PACIFIC INTERNATIONAL ENGINEERING PLLC

Response to Review Comments Prepared by Wilbert O. Thomas on behalf of FEMA Region X for the Skagit River Flood Frequency Analyses

Introduction

In November 2005, after a series of four community workshops (including attendance by FEMA and USACE representatives) addressing the hydrology and hydraulics (H&H) of the Skagit River watershed, FEMA requested Skagit County to prepare and submit a report documenting the H&H work of its engineering consultant, Pacific International Engineering (PIE). The requested PIE report, together with supporting analysis and H&H models, was sent to FEMA on December 13, 2005. On February 10, 2006 FEMA Region X transmitted, by letter to the Skagit County Commissioners, a report prepared by Wilbert O. Thomas entitled "An Evaluation of Flood Frequency Analyses for the Skagit River" (Thomas Report). The Thomas Report documents review comments by FEMA's consultant, Michael Baker, Jr., Inc. on the H&H report submitted by PIE. As will be demonstrated in this response, the Thomas report was not a rigorous independent technical review of the PIE H&H analysis; nevertheless, the Thomas report has been cited by the USACE and others as an authoritative dismissal of PIE's H&H technical work, which was conducted on behalf of Skagit County from late 2003 through 2005.

Many of the comments and conclusions presented in the Thomas Report are not accurate. The purpose of this response is to address the inaccuracies in the Thomas report so that FEMA can reevaluate its interpretation and resulting conclusions about these important H&H issues. This response is organized by sequentially restating each Thomas Report comment and then providing a response (italics) to each comment. The comments follow the headings of the Thomas Report. This response has been prepared by PIE on behalf of and funded by the Skagit River Impact Partnership (SRIP).

Background

Comment 1

The Seattle District of the U.S. Army Corps of Engineers (USACE) is conducting a flood damage reduction feasibility study for the Skagit River in cooperation with Skagit County, Washington. The purpose of the study is to formulate and recommend a comprehensive flood hazard management plan for the Skagit River floodplain that will reduce flood damages in Skagit County. The results of this study will also be used to revise the Flood Insurance Study and Flood Insurance Rate Map for Skagit County.

Response 1

The cities, towns, and dike districts most impacted by Skagit River flooding (communities) believe they have not been properly consulted by FEMA in order to review and accept the USACE study methodology and results. These most impacted *communities do not agree that the USACE study should be used to revise the Flood Insurance Study (FIS) and Flood Insurance Rate Map for Skagit County (Skagit County, 2005 and 2006). The communities are also concerned that the USACE flood damage feasibility study H&H results and decision criteria are being used inappropriately in the context of the FEMA FIS, and that the decision criteria used for the USACE flood damage feasibility study seriously compromise the results of the FIS.*

The flood damage reduction feasibility study for the Skagit River was conducted by the USACE and the County within the context of the USACE General Investigation (GI) program. GI studies require a 50 percent cost share by the local sponsor, in this case, Skagit County. The USACE and Skagit County entered into an agreement whereby the County performed the H&H studies as in-kind services as its part of the 50 percent cost sharing program requirement. Early on in this work, the County requested its consultant, PIE, conduct more in-depth hydrologic analysis than had been conducted by the Corps in order to develop an accurate hydraulic model which would enable the County to predict, in real time, what the downstream flood flow would be, given on-the-ground stream gage information and weather information developed early in a flood event. On a parallel track, the Corps continued to develop its own hydrologic and hydraulic analysis pursuant to its role in the GI process but also because it was under contract with FEMA to provide the basis of FEMA's new base flood elevation maps. When it became apparent that the USACE and County hydraulic models differed, the County requested USACE's Hydrologic Engineering Center, Davis, California, (HEC) to perform an internal technical review of the USACE and PIE work. The HEC review work was never completed.

The difference between the decision criteria associated with the level of detail of technical work that the USACE applied on a GI study and FEMA's decision criteria are also different. A high level of accuracy requires more rigorous development and analysis of the data and therefore, is more expensive. The USACE criteria in a GI study is that a material change in the results must justify the cost of performing additional work. The USACE decision criteria to do additional work (or even review the work submitted by a GI study sponsor) is determined by whether or not the potential change would adjust the economic damages caused by the 100-year flood by more than 10 percent. In a GI study, flood flow values resulting from the USACE H&H studies are used to quantify the expected economic loss that would result from flood damage under "without project" conditions. This figure is then compared to the cost of implementing flood damage reduction measures and the cost of the expected flood damage after project measures are constructed. This economic analysis establishes the cost to benefit ratio for the justification for the USACE to recommend a project for construction using federal funds. The USACE criteria for the amount of detail in the analysis is therefore determined by how much the additional detail will affect the estimate of economic loss. In general, if the economic loss will not be changed by more than 10 percent, the USACE will not perform additional investigations. In the case of the Skagit River, the USACE analysis predicts that most of the economic loss occurs around the 25 to 50-year flood events, and little additional economic loss is added above the 50-year event. In the context of the USACE GI study, more detailed analysis for the Skagit that addresses flood flows beyond the 50-year event is hard to justify because it would take a huge change in the 100-year *peak flood flow to add another 10 percent in damages (everything of economic value would be destroyed when the levees were overtopped before the 100-year peak flow was reached).*

