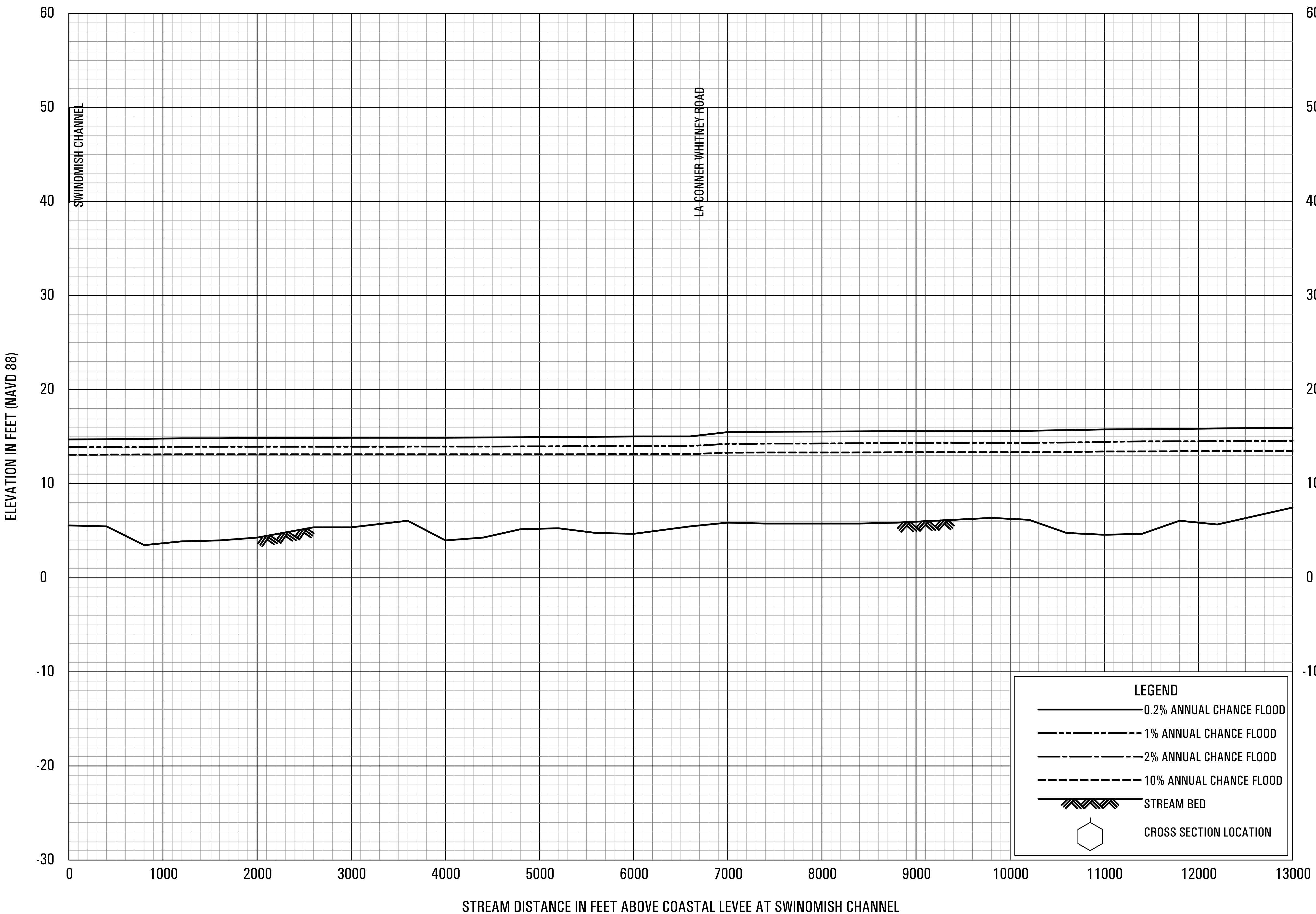


FLOOD PROFILES

SKAGIT RIVER DELTA, OVERBANK FLOW PATH 2

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA
AND INCORPORATED AREAS



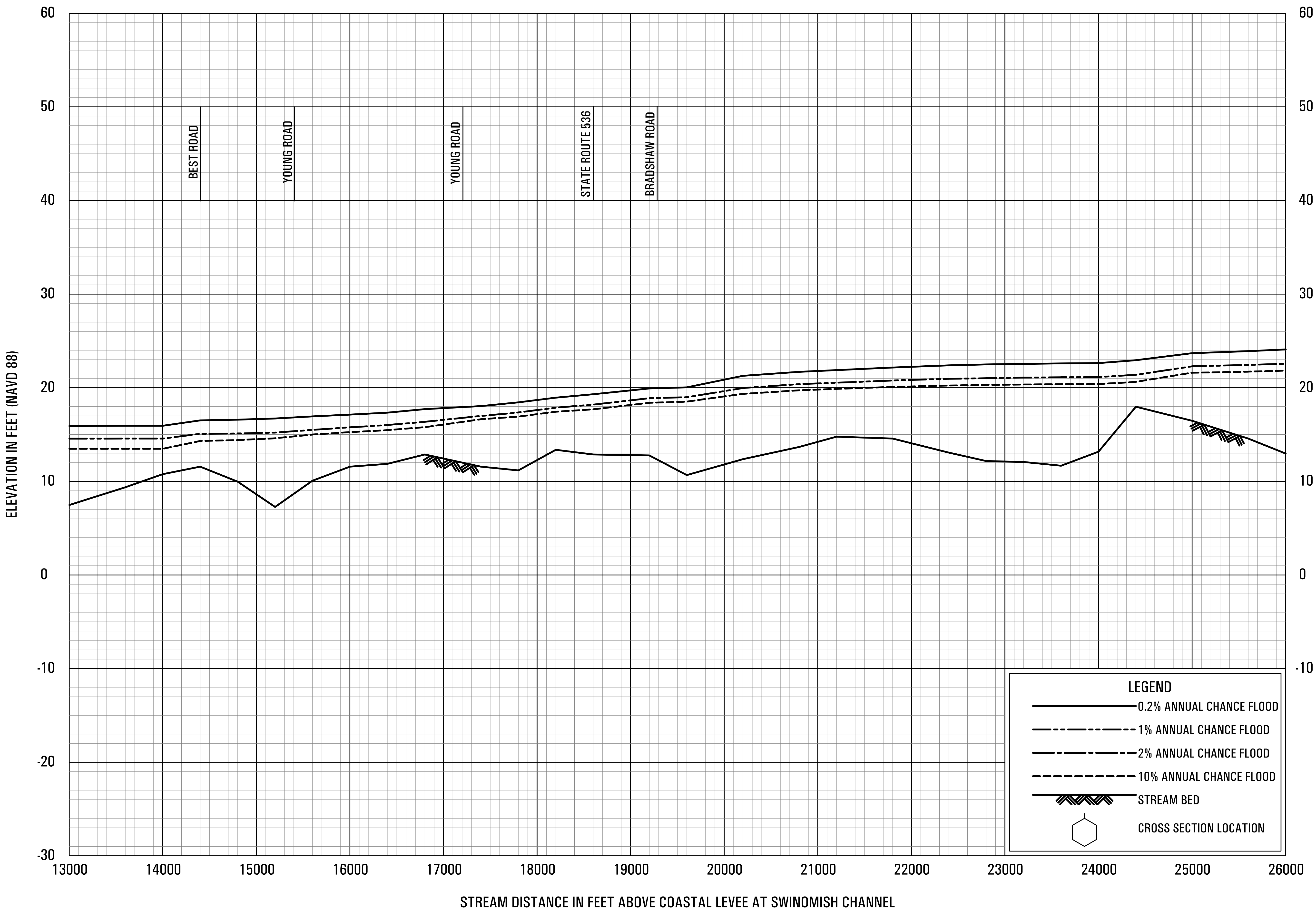
FLOOD PROFILES

SKAGIT RIVER DELTA, OVERBANK FLOW PATH 3

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



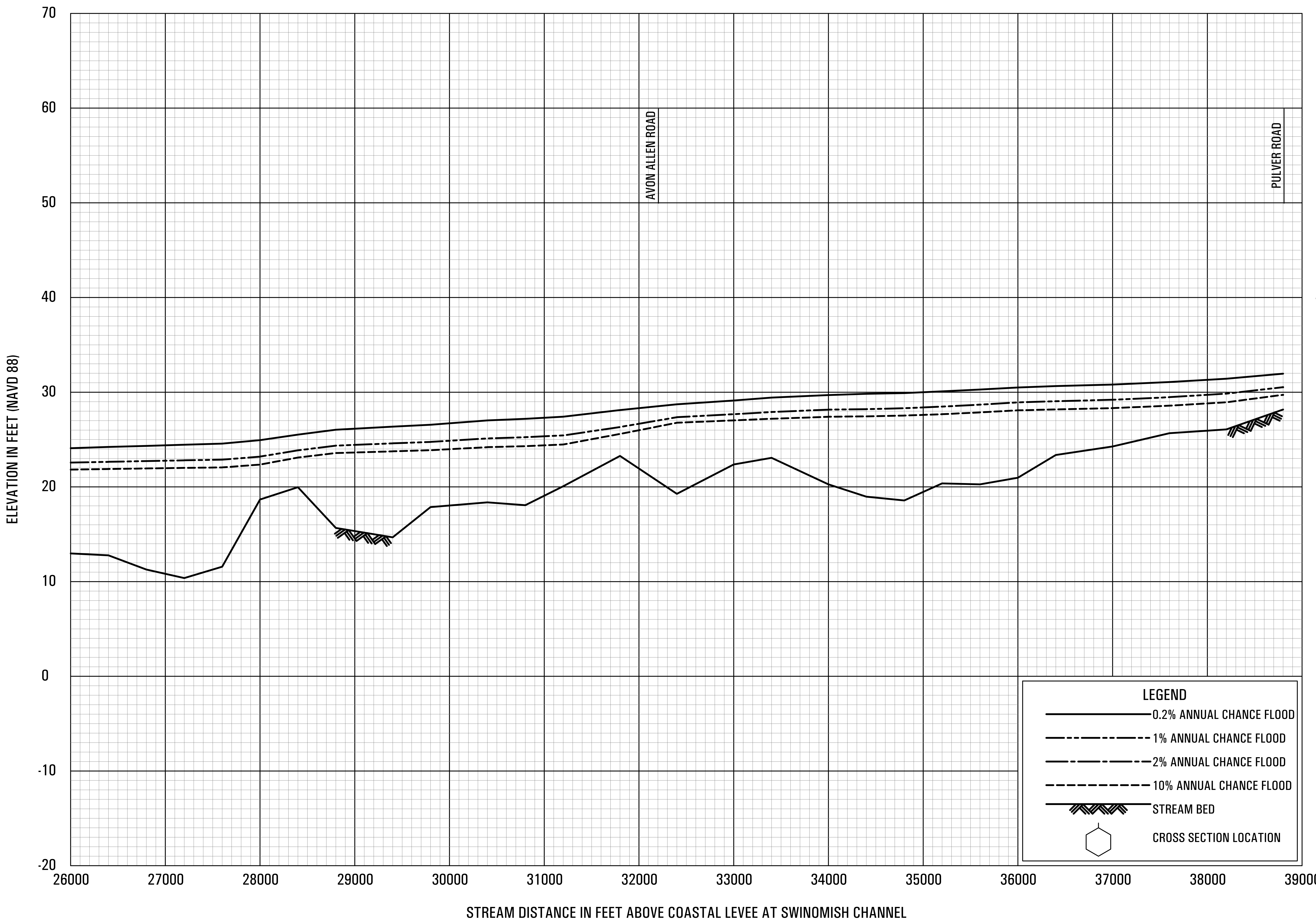
FLOOD PROFILES

SKAGIT RIVER DELTA, OVERBANK FLOW PATH 3

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



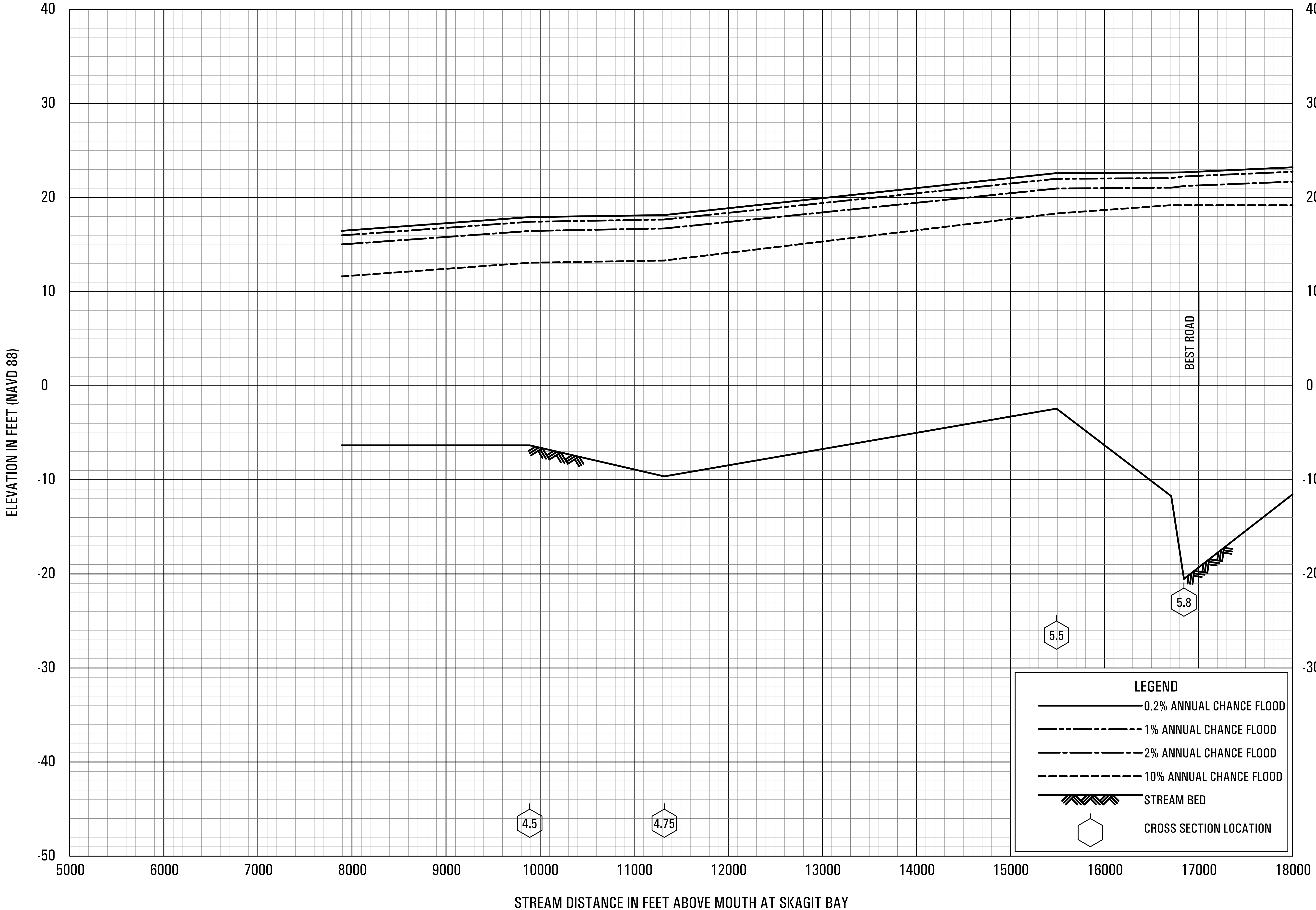
FLOOD PROFILES

SKAGIT RIVER DELTA, OVERBANK FLOW PATH 3

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



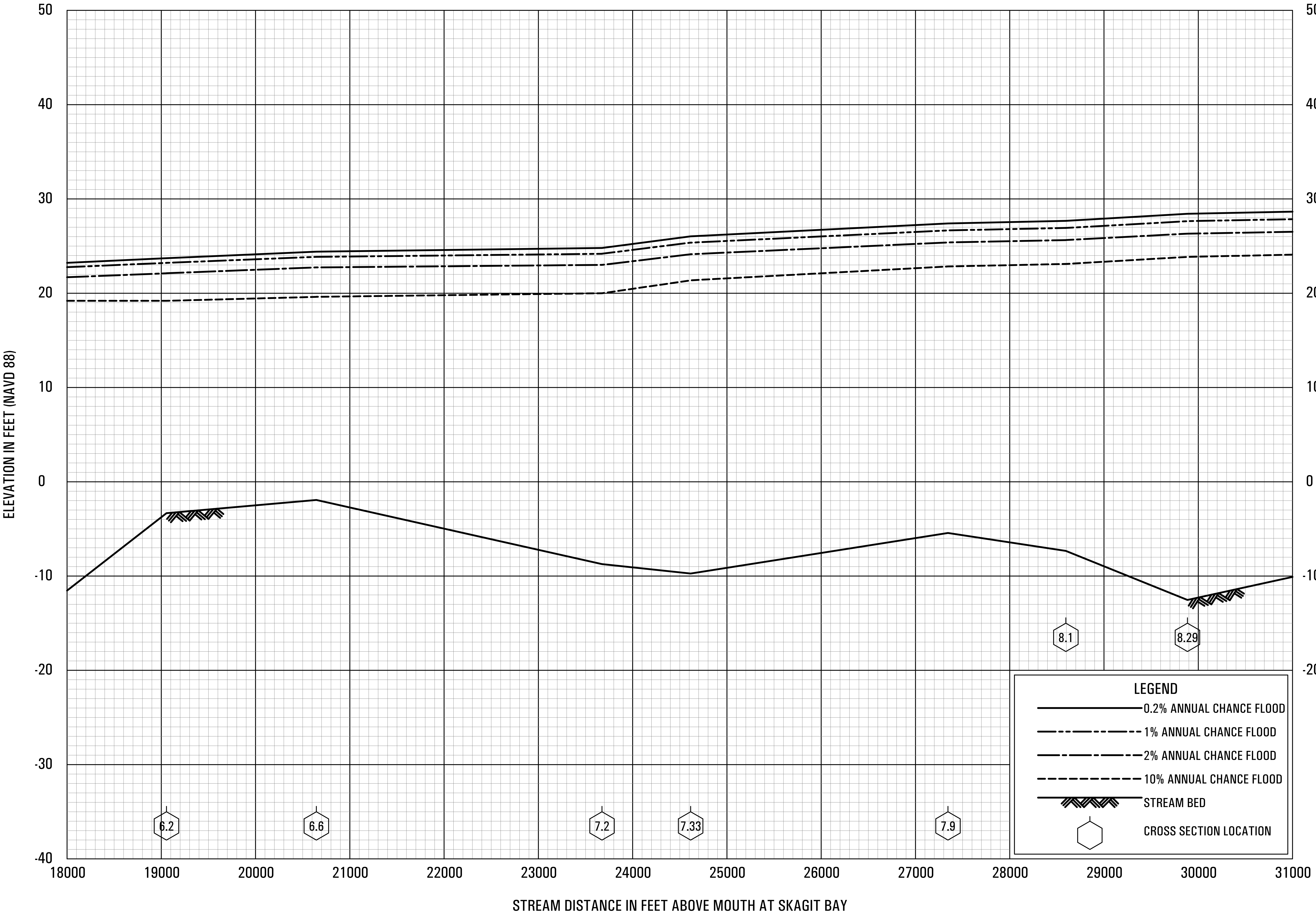
FLOOD PROFILES

NORTH FORK SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



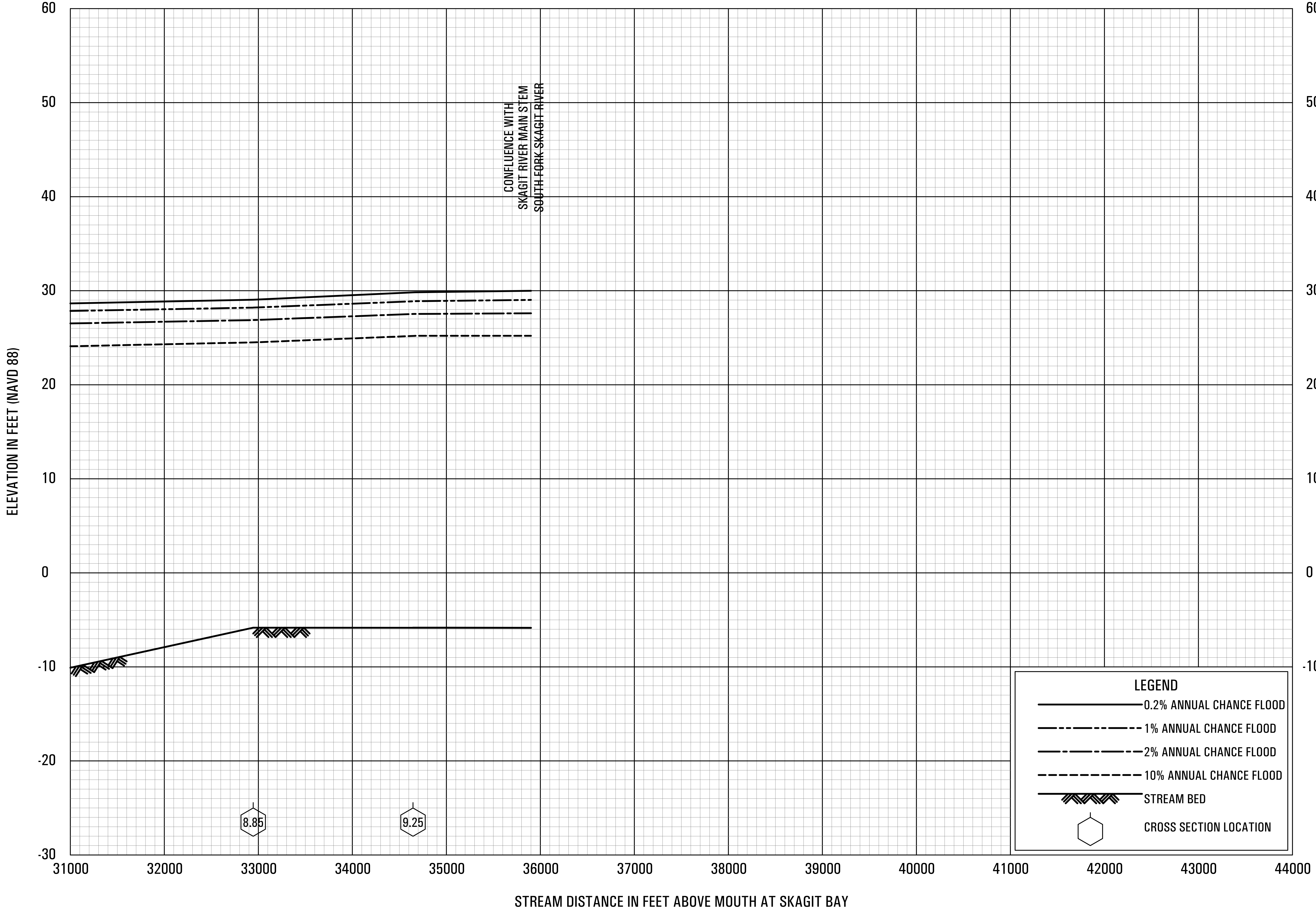
FLOOD PROFILES

NORTH FORK SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



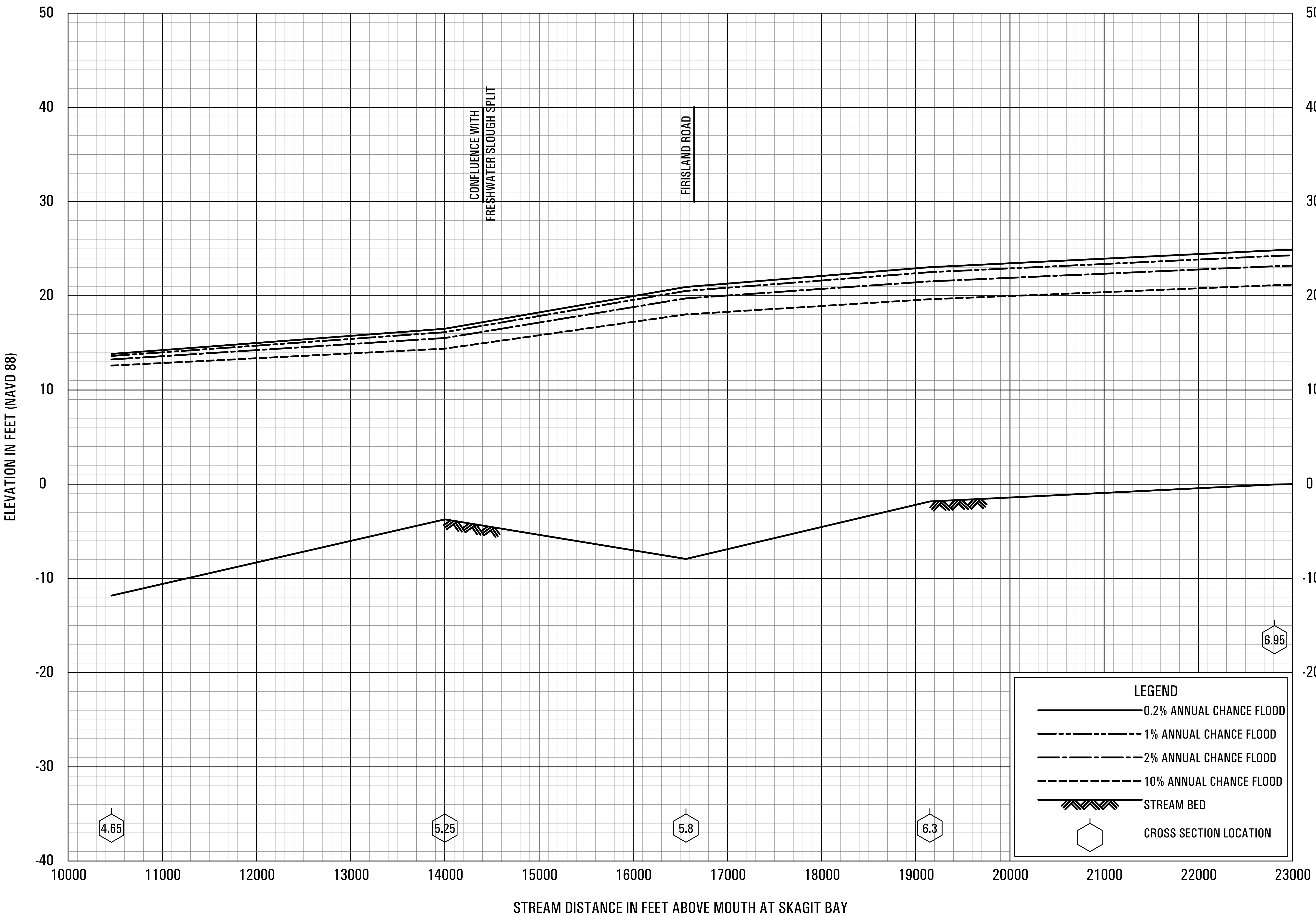
FLOOD PROFILES

NORTH FORK SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

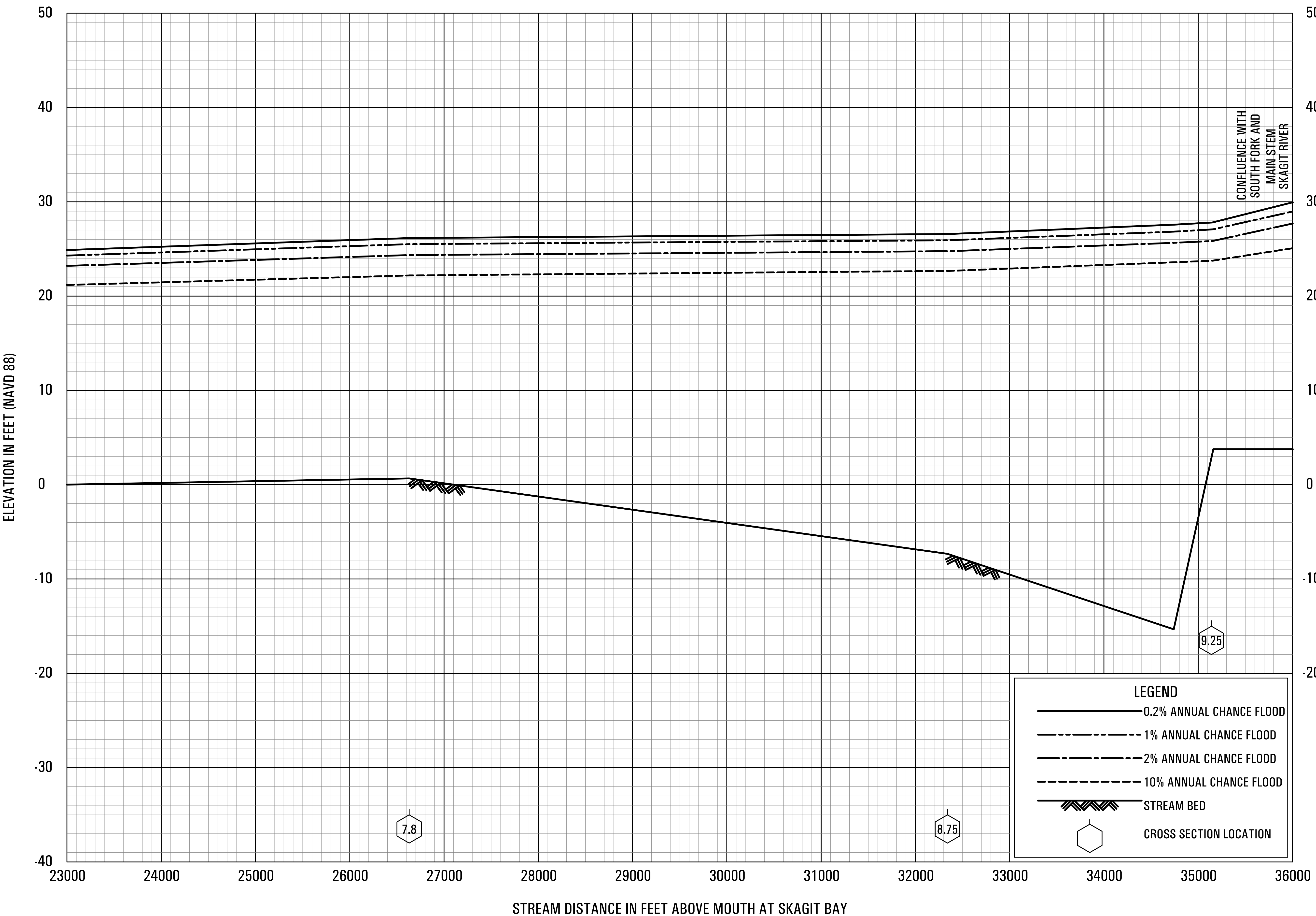


FLOOD PROFILES

SOUTH FORK SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA
AND INCORPORATED AREAS