FEMA establishes a base flood elevation (BFE) and requires property to be protected at the 100-year flood (base flood) flow elevation in order to be outside the floodplain delineated under the FIS program. Obviously, being in or out of the 100-year floodplain has a significant impact on the economic viability of a community. It can mean the difference between being required to be insured or not, or being able to develop a property or not. These economic values which are critical to a community, would not necessarily trigger a USACE decision to do a more detailed (and expensive) analysis. In the case of the Skagit FIS, the USACE has used the GI criteria to determine that additional analysis of PIE H&H studies is not warranted and that the USACE results are good enough. As an example, the USACE has chosen not to perform work to evaluate 22 years of recorded flows by the USGS even though PIE demonstrated that use of this data has the potential to reduce the 100-year flood flow peak by 2,000 to 12,000 cfs, or one to six percent. As an aside, the USGS uses a criterion of 20 percent accuracy to determine whether or not to reevaluate the Stewart estimated unrecorded peak flood flows. FEMA's use of the USACE GI study, (which does not include decision criteria that will result in a focus to accurately evaluate the H&H associated with the 100-year event) as the vehicle for performing a FEMA flood insurance study (the whole point of which is to accurately determine the 100-year base flood elevation) is only appropriate if the USACE decision criteria relative to the level of detail of the technical analysis are changed to be compatible with the FIS.

Therefore, in the particular case of the Skagit, where nearly all of the flood damage costs are associated with 50-year or smaller flood events according to the Corps' hydraulic model, using a USACE GI investigation as a vehicle to perform a FIS study is not a correct application of the GI process due to the reasons outlined above (e.g., most damage occurs at the 50-year level or less). While we disagree that the Skagit basin USACE GI H&H study is adequate, even for GI purposes, the USACE H&H analysis is clearly inadequate for the purpose of the FEMA FIS and is being inappropriately thrust upon the impacted communities as the basis for a flood insurance study.

Comment 2

The flood discharges estimated by PIE are different than those developed by USACE and this review was undertaken to determine which results are most reasonable.

Response 2

The PIE report submitted to FEMA on December 13, 2005 did not present a comparative analysis with the USACE discharge estimates. PIE review comments on the USACE analysis are comprehensively documented in many other formats including memoranda (PIE 2004c and 2004d). This documentation is extensive and it is clear that Mr. Thomas only reviewed parts of it. It is PIE's position that the USACE has made fundamental errors in generating the hydrologic data set on which all the H&H is derived. Since the USACE analysis is based on a flawed data set, the USACE H&H analysis will result in incorrect conclusions in the FIS. The purpose of PIE's submittal was to document the

PIE H&H analysis for FEMA comments. It was expected that FEMA would review the analysis and identify any deficiencies and make a request for explanation or additional discussion of the issues that FEMA questioned. But there was no interaction. The objective of this process was to reach agreement with FEMA on the PIE H&H analysis and then have FEMA use the PIE H&H in the FIS.

Peak Discharges for Four Historical Floods

Comment 3

Table 1. Summary of four historic peak discharges, in cubic feet per second (cfs), for the Skagit River near Concrete, Washington.

Date of flood	USGS published peaks (1961)	USGS (1950)	USGS (1951-52)	PIE HEC-RAS (2005)
November 1897	275,000	230,000	265,000	238,000
November 1909	260,000	220,000	240,000	217,000
December 1917	220,000	190,000	205,000	184,000
December 1921	240,000	210,000	225,000	202,000

The variability of estimates in Table 1 indicate there is uncertainty associated with the determination of peak discharges for these historic floods as reflected by the location and quality of the high water marks, cross-sectional data, and Manning's n values. However, all subsequent estimates are generally within 20 percent of the USGS published values and within the uncertainty of peak discharges determined by indirect methods (slope areas and contracted-opening measurements).

Response 3

Stating that "all subsequent estimates are generally within 20 percent of the USGS published values and within the uncertainty of peak discharges determined by indirect methods (slope areas and contracted-opening measurements)", the reviewer assumed that Stewart's high water marks (HWMs) were accurately converted from a hotel on the bank of the Baker River to the Dalles reach for the 1897, 1909, and 1917 floods. "All subsequent USGS estimates" referred to by the reviewer in this comment are also based on this assumption.