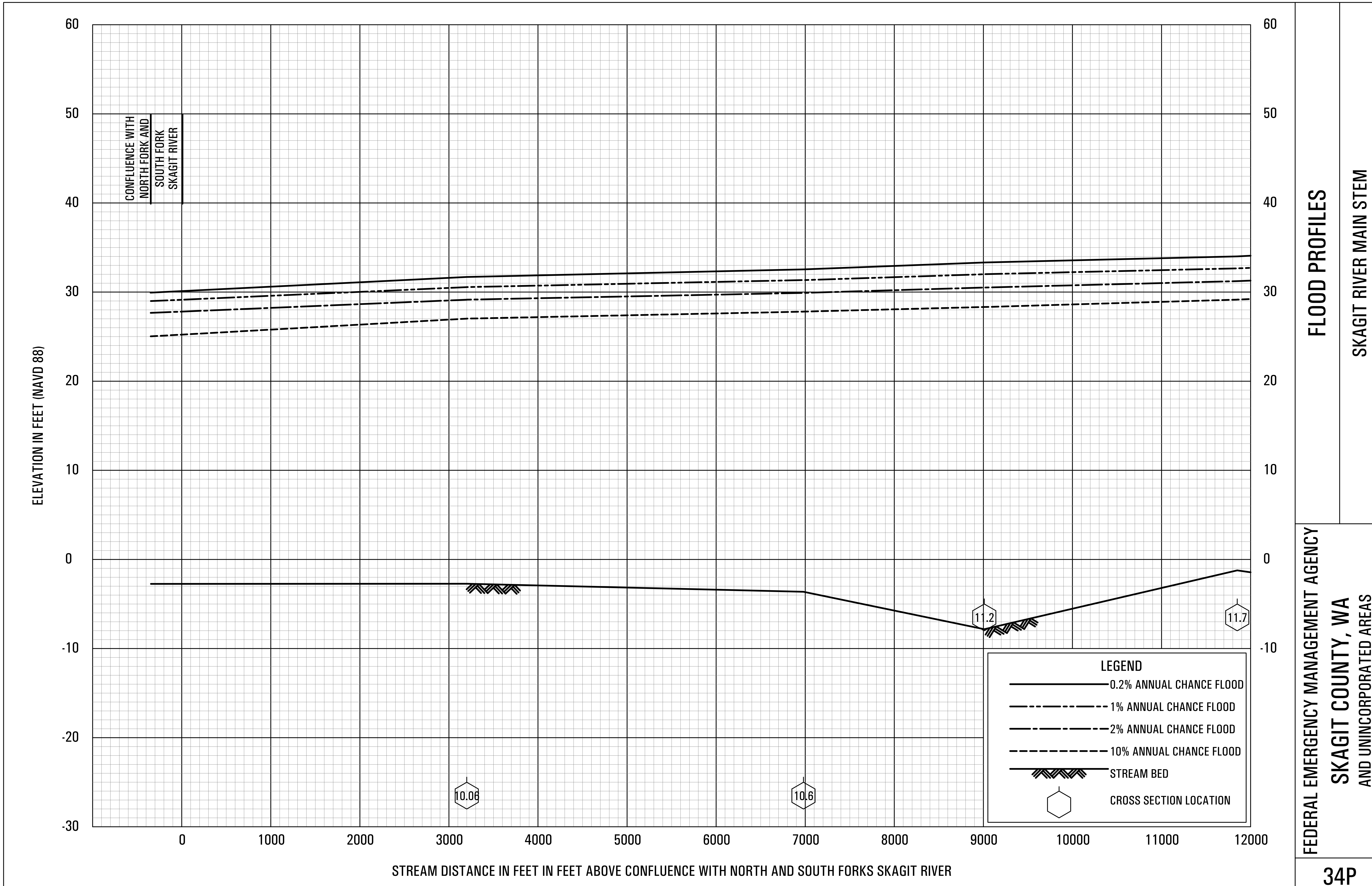
FLOOD PROFILES

SOUTH FORK SKAGIT RIVER

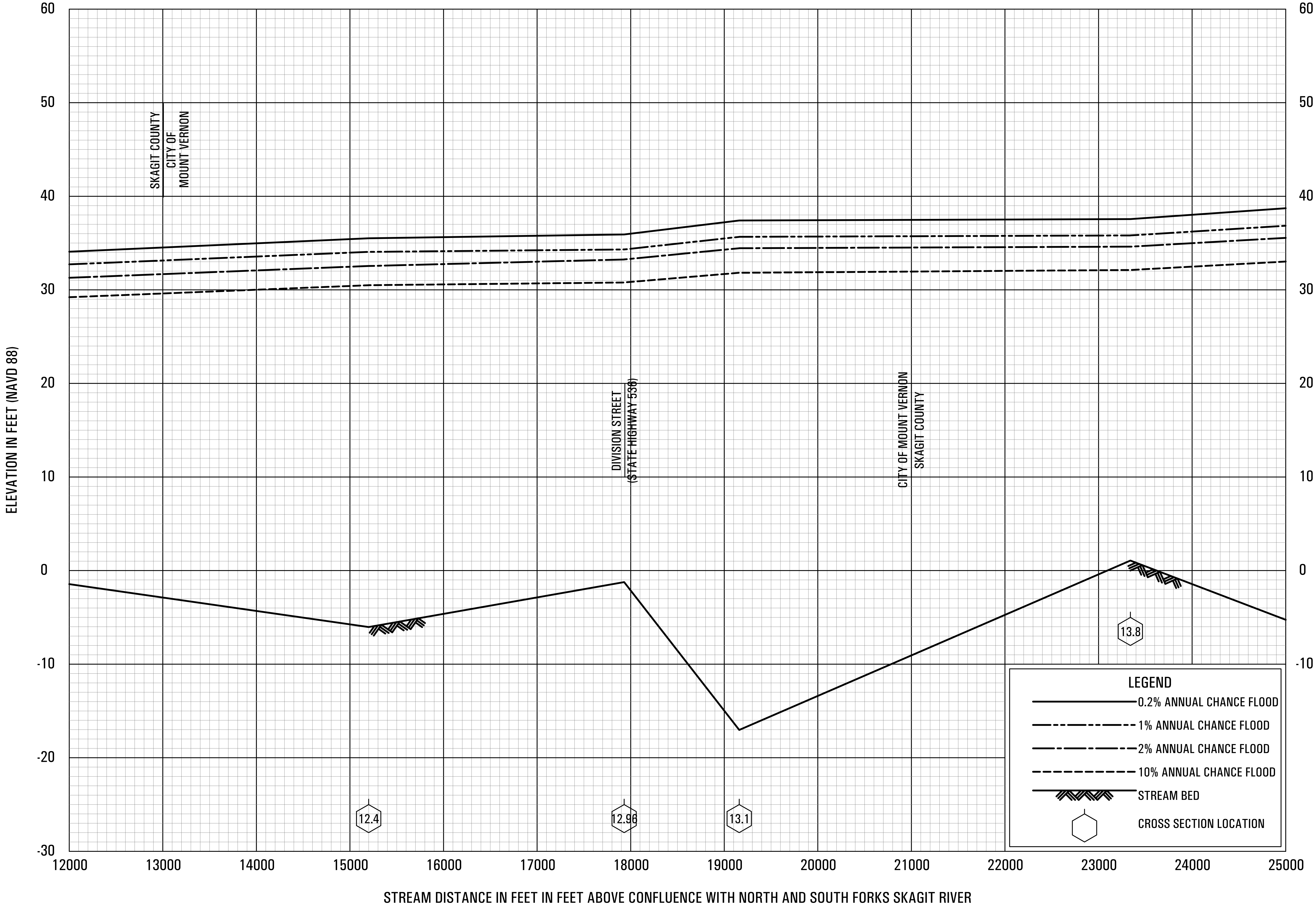
FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



FLOOD PROFILES
SKAGIT RIVER MAIN STEM
FEDERAL EMERGENCY MANAGEMENT AGENCY
SKAGIT COUNTY, WA
AND UNINCORPORATED AREAS
34P
ELEVATION IN FEET (NAVD 88)
STREAM DISTANCE IN FEET IN FEET ABOVE CONFLUENCE WITH NORTH AND SOUTH FORKS SKAGIT RIVER
CONFLUENCE WITH NORTH FORK AND SOUTH FORK SKAGIT RIVER
60
50
40
30
20
10
0
-10
-20
-30
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
11000
12000
10.06
10.6
11.2
11.7
LEGEND
0.2% ANNUAL CHANCE FLOOD
1% ANNUAL CHANCE FLOOD
2% ANNUAL CHANCE FLOOD
10% ANNUAL CHANCE FLOOD
STREAM BED
CROSS SECTION LOCATION