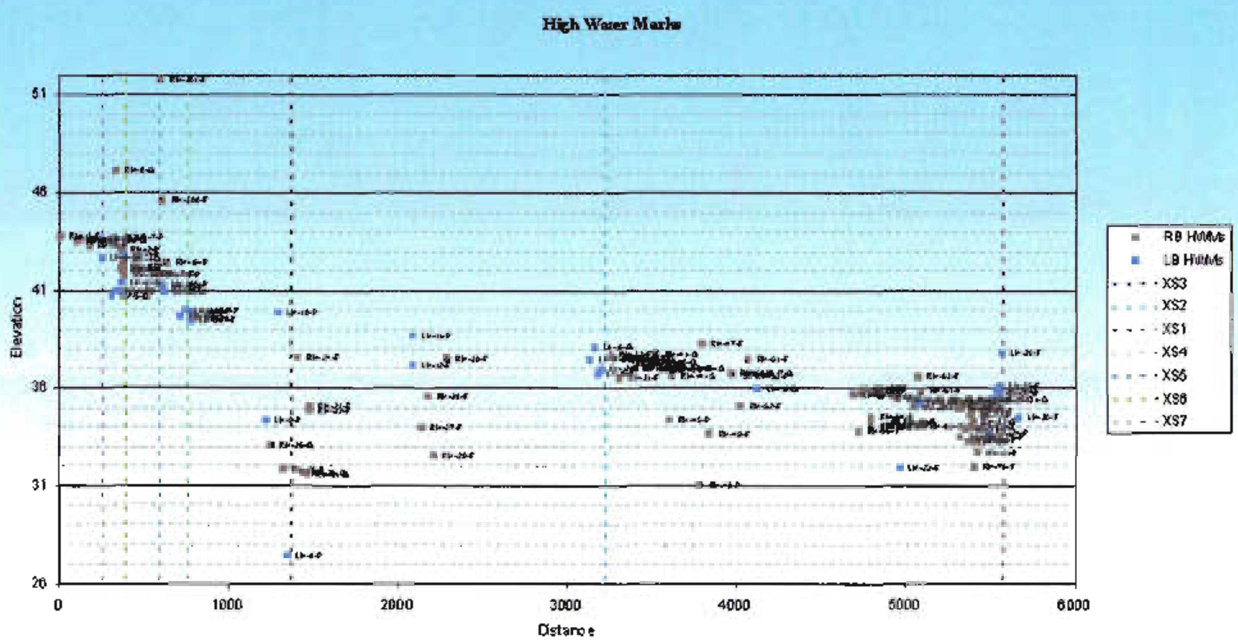
The 1921 flood peak discharge published in Water Supply Paper 1527 by USGS (Stewart and Bodhaine, 1961, WSP 1527) was estimated by James E. Stewart who used the slope-area method in the Dalles reach to make the calculations. Stewart also estimated peak discharges of three other unrecorded historical floods also published by USGS in the same WSP 1527. Stewart estimated the three other unrecorded flood peaks using a stage-discharge rating curve which he created on the basis of his derivation of the peak discharge of the 1921 flood. All of Stewart's estimates are therefore dependent on the assumptions and quality of his slope-area analysis of the 1921 flood peak.

PIE questions the technical validity of using the Stewart peak flood flow estimates for three reasons: 1) it is not appropriate to use the slope area method to estimate the 1921 flood flow in the Dalles reach; 2) the HWMs were not transferred correctly to the Dalles reach; and 3) the coincident flows between Sedro-Woolley and Concrete (at the Dalles) for the Stewart estimates don't match USACE estimates of flood peaks and peaks associated with recent floods.

Application of the slope area method is based on the requirement that the velocity remains constant between cross-sections or the area of the cross-sections does not change. During flood flows in the Dalles reach, the velocities are very high (about 12 fps to 14 fps based on Stewart's estimate of the 1921 flood peak), the velocities vary significantly from section to section, and surge is significant. These factors make application of the slope-area method improper to evaluate flows from HWM's in the Dalles reach. The velocity-head difference between sections, the transition losses due to velocity changes, and the surging effect on HWM readings were significant for the 1921 flood flow and were not factored into Stewart's estimate of the 1921 peak flood flow. Even Stewart, in a letter (Stewart, 1950) to USGS cautioned the use of this method and suggested others (who continued to finish his works) the need to verify velocities not varying between sections and surging not significantly affecting the HWM readings. (Note: Stewart never finished and finalized his report. The USGS WSP 1527 was published in 1961 after Stewart was no longer with USGS.) In late 2004, a year following the flood of record that occurred in October 2003, the USGS conducted an N-Verification study (Mastin and Kresch, 2005) in the Dalles reach, in the section Stewart cautioned about. The results of the USGS study clearly indicate Stewart's concerns were justified, as USGS-surveyed high water marks varied by over 12 feet in one cross-section, and between 2 to 7 feet in all cross sections (see chart below). This data indicates significant surge occurs in the Dalles reach and the use of this reach to calculate discharge estimates using the slope area method is clearly not consistent with accepted engineering principles.