FLOOD PROFILES

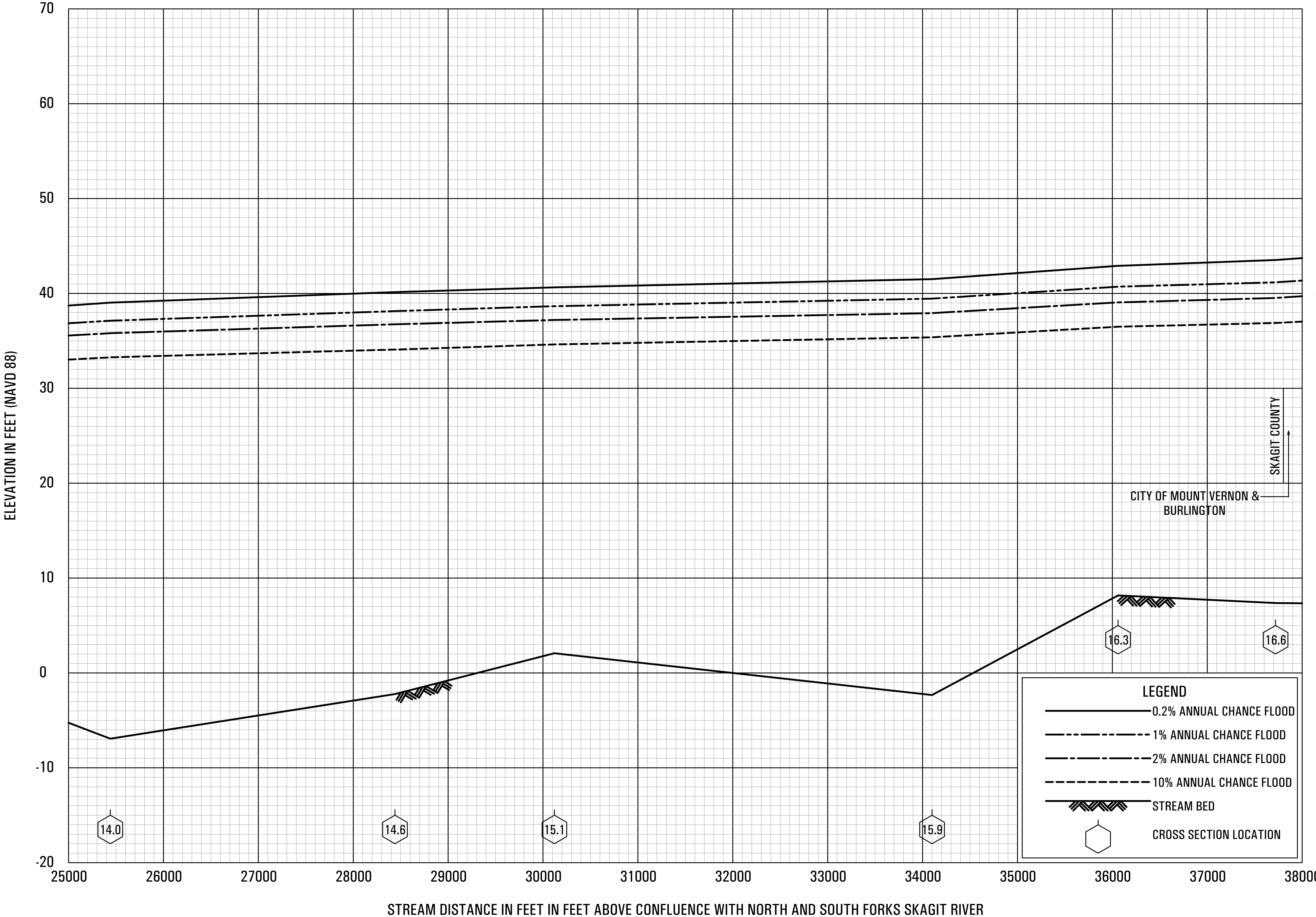
SKAGIT RIVER MAIN STEM

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND UNINCORPORATED AREAS

35P



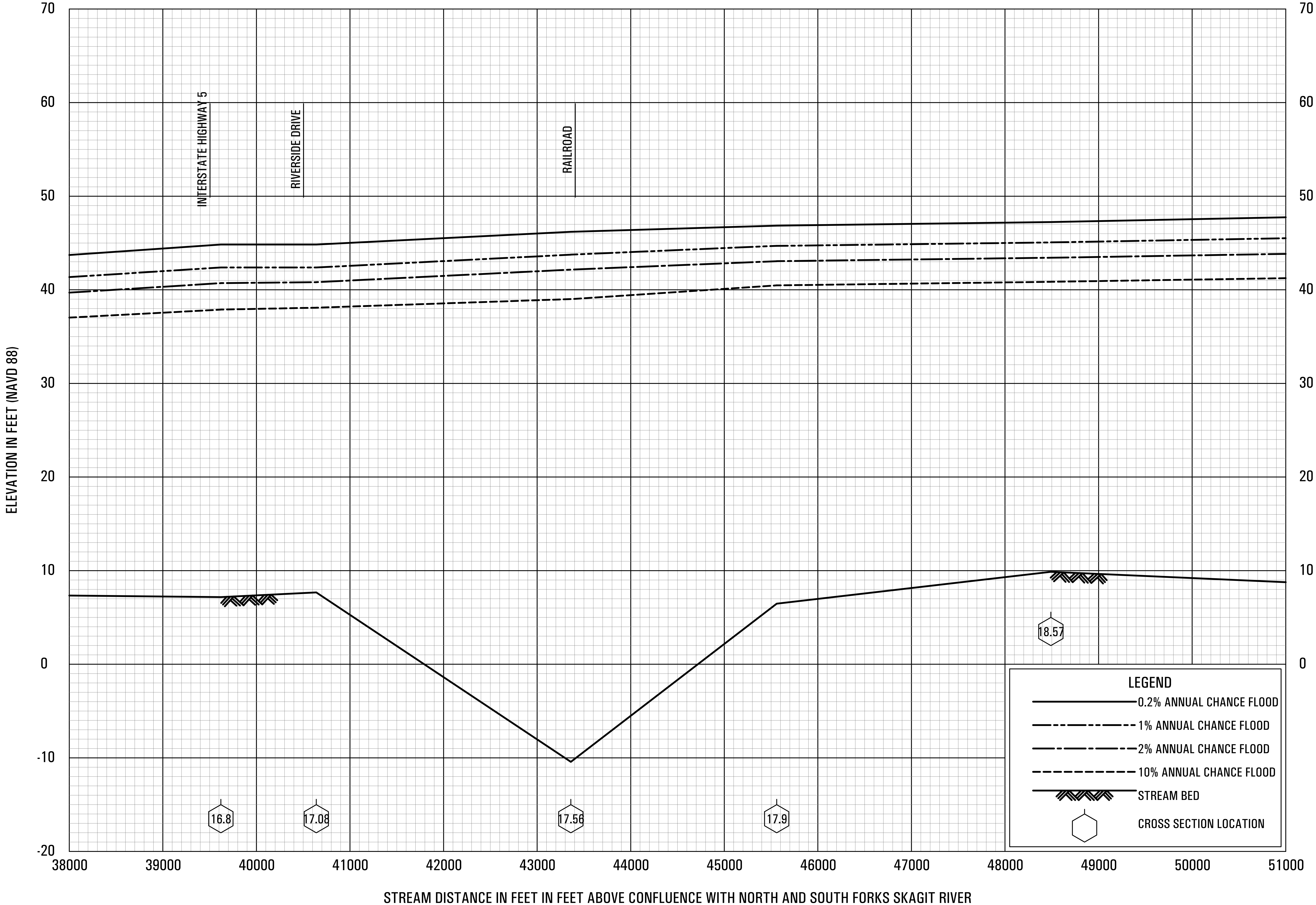
FLOOD PROFILES

SKAGIT RIVER MAIN STEM

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND UNINCORPORATED AREAS



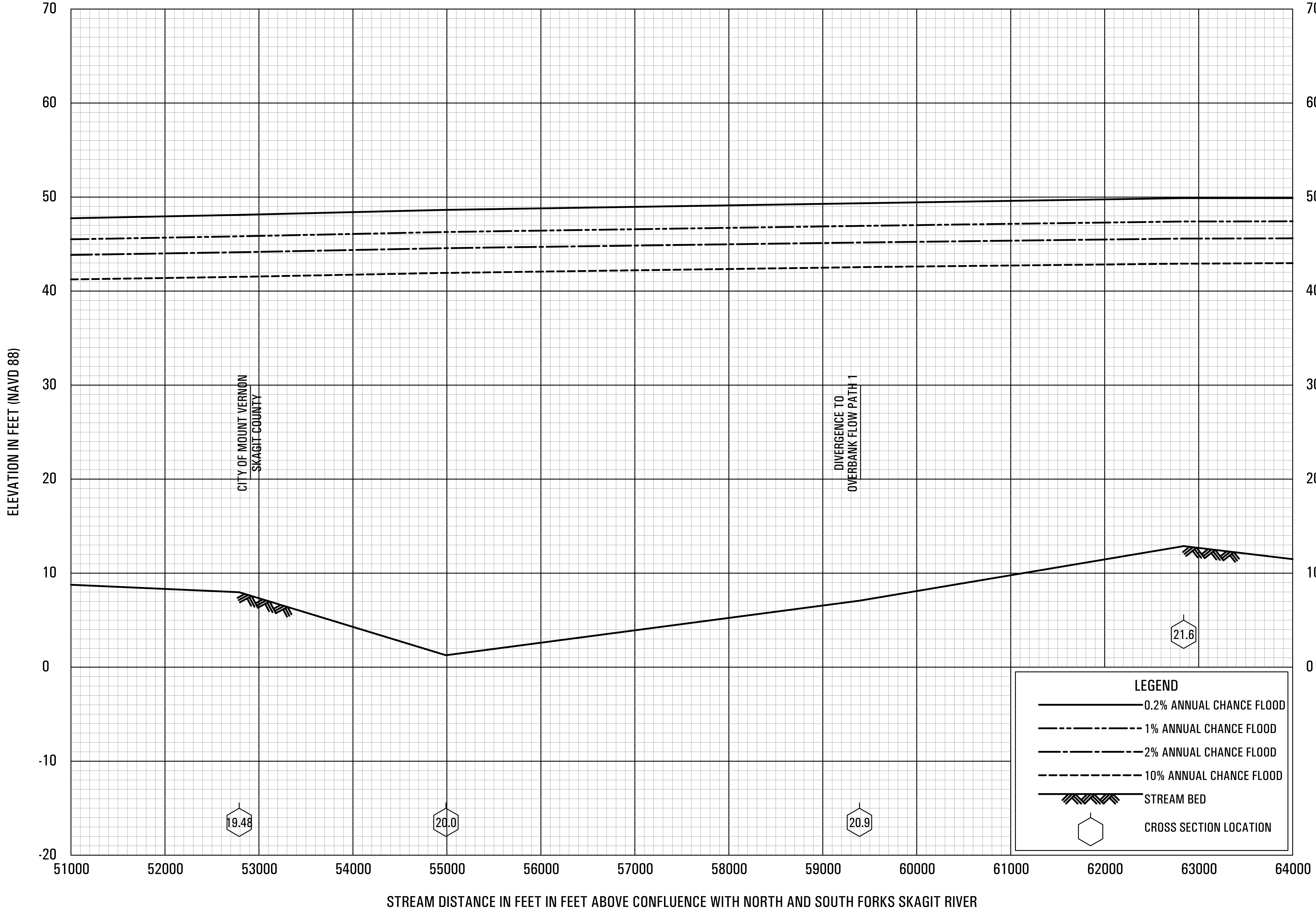
FLOOD PROFILES

SKAGIT RIVER MAIN STEM

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND UNINCORPORATED AREAS



FLOOD PROFILES

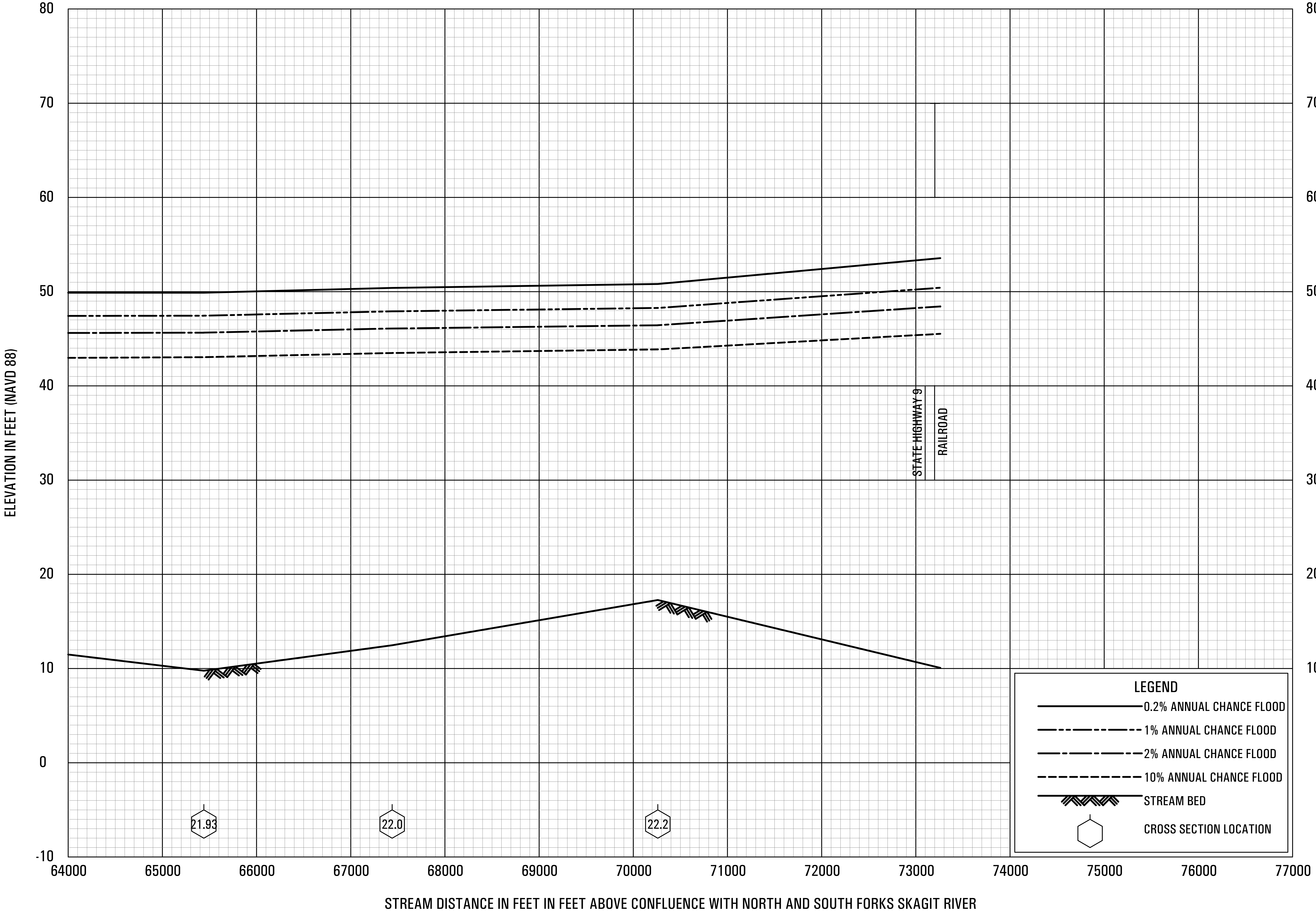
SKAGIT RIVER MAIN STEM

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND UNINCORPORATED AREAS