Stewart's transfer of HWMs was not clearly documented and likely not accurately performed. Stewart's high water marks for the other three unrecorded floods were at a hotel located "about one mile upstream" (Stewart and Bodhaine, 1961). The distance between the hotel and the Dalles is over 2.5 miles, while Stewart incorrectly assumed only one mile (Stewart and Bodhaine, 1961). The water surface gradient PIE developed using a calibrated HEC-RAS model is over 5 ft/mile at the Dalles and at the hotel site. Stewart assumed 2 ft/mile (Stewart and Bodhaine, 1961). The HWM conversion could not be substantiated by any Stewart or USGS documents. USGS has stated it could not find in its files any supporting evidence of Stewart's HWMs and conversion calculations (PIE, 2005a).

High Water Marks Profile Plot
at Concrete USGS Gauge Location



2

2003 Flood High Water Marks Surveyed by USGS (source: USGS)

Stewart's estimate for the 1897 flood at Concrete is 45 percent higher than his estimate at Sedro-Woolley (275,000 cfs versus 190,000 cfs). The difference of peak flows at these two locations during a large flood should be within a four percent range, according to both the USACE and PIE's hydraulic models and observed floods. See the tables below for a comparison of concurrent flood peaks at these two locations. Stewart did not address this issue of coincident flow variation between the estimates for Concrete and Sedro-Woolley. Very little in-valley storage exists between these two gage locations. Both of the Stewart discharge estimates are published by the USGS, but they contain a conundrum that is central to the use of the historic flood data, and must be resolved. If the Sedro-Woolley estimates are accurate, then the Concrete discharge estimates are too high. On the other hand, if the Concrete estimates are accurate, then the written record would document much higher flood damage in the lower basin. This is not the case and therefore, we believe Stewart's estimates of the discharge at Sedro-Woolley are closer to the actual discharge.

USACE Estimated Peak Flows*

Frequency	Flow at Sedro-Woolley (cfs)	Flow at Concrete (cfs)	% Difference
10-year	122,190	117,430	-3.9%
50-year	192,830	185,650	-3.7%
100-year	235,300	226,400	-3.8%
500-year	341,600	345,630	1.2%
		Average Difference	-2.6%

*See USACE, 2005

Recent Recorded Peak Flood Flows

Flood Date	Flow at Sedro-Woolley* (cfs)	Flow at Concrete (cfs)	% Difference
Nov 1990	154,400	149,000	-3.5%
Nov 1995	154,300	159,000	3.0%
Oct 2003	157,450	164,000	4.2%
		Average Difference	1.2%

*Modeled by PIE (PIE, 2004e and 2005c)

USGS Estimated Peak Discharges of Four Unrecorded Floods

Flood Date	Stewart Estimate at Sedro-Woolley* (cfs)	Stewart Estimate at Concrete* (cfs)	% Difference
Nov 19, 1897	190,000	275,000	44.7%
Nov 30, 1909	220,000	260,000	18.2%
Dec 30, 1917	195,000	220,000	12.8%
Dec 13, 1921	210,000	240,000	14.3%
		Average Difference	22.5%

*See Stewart and Bodhaine, 1961

USGS Analyses Using Data for the October 2003 Flood

Comment 4

Therefore, USGS (Mastin and Kresch, 2005) concluded that the December 1921 peak discharge of 240,000 cfs and the peak discharges for the other three historic floods estimated by Stewart were reasonable.

Response 4

This USGS analyses (Mastin and Kresch, 2005) was designed to calculate a Manning's n value, and then back-calculate Stewart's historic flood discharge estimates based upon the slope area method for the Dalles reach. The analyses suggested a value of 0.024 be used in this reach. A roughness coefficient of 0.024 corresponds to a major natural stream with no boulders or brush (Chow, 1959. Table 5-6 and Figure 5-5). PIE believes USGS n value is unreasonably low for the Skagit River though the Dalles under flood flow conditions which is a natural river channel with densely vegetated overbanks (see picture below). The n value verification performed by USGS is not a process appropriate to justify the applicability of the slope area method to estimate the 1921 flood peak discharge in the Dalles reach (see Response 3). Likewise, using a Manning's n evaluation to conclude that the peak discharges for the other three historical floods estimated by Stewart were reasonable assumes the HWMs were accurately converted from the Concrete location in the first place. Because this precursor issue was not addressed, the premise of the USGS n verification study – indirectly validate Stewart's historic unrecorded peak flow estimates by verifying the roughness coefficient in the Dalles reach – does not make sense. The analysis did not evaluate the accuracy of Stewart's HWMs even though these HWMs were not observed at the Dalles but converted with an estimated distance and water surface gradient from the HWMs Stewart observed at a hotel on the bank of the Baker River, about 2.5 miles upstream from the Dalles (see Response 3).

B. View from Dalles Bridge looking downstream at the upstream end of island at Skagit River near Concrete, Washington, August 2004. (Photograph taken by D. Miller, U.S. Geological Survey, 2004.)

Source: Mastin and Kresch, 2005.

PIE HEC-RAS Model

Comment 5

A review of this HEC-RAS model indicated that some cross sections were subdivided in places they should not have been, that high n values were used in the main channel for some cross sections, and that n values increased with elevation at a few cross sections around the Dalles Bridge. The peak discharges estimated by PIE for the four historic floods (Table 1) also assume that the peak stages reported by Stewart are applicable to a location 200 feet upstream of the present gage location and that there is up to 2 feet in fall in water surface elevation between these two locations for major floods. These issues decrease the credibility of the PIE estimated discharges for the four historic floods.

Response 5

PIE addressed important, substantive engineering issues that the USACE hydraulic analysis left unanswered. Subdividing cross sections to assign appropriate n values for varying vegetation coverage of the channel, banks and overbanks is an accepted and appropriate HEC-RAS modeling technique. Likewise, assigning various n values with elevation in our HEC-RAS unsteady flow model is the right approach to use in today's state-of-the-art modeling. In making this comment, PIE questions whether the reviewer has expertise with hydraulic analysis using the HEC-RAS model.

The statement that the old staff gage location is at 200 feet upstream from the current Concrete gage site, and the Stewart reported gage heights were at the old staff gage site *is not an assumption, but is documented from USGS published WSP 1527 (Stewart and Bodhaine, 1961) and a USGS internal memorandum prepared by F. J. Flynn in 1954 (Flynn, 1954). The significant water surface elevation change of up to 2 feet between these two gages not only was demonstrated from our HEC-RAS modeling, but was also demonstrated by the 2003 HWMs surveyed by USGS and provided in the USGS analyses (Mastin and Kresch, 2005) (see chart in Response 3). This significant elevation differential was not accounted for in estimating the four historical flood peaks in any of the USGS studies or WSP 1527.*

Conclusions

Comment 6

Given all this information, the historic peak discharges published by USGS in 1961 should not be revised.

Response 6

On the contrary, the historical peak discharges at the Concrete gage published by USGS in 1961 (Stewart and Bodhaine, 1961) were based on unsupported HWMs and inapplicable slope area methodology as discussed in Responses 3, 4, and 5. The published peak flow discharges as estimated by Stewart at the Dalles should not be used in any of the hydrologic calculations.

Unregulated Frequency Analysis

Use of data for the period 1925 to 1943

Comment 7

However, the annual peak flows greater than 100,000 cfs indicate a greater effect of regulation. For example, the observed value of 147,000 cfs for the February 1932 flood would be about 218,000 cfs for unregulated conditions if estimated from the orange line.

Analyses in USGS Water Supply Paper 1527 (Stewart and Bodhaine, 1961) are similar indicating that the reservoirs in place in 1932 reduced the February flood from 182,000 cfs to 147,000 cfs. It appears that the larger floods in the period 1928 to 1943 were sufficiently affected by regulation and should not be included in an **unregulated** frequency analysis.

Response 7

If the two estimates for the February 1932 flood for unregulated conditions (218,000 cfs from the orange line plotted by the reviewer and 182,000 cfs from the USGS WSP 1527) are accurate, then the incidental flood control storage required to reduce these estimates to the flow recorded at the Concrete gage would be huge, around 280,000 acre-ft and 140,000 acre-ft respectively. This amount of flood control storage did not exist in 1932 in the Skagit and the Baker River systems combined. Therefore, both peak flow estimates for unregulated conditions are grossly overestimated. In 1932, only small amounts of incidental flood control storage were provided by Lower Baker, George, and Diablo Dams, while Ross and Upper Baker Dams that provide the current specific flood control storage total of 194,000 acre-feet for the basin were not even in construction.