38P



FLOOD PROFILES

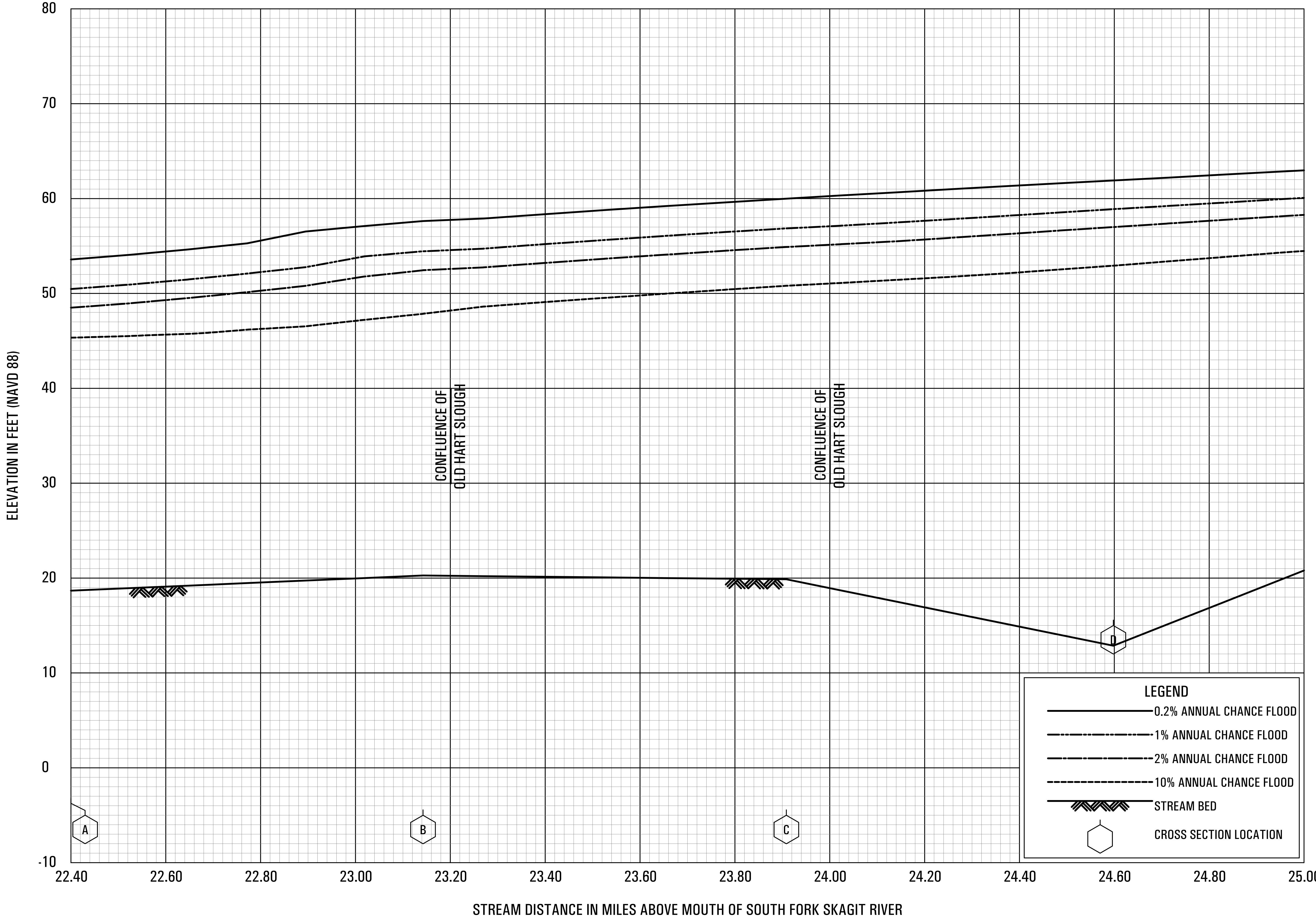
SKAGIT RIVER MAIN STEM

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND UNINCORPORATED AREAS

39P



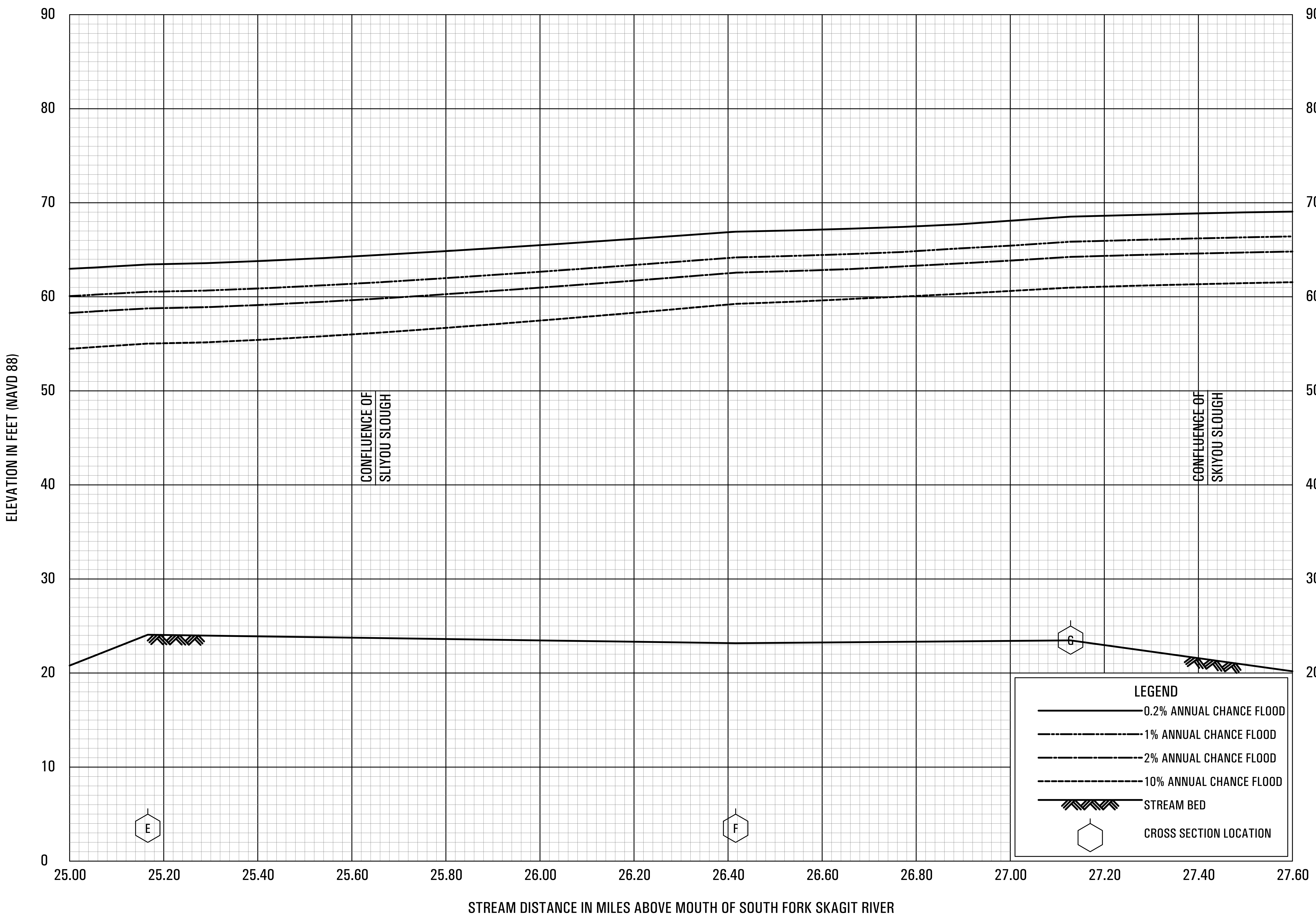
FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



FLOOD PROFILES

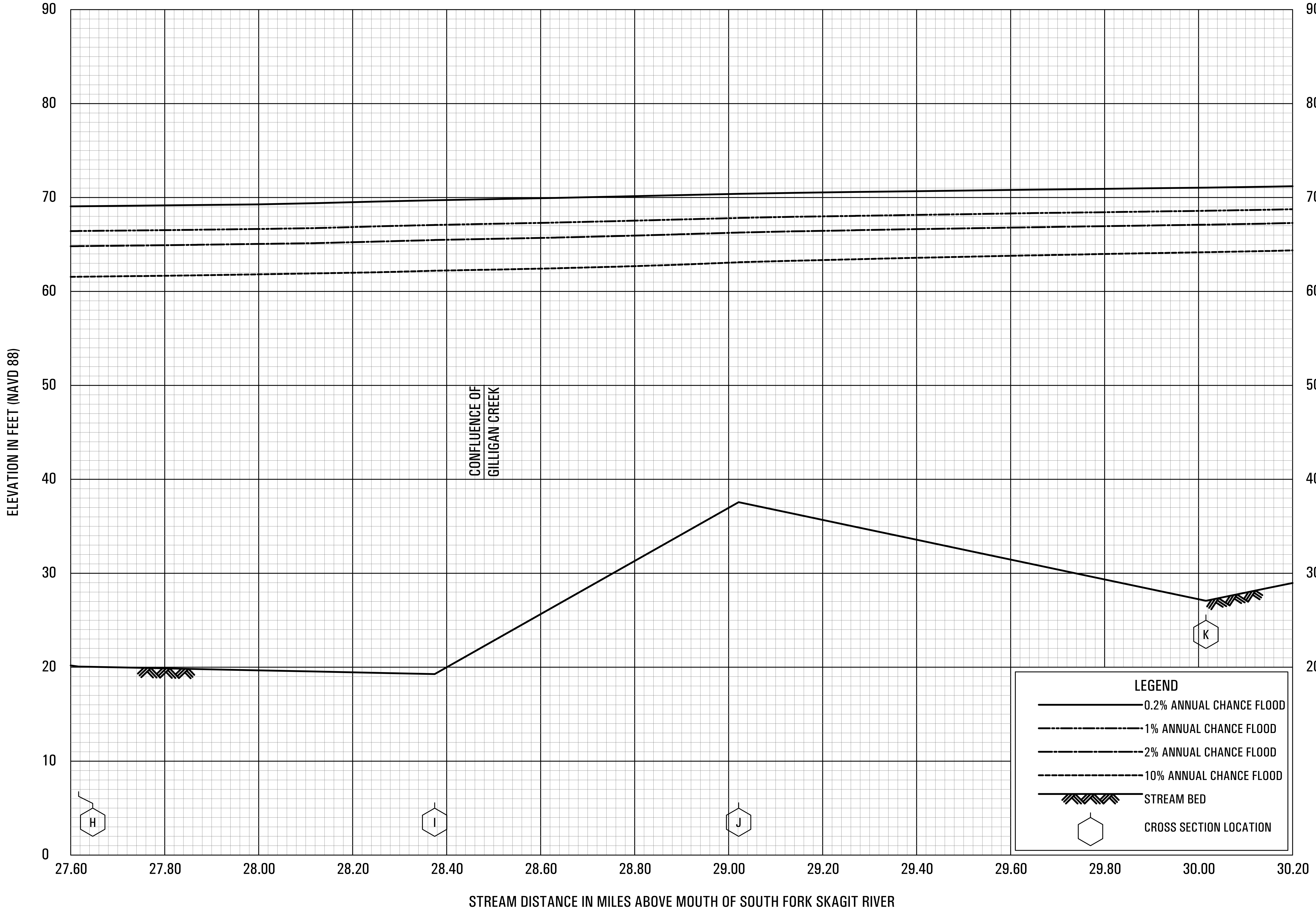
SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

41P



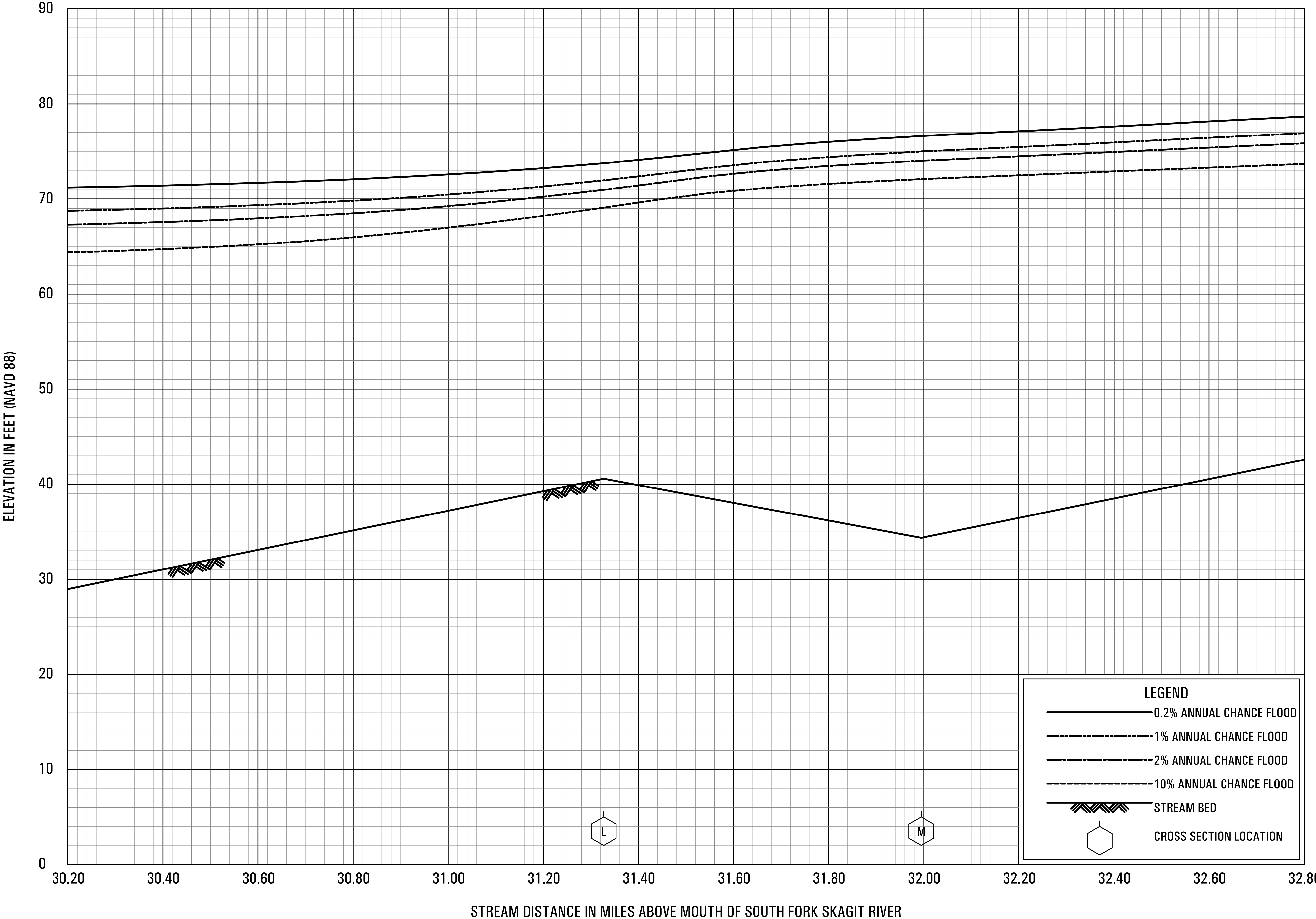
FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



FLOOD PROFILES

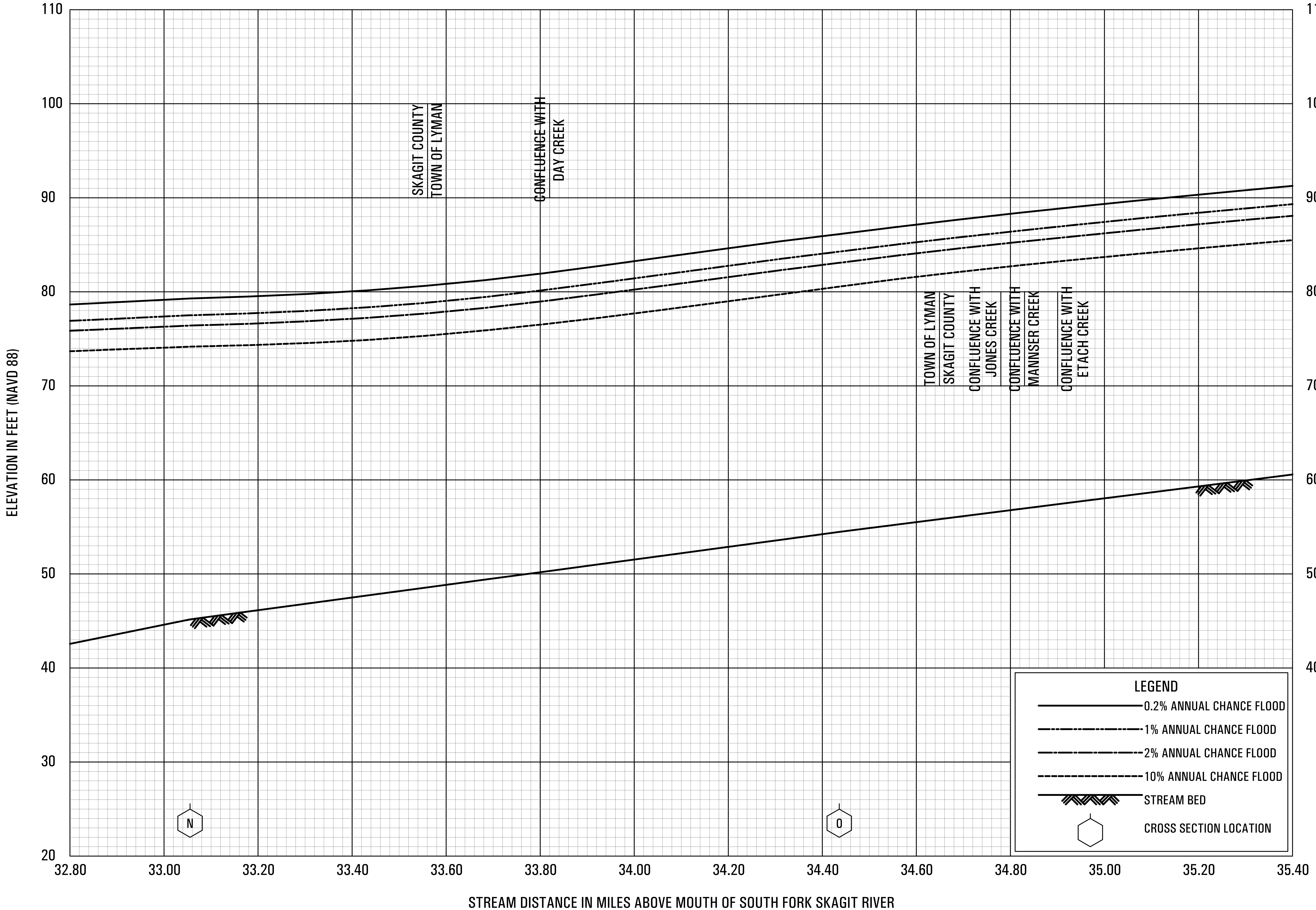
SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

43P



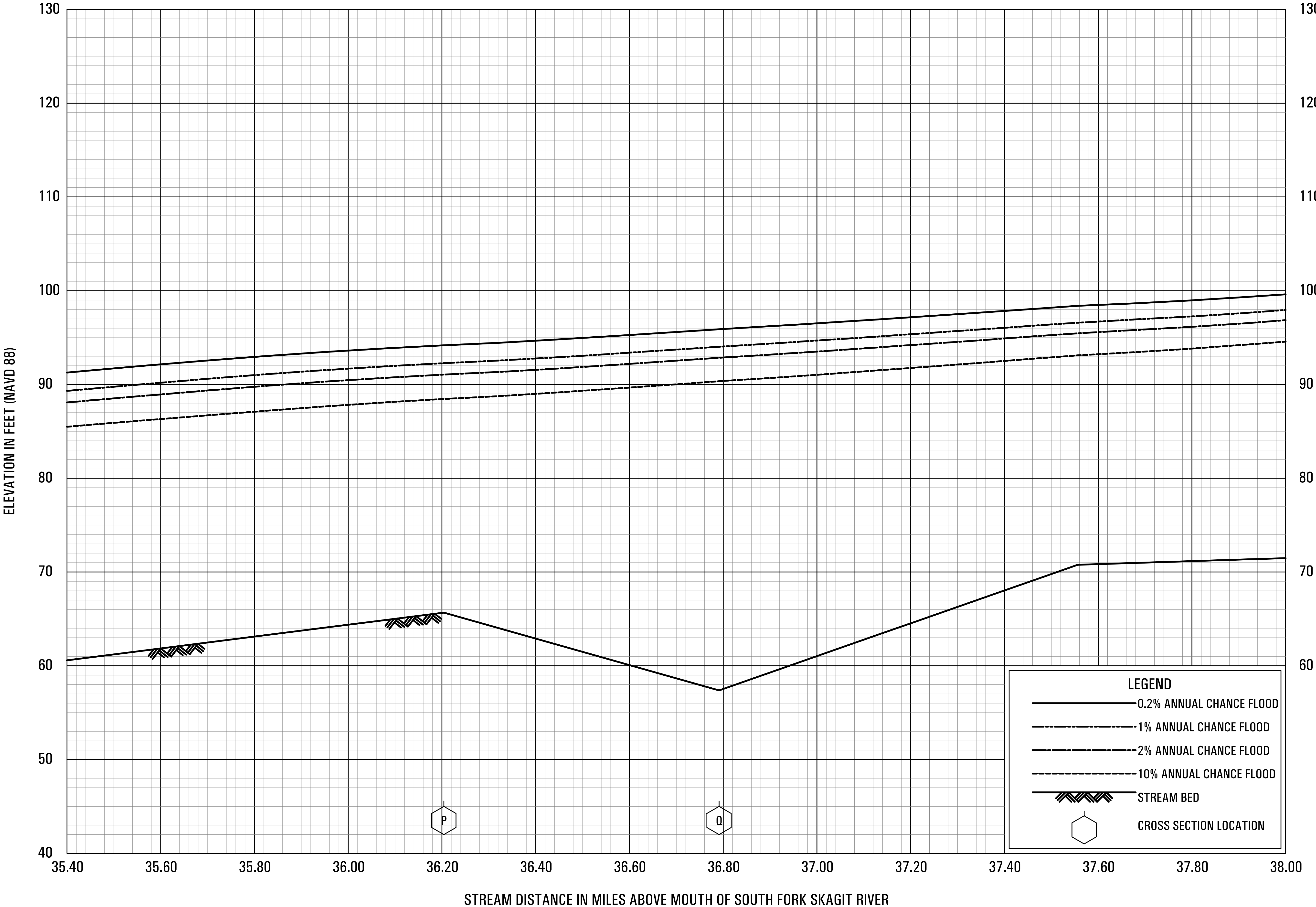
FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

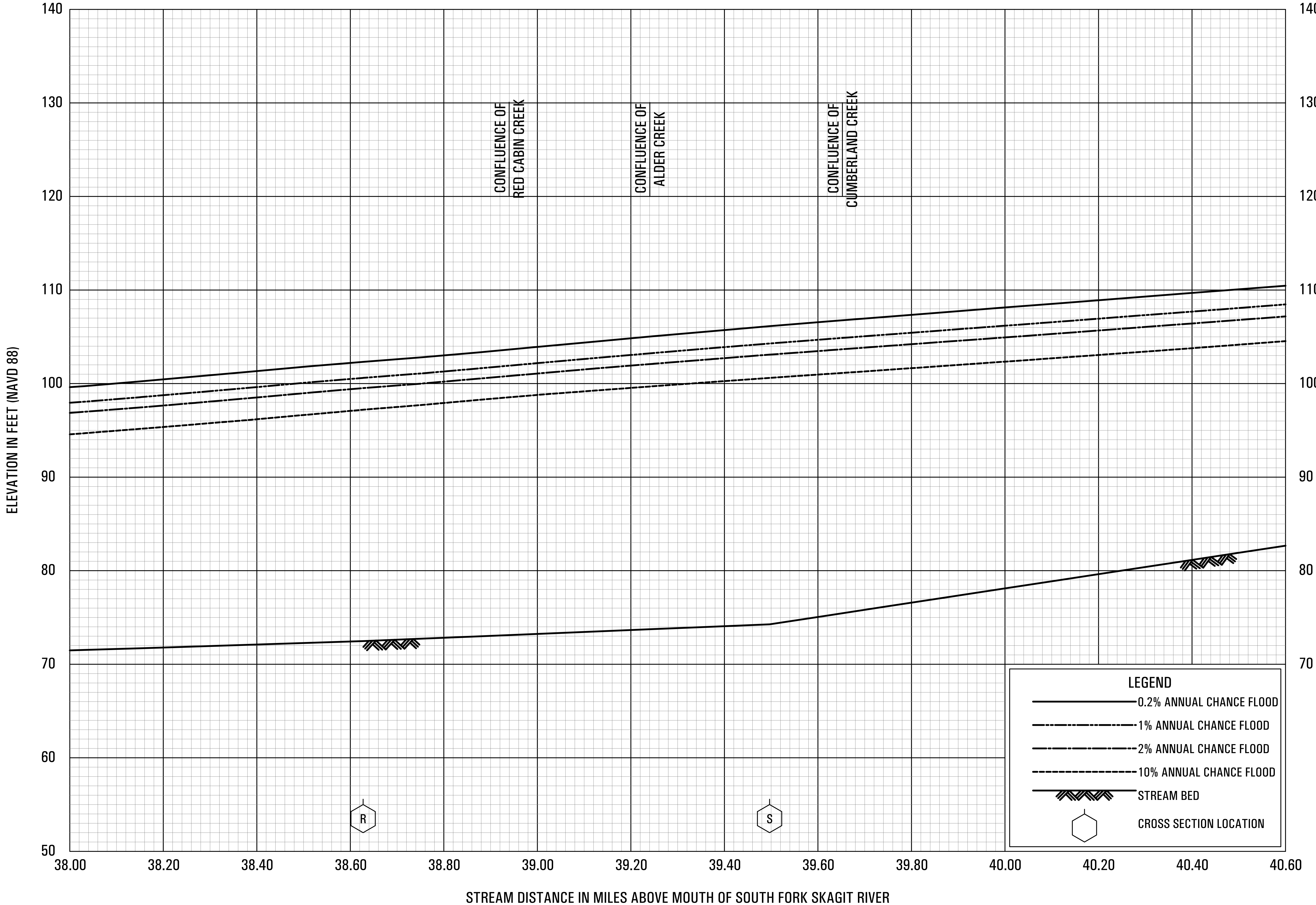


FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA
AND INCORPORATED AREAS



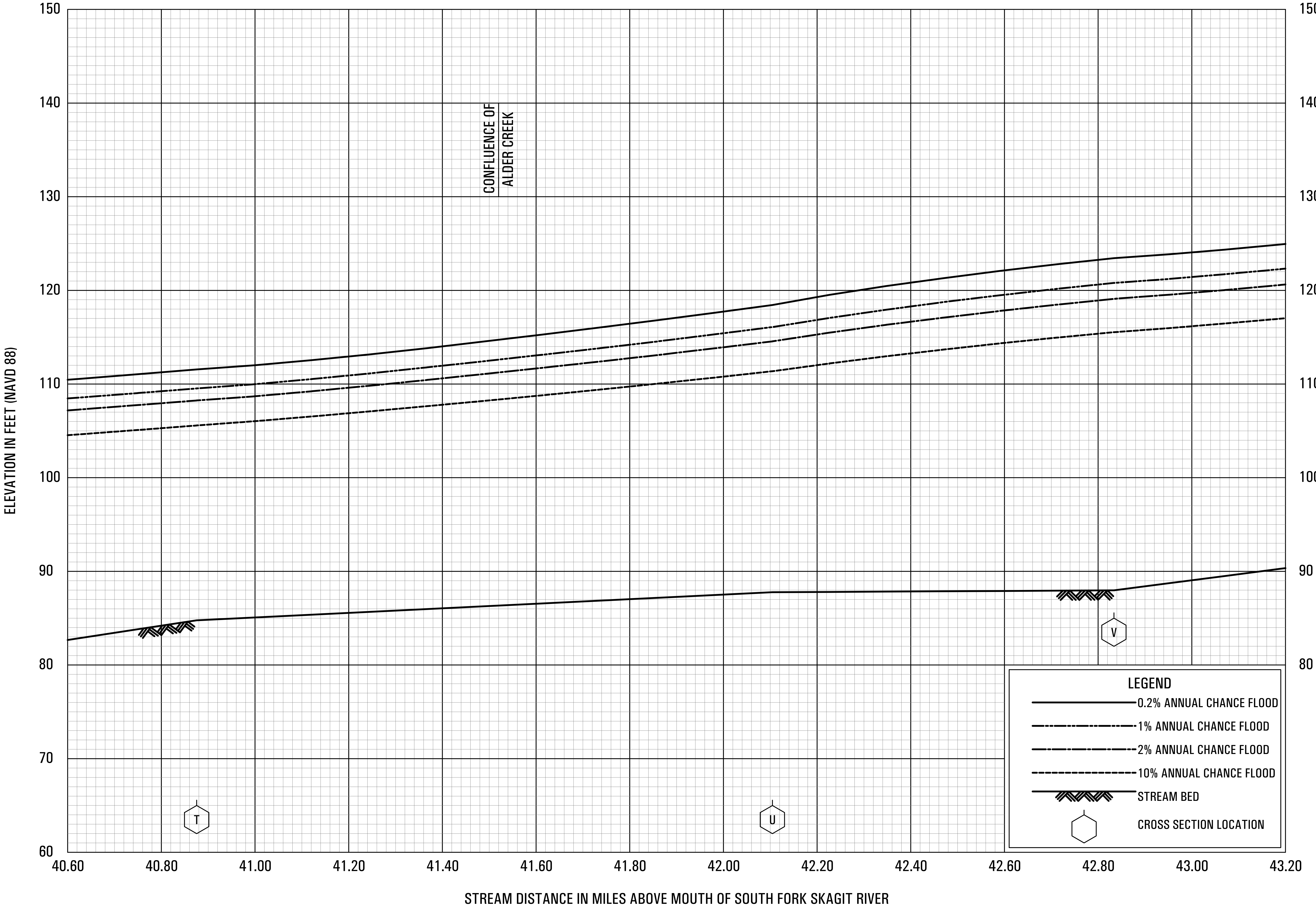
FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



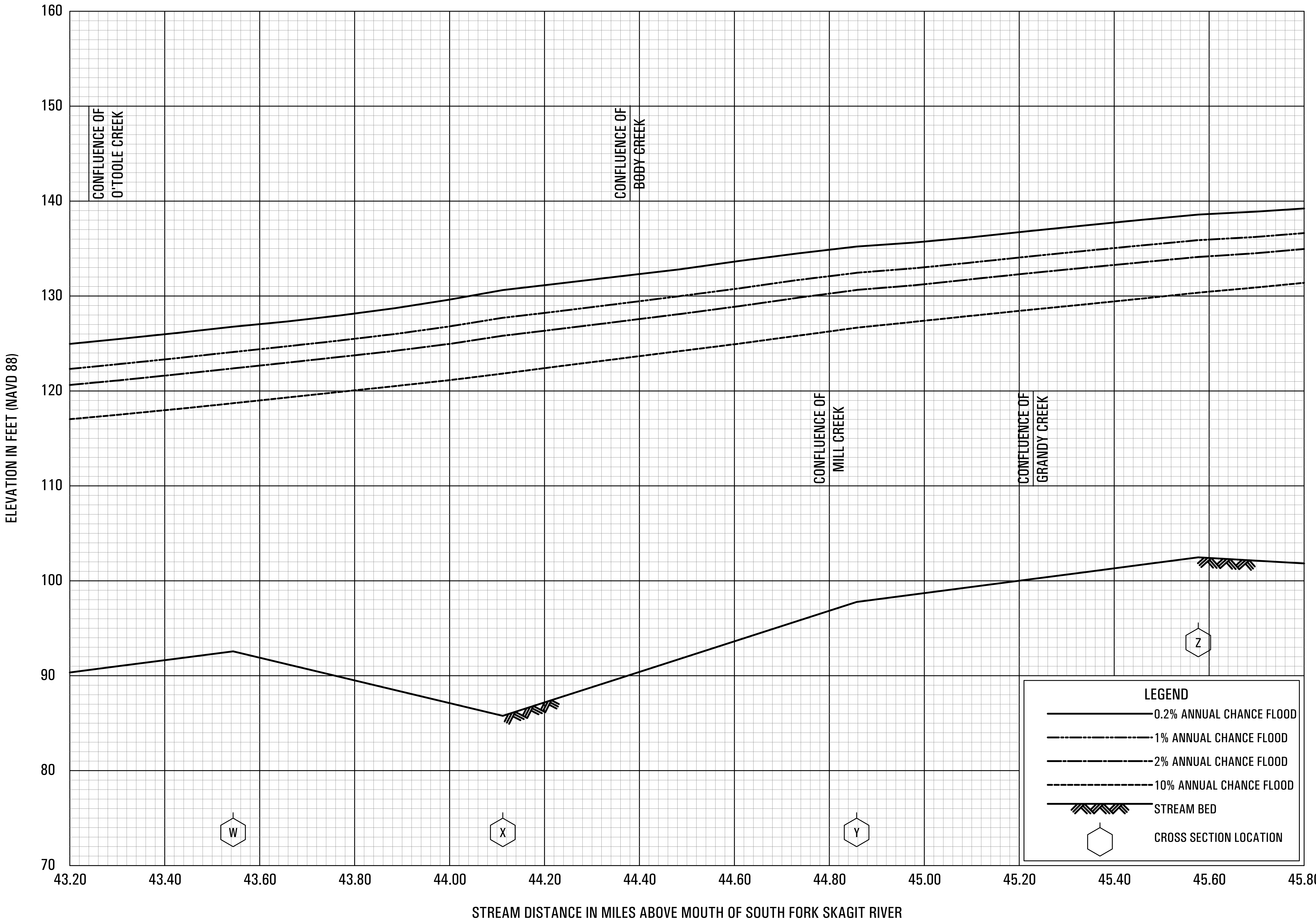
FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