The reviewer infers that PIE used suspect data (1928 to 1943) to influence the hydrologic analysis, based on the premise that the effects of dam regulation during this period are not known. But consider that the observed flood peaks at Concrete larger than 100,000 cfs in the period 1928 to 1943 only occurred in four of the years, 1932 to 1935. The observed peak and peak one-day values, and the peak to one-day flow ratios are listed below:

1932 – 35 Observed Peak Flow Values at Concrete

Water Year	Observed Peak Flow (cfs)	Observed One-Day Flow (cfs)	Peak to One-Day Flow Ratio
1932	147,000	129,000	1.14
1933	116,000	97,800	1.19
1934	101,000	85,000	1.19
1935	131,000	120,000	1.09

These peak to one-day flow ratios are very reasonable for unregulated flows and well within the range of all observed or synthetic unregulated flood values. PIE also analyzed the storage regulation effects using water volumes stored in the Lower Baker Dam during the 1935 flood that were reported in the newspaper. This evaluation concluded that the regulation effects were about two percent (PIE, 2005d). This example demonstrates how the USACE hydrologic analysis did not include evaluation of the available data at a level of detail that has an important affect on the outcome. By not performing this evaluation, the USACE also loses important one-day flow information. PIE's analysis has demonstrated that the regulation effect during these years was minimal. It is PIE's opinion this data is too valuable to ignore, extends the record with quality data, adds depth and substance to the overall hydrologic analysis, and provides a better sense of the flooding characteristics of the river.

Comparison of USACE (2005) and PIE (2005) unregulated analyses

Comment 8

The lower and upper 50 percent confidence limits for the USACE base flood estimate of 284,000 cfs are 249,000 cfs and 324,000 cfs, respectively. The PIE base flood estimate of 246,300 cfs is only slightly below the lower 50-percent limit.

Response 8

The lower and the upper confidence limits for the PIE unregulated base flood estimate of 246,300 cfs are 211,900 cfs and 297,700 cfs, respectively (PIE, 2005d). The USACE (USACE) unregulated base flood estimate of 284,000 cfs is well below the upper limit and within these confidence limits.

Regulated Frequency Analyses

Comment 9

This analysis only used the 49 observed regulated peak flows from 1956 to 2004 and provided a base flood discharge of 198,500 cfs. That is, the six synthetic events that were originally derived from the unregulated frequency analysis were not actually used in shaping or defining the upper end of the regulated frequency curve. This is surprising given all the discussion and analyses related to the four historic floods.

Response 9

The 49 observed peak flows from 1956 to 2004 are the only regulated flow data available for the analysis. The six synthetic events were derived using all the USGS measured data (water years 1924 to 2004) as well as estimates for the four historical floods. PIE's estimates for the four unrecorded floods was calculated by application of the Corps' HEC-RAS model using Stewart's HWMs at the Concrete staff gage published in the WSP 1527. The reasons for not using Stewart's peak flow estimates for the four unrecorded floods is discussed in Responses 3, 4, 5, and 6.

When the Stewart flows are included in the analysis, the upper end of the frequency curve bends upward. It is PIE's evaluation that this curve is skewed. If PIE's estimates for the four unrecorded floods is used the upper end of the frequency curve remains straight. The synthetic events fall close to the frequency curve and therefore will not have a shaping effect on the curve. Therefore, there is no surprise with PIE using (or not using) the synthetic events because they do not have the effect of skewing the analysis.

Comment 10

The PIE curve in Figure 3 has a skew of about zero and is basically a straight line on lognormal probability paper. This is unreasonable for a regulated frequency curve. As the flood event becomes more extreme, the reservoir system has less ability to store and regulate the event so that the regulated frequency curve should become concave upward (positive skew) and tend to converge with the unregulated frequency curve when reservoir capacity is exceeded.

Response 10

PIE agrees with the reviewer that the regulated frequency curve should converge with the unregulated frequency curve when reservoir capacity is exceeded (during an extreme flood). But the reviewer concluded too soon based on incomplete data that the PIE curve is not reasonable by projecting non-convergence of the PIE regulated and unregulated curves based on his own plots.

PIE provided the reviewer with PIE's routing models including HEC-5 and HEC-RAS. It is apparent from this response that the reviewer did not run these models to determine the shape of the curves. Using these models, it is straight forward to demonstrate whether PIE's regulated and unregulated curves converge or not when the reservoir capacity is exceeded during an extreme event. For example, PIE routed four extreme events with unregulated flows at 1.50, 1.75, 2.00, and 4.00 times the base flood flow at Concrete. The routing results indicate the ratios of the regulated and unregulated peaks to be 0.81, 0.87, 0.91, and 0.98, respectively, for these four events. This demonstrates that PIE's regulated and unregulated frequency curves converge, eventually approaching the ratio of 1.00. See the following table for the ratio of PIE regulated to unregulated peak flows relating to the flood volume vs. flood control storage volume.