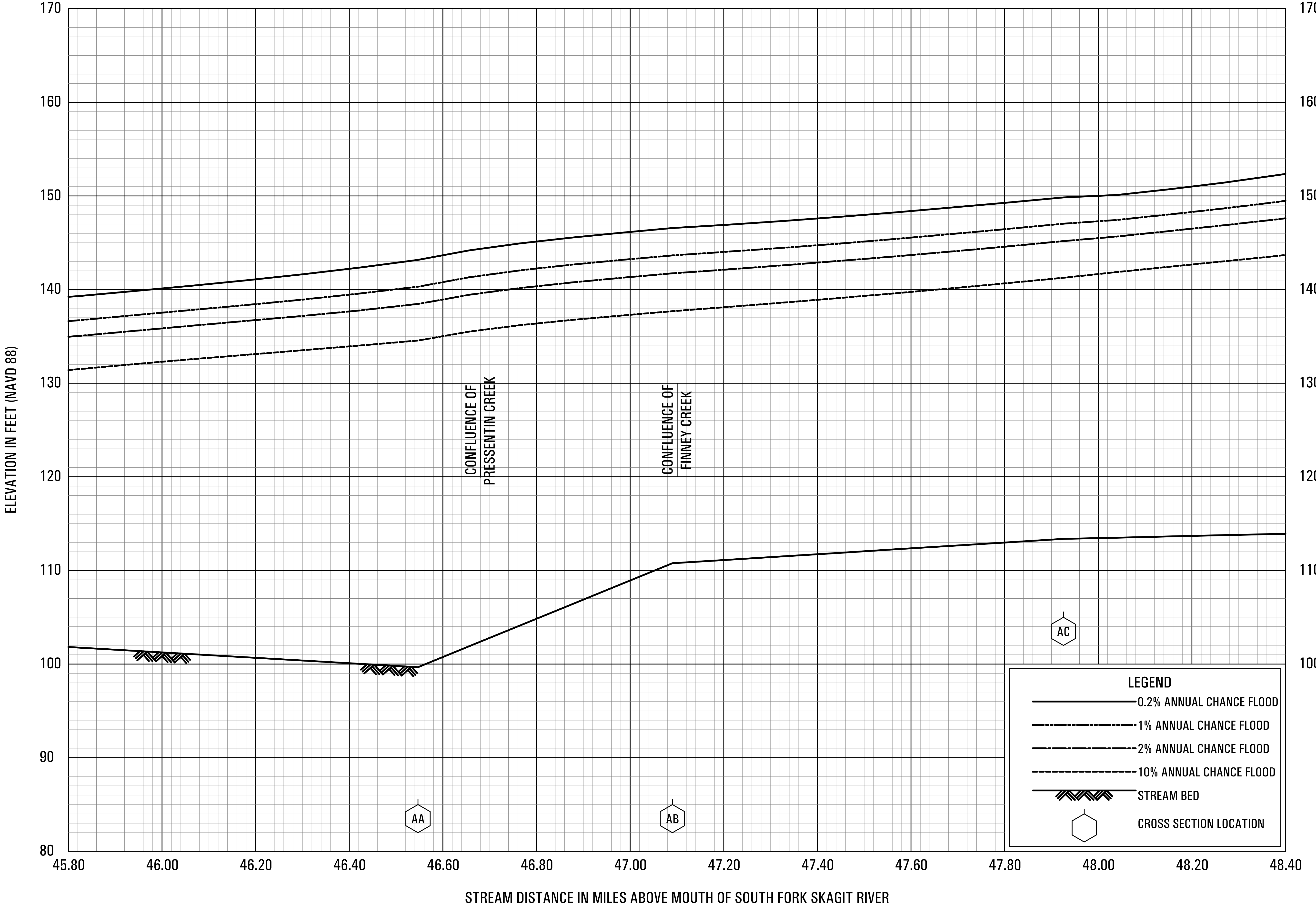
FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



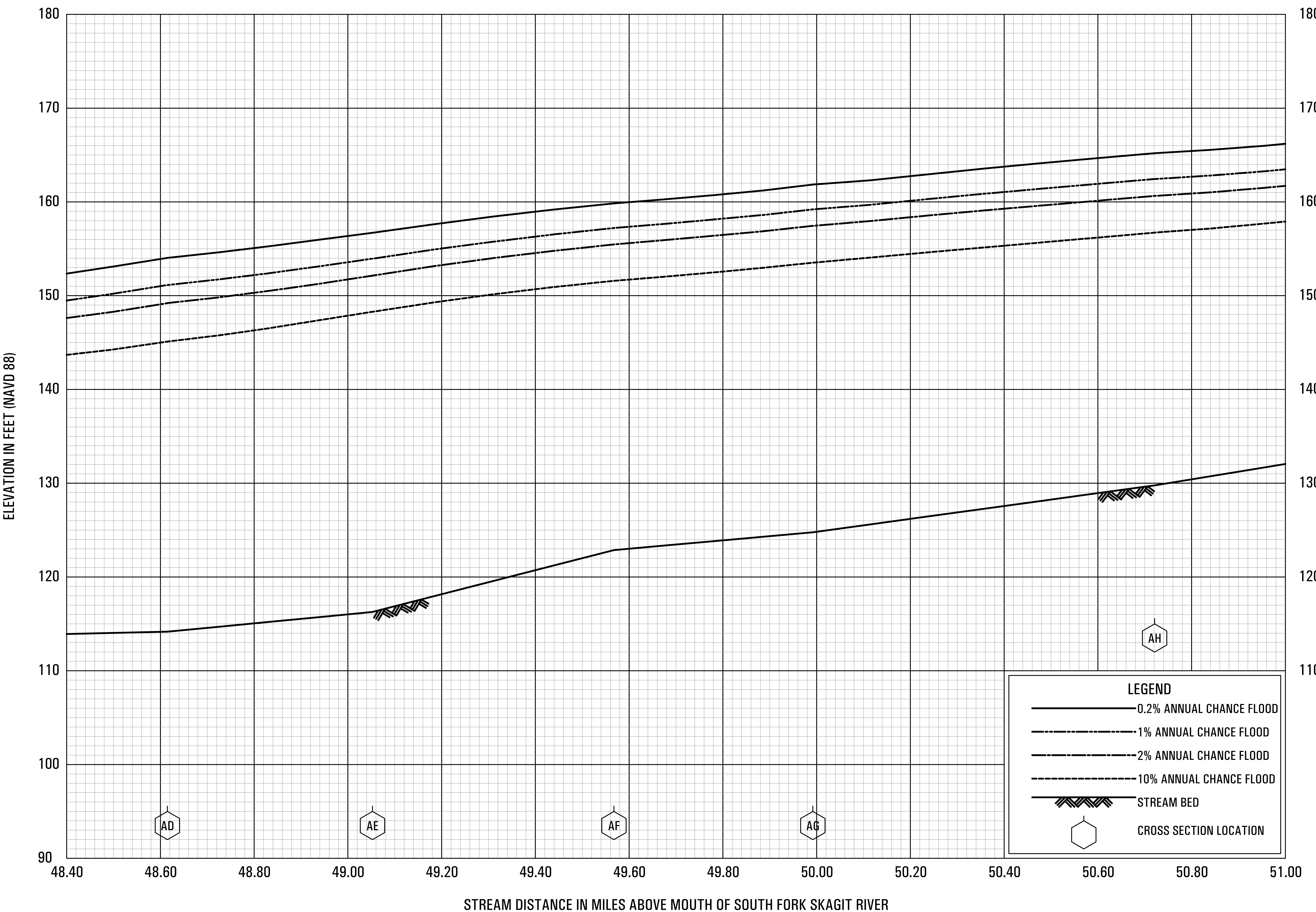
FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



FLOOD PROFILES

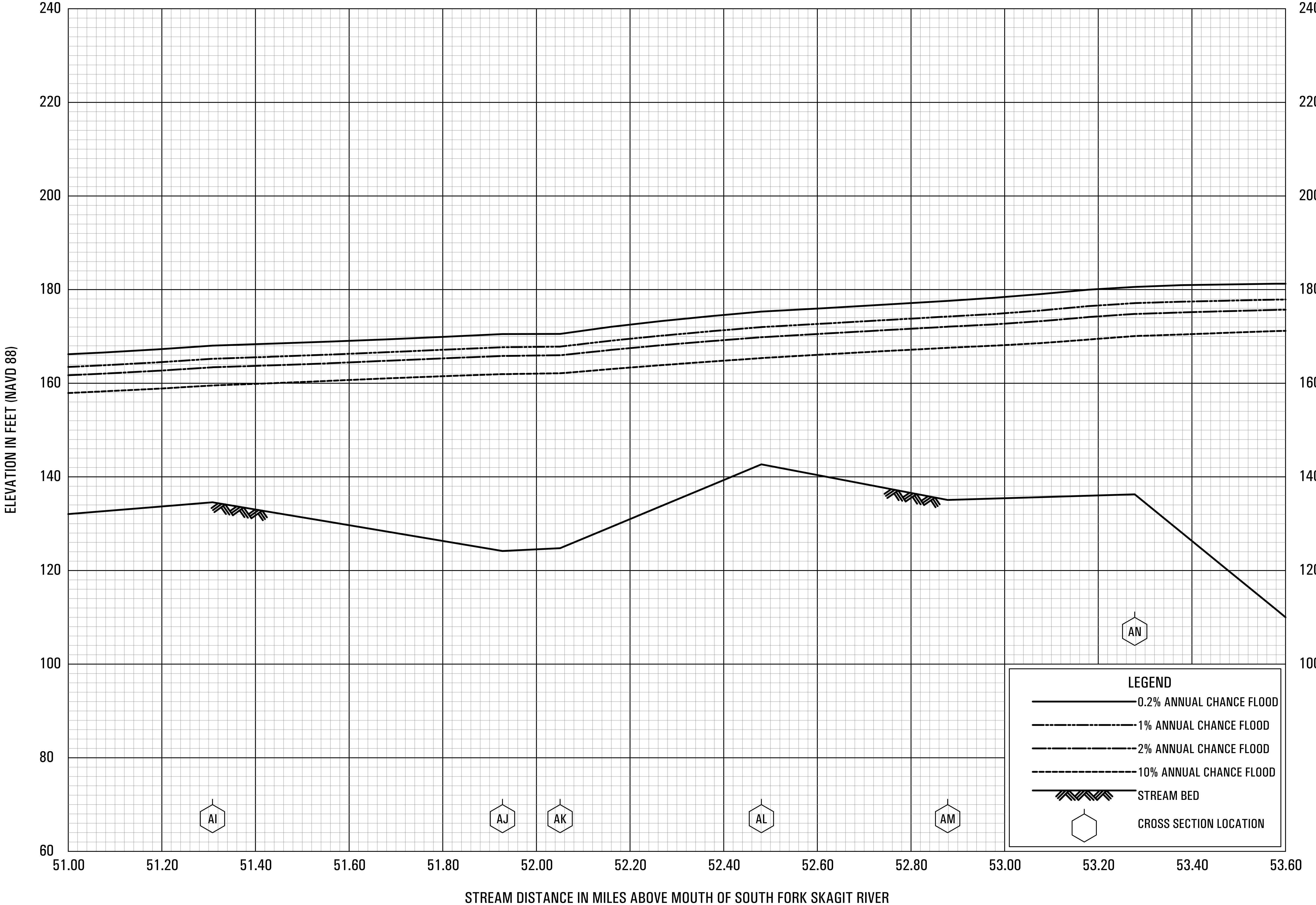
SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

50P



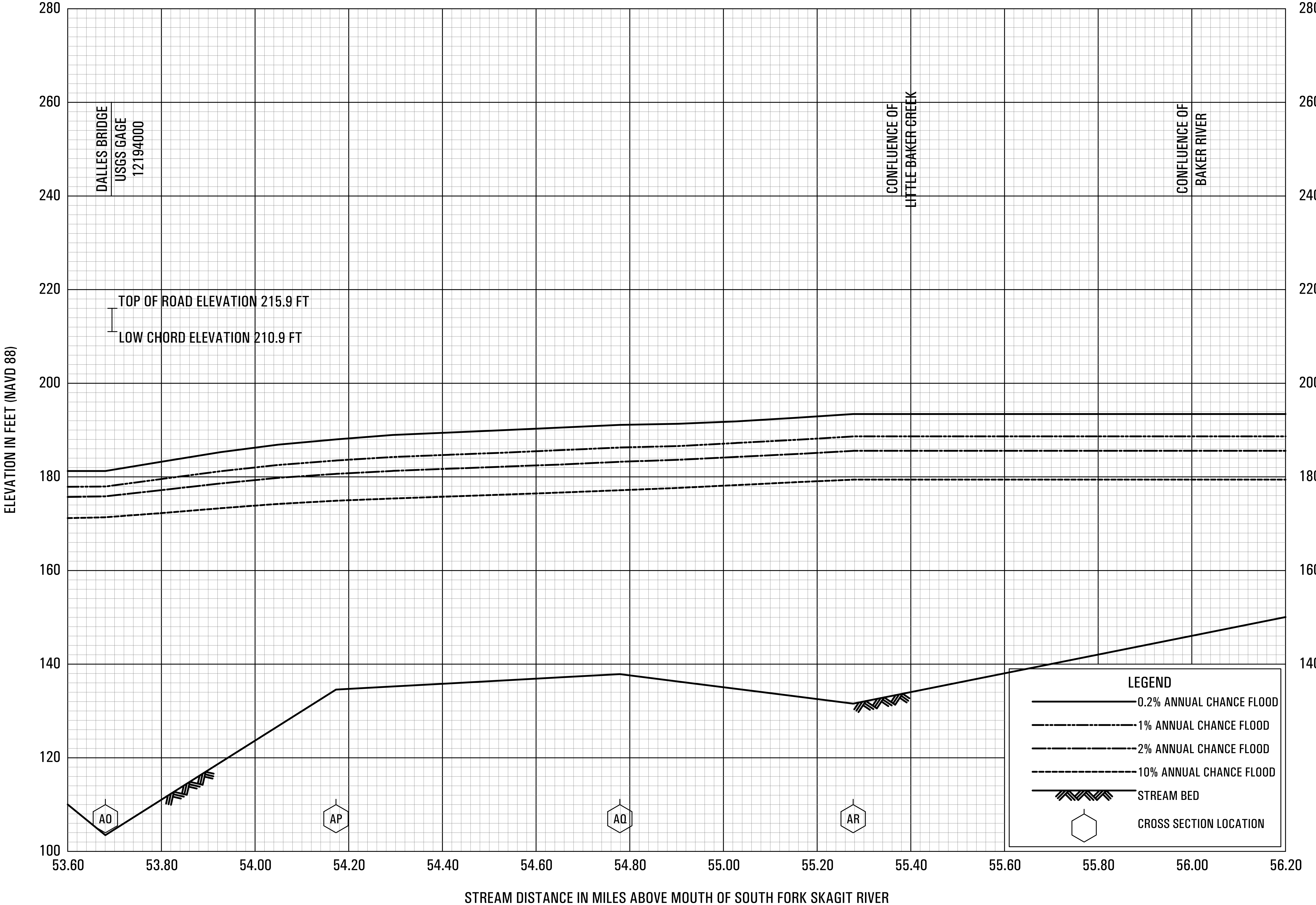
FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