Flood Event	Ratio of Regulated to Unregulated Peak Flows	Flood Volume above Control Flow* (acre-ft)	Ratio of Flood Control Storage to Flood Volume**
10-year	0.80	298,000	0.65
50-year	0.79	673,000	0.29
100-year (BF)	0.78	878,000	0.22
500-year	0.78	1,530,000	0.13
1.50 x BF	0.81	1,860,000	0.10
1.75 x BF	0.87	2,230,000	0.09
2.00 x BF	0.91	2,600,000	0.08
4.00 x BF	0.98	5,500,000	0.04

*Control flow of 90,000 cfs at Concrete gage

**Total flood control storage at Ross and Upper Baker is 194,000 ac-ft.

Comment 11

This implies that the PIE regulated frequency curve will never converge to the unregulated frequency curve no matter how large the event. This is not reasonable. The data used in plotting Figure 5 are given in Table 2 which includes the ratio of regulated to unregulated flood discharges.

Response 11

See Response 10.

Comment 12

Table 2. Summary of regulated and unregulated flood discharges in cubic feet per second (cfs) and their ratios for the USACE and PIE engineering analyses.

Event	USACE regulated	USACE unregulated	USACE ratio	PIE regulated	PIE unregulated	PIE ratio
10-year	117,430	158,000	0.743	125,400	145,700	0.861
50-year	185,650	242,000	0.767	176,000	214,100	0.822
100-year	226,400	284,000	0.797	198,500	246,300	0.806
500-year	345,630	398,000	0.868	253,600	329,400	0.770

Response 12

The Reviewer's table is not correct. PIE regulated numbers are 116,900 cfs, 169,000 cfs, 192,300 cfs, and 256,600 cfs (see Table 9, PIE, 2005d), for the 10-, 50-, 100-, and 500-year floods, respectively, instead of those used by the reviewer as listed in Table 2 above. The reviewer's table should be corrected as shown below:

Event	USACE regulated	USACE unregulated	USACE ratio	PIE regulated	PIE unregulated	PIE ratio
10-year	117,430	158,000	0.743	116,900	145,700	0.802
50-year	185,650	242,000	0.767	169,000	214,100	0.789
100-year	226,400	284,000	0.797	192,300	246,300	0.781
500-year	345,630	398,000	0.868	256,600	329,400	0.779

As can be seen, PIE regulated-to-unregulated ratio is nearly bottomed out and trending back toward 1. See Response 10 for further discussion on other flood ratios and the converging trend of regulated and unregulated frequency curves.

Comment 13

As shown in Table 2, the ratio of the regulated to unregulated flood discharges for the PIE analysis is actually decreasing as the flood event becomes more extreme while the USACE ratio increases as it should.

Response 13

PIE analysis shows a very typical convergence/divergence trend of the regulated and unregulated frequency curves for a regulated stream. For a regulated stream basin with flood control storage dams like the Skagit River, the regulated and unregulated frequency curves typically show the following trends of divergence (the ratio of regulated and unregulated flows decreases below 1.00) and convergence (the ratio increases towards 1.00) as the flood magnitude increases:

A. *The curves are identical (converged) at the lower flows when flood control storage is not engaged.*
B. *The curves begin diverge as flood control storage becomes more significant in proportion to flood volume and the storage is used to reduce flood peaks.*
C. *The curves begin to converge as flood control storage becomes less significant in proportion to flood volume and the storage has a lessening impact in reducing the flood peak.*
D. *The curves converge when the flood control storage is used up prior to the arrival of flood peaks.*

The PIE ratio changes in a manner consistent with this phenomenon.

Also see Responses 10 and 12.

Comment 14

As shown in Table 2, the regulated 100-year or base flood discharge from the PIE analysis is 198,500 cfs while it is 226,400 cfs from the USACE analysis. The PIE base flood discharge is 12.3 percent less than USACE. USACE (2005) provided confidence limits for their regulated frequency curve in terms of one and two standard deviations. Assuming that the 50-percent confidence limits for the USACE (2005) regulated frequency curve are estimated as 204,000 cfs and 252,000 cfs, respectively. The PIE regulated estimate of 198,500 cfs is only slightly below the lower 50-percent limit.

Response 14

The lower and the upper confidence limits for the PIE regulated base flood estimate are 166,900 cfs and 250,300 cfs, respectively, based on the frequency analysis (PIE, 2005d). The USACE regulated base flood estimate of 226,400 cfs is well below the upper limit and within these confidence limits.

Comment 15

Given the uncertainty in the historic and observed flood data, the uncertainty in converting the unregulated flows to regulated conditions, and the uncertainty of the regulated frequency analysis, a difference of 12.3 percent in the regulated base flood discharges estimates as determined by PIE and USACE is not significant from a hydrologic viewpoint.

Response 15

The 12.3 percent difference at Concrete results in peak flow difference of approximately 25,000 cfs in the reach through the Cities of Mount Vernon and Burlington as well as areas downstream. In this reach the communities are protected by a system of levees more than 40 miles in length, of which approximately eight miles protect urban areas. An additional 25,000 cfs would require at a minimum all the levees protecting urban areas to be more than 2.5 feet higher and would require placement of more than one million yards of material. The incremental cost and environmental impacts would be prohibitive and this work would not be insignificant to the communities.

The communities' least cost alternative is to construct additional storage off channel in a natural storage area. The additional storage needed to control the 100-year flood using PIE hydrology is about 60,000 ac-ft. This amount of storage is available within several combinations of flood control measures currently being studied and it appears to be affordable as well. The additional storage needed to control the 100-year flood using the Corps hydrology is estimated at about 240,000 ac-ft. This is a 300 percent increase over the PIE estimate and is not available anywhere in the system using any combination of flood control measures.

This 12.3 percent "insignificant difference" is in fact the key to whether a basin-wide flood project that provides 100-year protection to the Valley's urban areas can or cannot be built. See Response 1. This argument can be turned around: if the difference is not significant, why not use the PIE hydrologic analysis.

Conclusions

Comment 16

The historic peak flows used by USACE (2005) are based on published USGS estimates that have recently been verified by USGS (Mastin and Kresch, 2005). The PIE estimated historic flood discharges are based on a HEC-RAS model that used inappropriate subdivision of the cross sections and high n values.

Response 16

The historical peak flows at Concrete estimated by Stewart and published by USGS are based on poorly documented HWMs, improperly transferred HWMs and the slope area methodology that is not applicable to the Dalles reach. These estimated flows at Concrete also contradict with the flows at Sedro-Woolley estimated by Stewart for the same events. These flows should be modified, based on PIE's analysis, documented in this response and in the technical work conducted by PIE.

The PIE estimated historical flood discharges are based on current state-of-the-art methodology and are consistent with the Stewart estimates at Sedro-Woolley, except for the 1897 flood.

See Responses 3 - 6 for more detailed discussion.

Comment 17

The use of the PIE historic peak flows only decreases the unregulated base flood discharge estimate by 10 percent, well within the uncertainty of the historic peak discharges.

Response 17

We would note that this response implies the PIE estimated unregulated base flood discharge should therefore be acceptable for use in the FIS. See Response 8.

Comment 18

PIE used observed annual peak flows during the period 1925 to 1943 for their unregulated frequency analysis and the larger peak flows in this period are considered regulated. USACE did not use these data and that is a more reasonable approach.

Response 18

PIE believes it is valuable to engage in a comprehensive evaluation of all recorded data points available and based upon an in-depth look at each data point make a decision if it should be used, adjusted or discarded. PIE does not agree with the USACE's decision to discard two decades of continuous gage data without an effort to at least adjust the data to reflect potential storage effects. See Response 7. PIE does not agree with the USACE's logic of incorporating the inaccurate Stewart estimates into the data set but not the recorded data. We believe that the technical problems associated with many aspects of the Stewart estimates far exceed the technical problems associated with accepting or performing appropriate corrections to be able to accept the recorded data.

Comment 19

The PIE unregulated base flood discharge estimate is only 13.3 percent lower than the USACE estimate and only slightly outside the 50-percent confidence limits of the USACE estimate. The difference in the two estimates is not statistically significant.

Response 19

If the difference is not statistically significant to FEMA, but makes a significant practical difference to the communities, then FEMA should logically accept the PIE analysis for use in the FIS.

Comment 20

The historic peak flows, converted to regulated conditions, were not used by PIE in their regulated frequency analysis. This is not a defensible approach.

Response 20

PIE's approach is very defensible. See Response 9 for detailed discussion.

Comment 21

The slope of the PIE regulated frequency curve is such that it will never converge with the unregulated frequency curve. This is not a reasonable result.

Response 21

The PIE regulated frequency curve does converge. The reviewer did not perform the proper calculations. See Response 10 for detailed discussion.

Comment 22

From a hydrologic viewpoint, a difference of 12.3 percent in regulated base flood estimates is not significant. The PIE regulated base flood discharge estimate is only slightly outside the 50-percent confidence limits of the USACE (2005) estimate.

Response 22

From a practical viewpoint a difference of 12.3 percent makes a significant difference. See responses 14 and 17. Again, if the difference is not significant to FEMA, then FEMA should adopt the PIE hydrology.

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