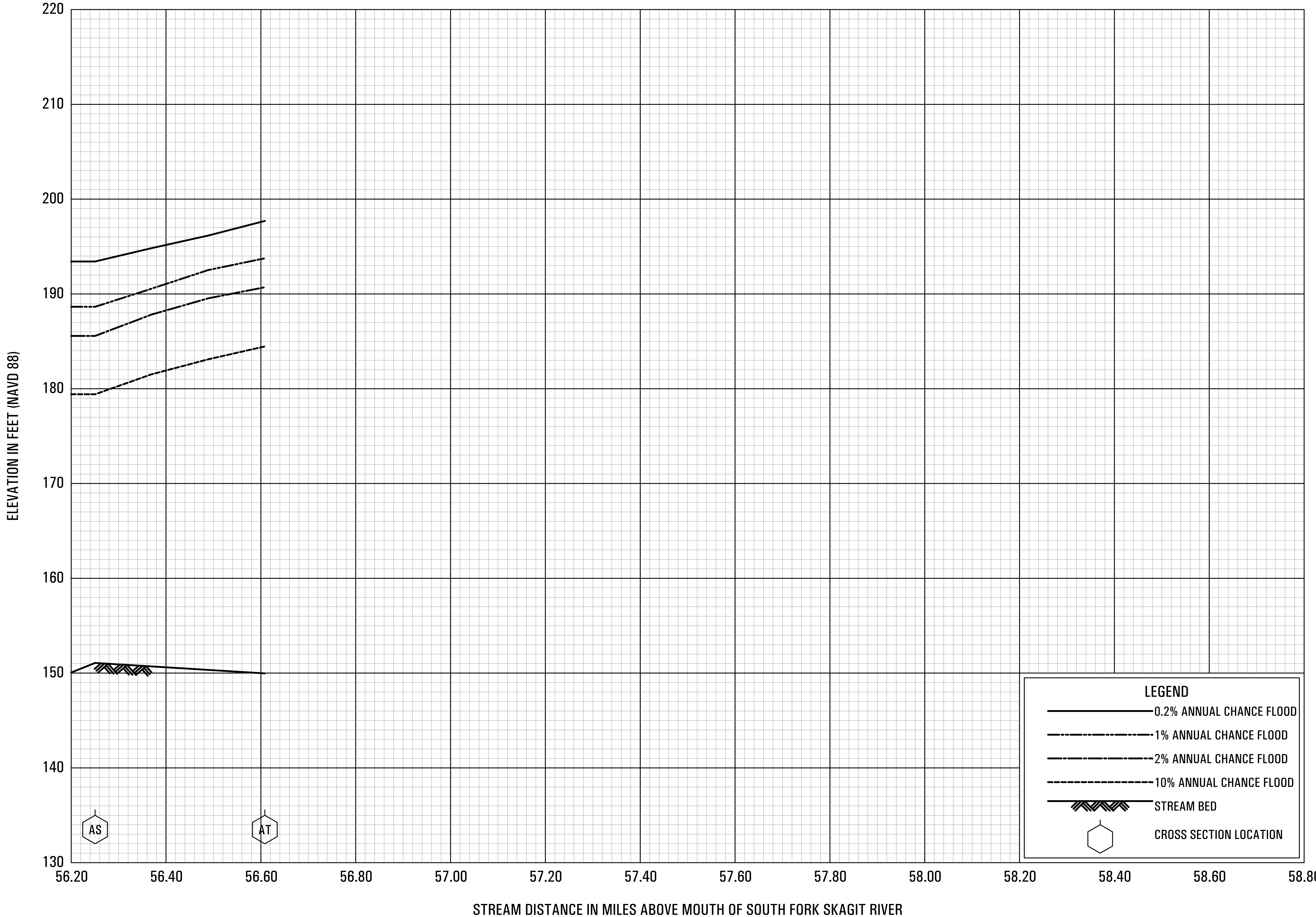
FLOOD PROFILES

SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS



FLOOD PROFILES

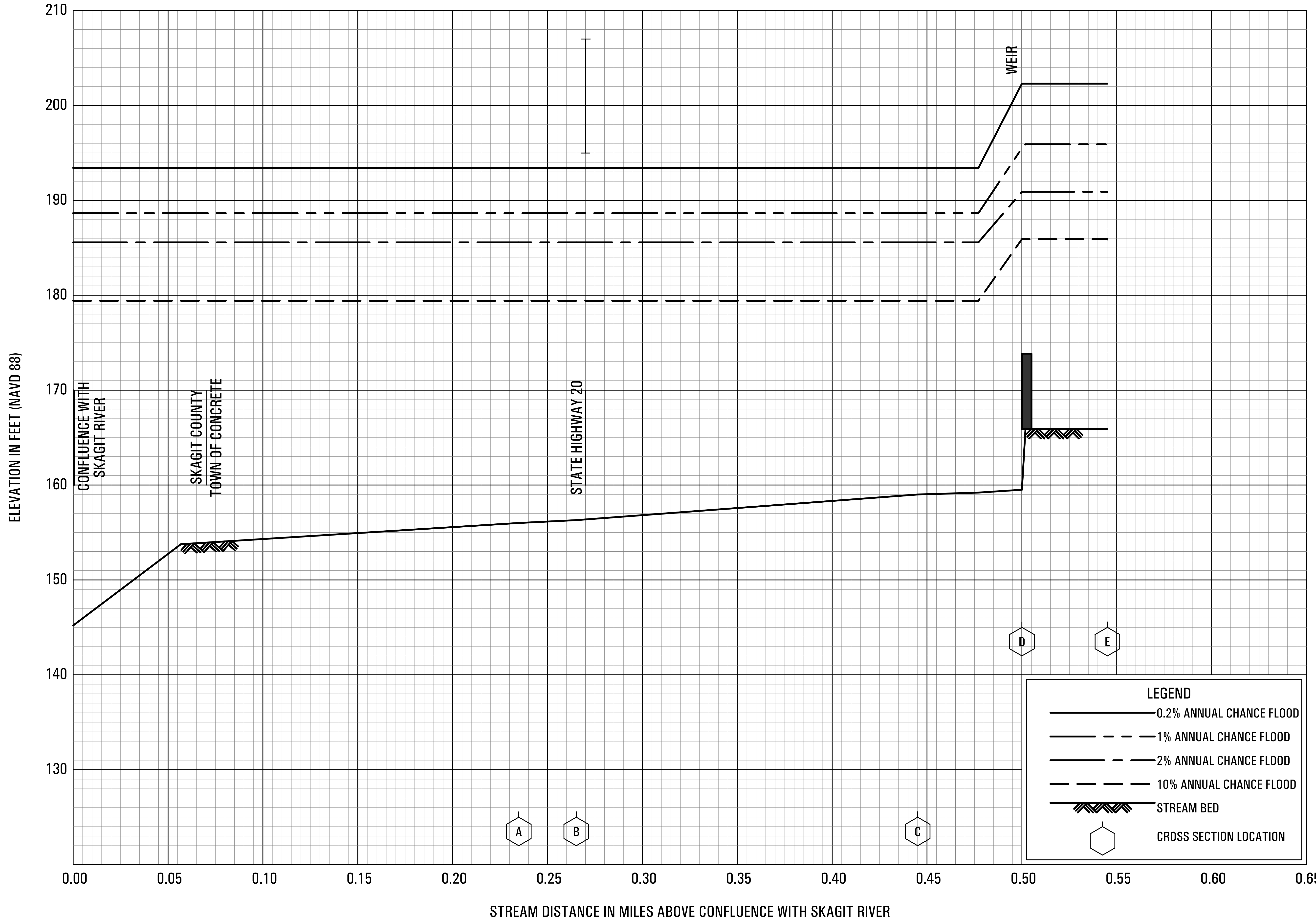
SKAGIT RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

53P



FLOOD PROFILES

BAKER RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA

AND INCORPORATED AREAS

01P

APPENDIX C

Revised Floodway Data (Table 9)

Skagit River Cross Sections A through AT

FLOODING SOURCE			FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	Model Station Number	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY (FEET NAVD)	WITH FLOODWAY (FEET NAVD)	INCREASE
Skagit River									
A	22.40	22.40	973	20,086	8.8	50.5	50.5	50.8	0.3
B	23.20	23.14	2,744	38,641	4.8	54.5	54.5	54.7	0.2
C	24.10	23.91	1,658	30,950	6.0	56.9	56.9	57.2	0.3
D	24.70	24.60	4,647	61,105	3.1	58.9	58.9	59.6	0.7
E	25.20	25.17	7,174	121,785	1.5	60.5	60.5	61.1	0.6
F	26.46	26.42	4,676	62,273	3.0	64.2	64.2	64.6	0.4
G	27.04	27.13	4,858	82,594	2.3	65.9	65.9	66.2	0.3
H	27.58	27.62	6,922	99,548	1.9	66.4	66.4	66.9	0.4
I	28.20	28.37	7,124	100,603	1.9	67.1	67.1	67.5	0.5
J	29.10	29.02	8,948	103,171	1.8	67.8	67.8	68.3	0.5
K	29.80	30.02	10,449	117,899	1.6	68.6	68.6	69.0	0.5
L	31.45	31.33	9,405	65,722	2.9	72.0	72.0	72.5	0.5
M	32.10	31.99	6,399	75,454	2.5	75.0	75.0	75.6	0.5
N	33.30	33.06	4,743	62,695	3.0	77.5	77.5	78.1	0.5
O	34.80	34.44	4,801	39,813	4.8	84.3	84.3	85.2	0.9
P	36.70	36.20	7,065	80,597	2.4	92.3	92.3	92.9	0.6
Q	37.34	36.79	7,670	71,238	2.7	94.0	94.0	94.7	0.6
R	39.00	38.63	4,984	51,508	3.7	100.6	100.6	100.9	0.3
S	39.80	39.50	3,394	42,528	4.5	104.3	104.3	104.7	0.4
T	41.10	40.88	2,572	31,649	6.0	109.5	109.5	110.5	1.0
U	42.50	42.10	1,611	23,029	8.2	116.1	116.1	116.6	0.5
V	43.15	42.83	2,016	24,468	7.7	120.8	120.8	121.2	0.4
W	43.90	43.54	837	19,123	9.9	124.1	124.1	124.8	0.7
X	44.50	44.11	2,592	26,148	7.2	127.7	127.7	128.1	0.4
Y	45.20	44.86	3,744	44,176	4.3	132.5	132.5	133.2	0.7
Z	45.90	45.58	1,917	28,442	6.6	135.9	135.9	136.4	0.5

[1] Miles above mouth of South Fork Skagit River

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

SKAGIT COUNTY, WA AND INCORPORATED AREAS)

FLOODWAY DATA

SKAGIT RIVER

FLOODING SOURCE			FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	Model Station Number	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY (FEET NAVD)	WITH FLOODWAY (FEET NAVD)	INCREASE
Skagit River									
AA	46.85	46.55	1,100	18,681	10.0	140.3	140.3	140.7	0.4
AB	47.55	47.09	1,953	26,875	7.0	143.7	143.7	144.2	0.5
AC	48.45	47.93	1,402	21,426	8.7	147.0	147.0	147.2	0.2
AD	49.20	48.61	1,652	23,045	8.1	151.1	151.1	151.3	0.2
AE	49.60	49.05	1,579	21,677	8.6	154.0	154.0	154.7	0.7
AF	50.05	49.57	1,788	32,850	5.7	157.2	157.2	158.2	0.9
AG	50.45	49.99	1,777	30,864	6.0	159.2	159.2	160.1	0.9
AH	51.10	50.72	2,594	47,518	3.9	162.4	162.4	163.1	0.7
AI	51.80	51.31	2,215	38,753	4.8	165.2	165.2	165.7	0.5
AJ	52.40	51.93	838	20,495	9.0	167.7	167.7	168.1	0.4
AK	52.55	52.05	698	16,820	11.0	167.8	167.8	168.2	0.3
AL	52.95	52.48	659	17,859	10.4	172.0	172.0	172.3	0.3
AM	53.30	52.88	657	17,260	10.7	174.3	174.3	174.5	0.2
AN	53.65	53.28	811	22,411	8.2	177.1	177.1	177.3	0.2
AO	54.10	53.68	270	11,737	15.7	178.0	178.0	178.1	0.2
AP	54.65	54.17	630	15,503	11.9	183.5	183.5	184.5	1.0
AQ	55.35	54.78	1,557	28,797	6.4	186.3	186.3	187.1	0.8
AR	55.75	55.28	1,386	35,482	5.2	188.7	188.7	189.5	0.9
AS	56.70	56.25	644	15,868	10.9	188.7	188.7	189.5	0.9
AT	57.10	56.61	563	17,074	10.1	193.8	193.8	194.3	0.6
AU	*	57.38	1,286	33,154	5.2	197.4	197.4	197.9	0.5
AV	*	57.74	2,057	42,666	4.0	199.1	199.1	199.6	0.5
AW	*	58.16	3,624	66,881	3.1	200.0	200.0	200.9	0.9
AX	*	58.51	3,746	66,543	3.1	200.4	200.4	201.3	0.9
AY	*	59.36	2,243	37,078	5.5	201.8	201.8	202.7	0.9
AZ	*	60	2,000	30,589	6.7	204.0	204.0	204.9	0.9

[1]Miles above mouth of South Fork Skagit River

*Data not available

TABLE 9	FEDERAL EMERGENCY MANAGEMENT AGENCY **SKAGIT COUNTY, WA** **AND INCORPORATED AREAS)**	**FLOODWAY DATA** **SKAGIT RIVER**