REVIEW OF

SKAGIT RIVER BASIN HYDROLOGY DRAFT REPORT
EXISTING CONDITION

PACIFIC INTERNATIONAL ENGINEERING PLLC
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REVIEWED BY:

NORTHWEST HYDRAULIC CONSULTANTS INC
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Summary of Findings

This document provides a review of Pacific International Engineering's "Skagit River Basin Hydrology Draft Report - Existing Conditions", August 2007, and its Appendices A through J. The review focuses on PIE's development of unregulated flood frequency curves for the Skagit River near Concrete. Limited review is also provided of regulated flood frequency curves and synthetic flood hydrographs for the regulated condition as developed by PIE.

The peak discharge estimates presented in PIE's August 2007 draft report are substantially lower than flows previously reported by either PIE in December 2005 or by the US Army Corps of Engineers in November 2005. For example, the latest estimate by PIE for the 100-year unregulated peak discharge at Concrete is 227,200 cfs compared with a December 2005 PIE estimate of 246,300 cfs and the November 2005 estimate by the US Army Corps of Engineers of 284,000 cfs. The reduction in 100-year unregulated peak discharge is the result of corresponding substantial reductions in the estimated peak discharges at Concrete for the historic events of water years 1898, 1910, 1918 and 1922. Reductions in peak discharge estimates for the historic events have been made by PIE such that those estimates fit into a framework which is consistent with its hydraulic modeling results at Hamilton, The Dalles, and Sedro-Woolley and which is also consistent with selected unpublished peak discharge estimates at Sedro-Woolley. Of particular note, historic peak discharges at Concrete as presented by PIE are:

- consistent with evidence that the Smith House in Hamilton was not flooded in the historic events of water years 1910, 1918 and 1922. (PIE hydraulic model results, incorporating some aspects of the river's 1911 geometry, indicate that peak flows could not have exceeded 188,000 cfs without flooding the Smith House.)

- consistent with J.E. Stewart's assumption that flows were contained in the channel above The Dalles during the historic events. (PIE hydraulic model results show that at discharges above 180,000 cfs, water would have spilled into the right bank spill channel about 4,000 ft above The Dalles.)

- consistent with PIE hydraulic modeling results that show a 2% increase in peak discharges between Concrete and Sedro-Woolley for the floods of 1990, 1995 and 2003. (To achieve this consistency, PIE makes use of unpublished USGS estimates of peak flows at Sedro-Woolley which are lower than currently published estimates).

PIE's latest analysis makes no use of the USGS estimates of peak discharges for the historic events at Concrete.

We agree in broad terms with PIE's approach to identifying threshold discharges which can be used to estimate an upper limit on peak discharges during the historic events. However, the thresholds determined by PIE are, in our opinion, somewhat too low. Reviews of the threshold values are provided below.

While agreeing with PIE's approach to determining threshold values, we have difficulty in accepting PIE's approach to estimating peak discharges for the historic floods at Concrete. PIE's approach is to take a set of unpublished peak discharges for the historic floods at Sedro-Woolley which fall below the threshold values, and to then transfer those estimates to Concrete. The result is that PIE is, in effect, rejecting USGS peak discharge estimates for the historic events at both Concrete and Sedro-Woolley. It is, in our opinion, unlikely in the extreme, that this approach would be accepted by the Federal agencies in an appeal of the FIS.

Specific review comments on various aspects of PIE's report follow.

Hydraulic Modeling Upstream from The Dalles

PIE's hydraulic modeling results show that at a discharge greater than about 180,000 cfs, water would overflow into a right bank side channel about 4,000 feet upstream from The Dalles. Stewart reports no such overflow during the historic flood events and on that basis PIE assumes that the peak discharges at The Dalles could not have exceeded 180,000 cfs during those floods.

1. PIE's estimate of a threshold flow of 180,000 cfs at which overflow would start has to be regarded as approximate. There is insufficient data on which to base calibration of a hydraulic model above The Dalles. PIE relied on calibration of its model to:

- the recorded peak water levels at the Concrete gage for the floods of October 2003 (166,000 cfs), November 1995 (159,000 cfs), and November 1990 (two peaks of 149,000 cfs and 146,000 cfs)
- seven high water marks reported by the USGS for the flood of October 2003.

Of the seven October 2003 high water marks, five are below the gauge site, one is at the gage site and one is about 200 ft upstream from the gauge site. For all practical purposes **there is no calibration or verification of PIE's hydraulic model for the approximately 4,000 ft reach upstream of The Dalles to the overflow point of the right bank spill channel.**

2. Flow at The Dalles is extremely complex, especially during high flows. Water flowing through The Dalles takes two 90-degree turns above the gage site, resulting in strong eddies, upwellings and extreme turbulence at the entrance to The Dalles. A one-dimensional model, such as HEC-RAS, is **not** suitable for modeling flow through The Dalles. While the model may show good reproduction of water levels at one point within The Dalles (the gauge site) for flows in the range 146,000 cfs to 166,000 cfs, **there is no assurance that the PIE model can accurately predict water surface profiles through The Dalles for either the calibration events or for the higher discharges of interest.**

3. HEC-RAS model parameters developed by PIE show considerable variation along the modeled reach. In-channel values for Manning's roughness vary from 0.028 to 0.038 with abrupt changes from one cross-section to the next for no apparent reason. Roughness below The Dalles, at the top end of the reach used by the USGS for n-verification studies, is 0.028. Roughness through and upstream from The Dalles is taken to be 0.038. Calibration of the model to match water levels at the gauge site appears to have been achieved through a combination of high in-channel roughness and high values for contraction and expansion coefficients. It is inappropriate to use Manning's roughness to represent energy losses in the complex flows at the entrance to and through The Dalles and there is no reason to believe that roughness values above The Dalles should be appreciably higher than below The Dalles. **The Manning's n value used by PIE above The Dalles appears to be unreasonably high. Use of a high Manning's roughness would result in the PIE model overstating water levels and hence understating the discharge at which water spills into the right bank spill channel.** The most recent work by the USGS determined a roughness of 0.0315 for the reach below The Dalles used for n-verification studies. This would be a more reasonable roughness value than that used by PIE.

4. The ultimate purpose of PIE's analysis is to determine an upper limit for the historic flows on the assumption that overflow in to the right bank spill channel did not occur in the 1898, 1910, 1918 and 1922 floods. PIE's model relies on channel surveys from October 2004 and overbank topography from the 1976 FIS. **There is no discussion or consideration in the PIE report of channel changes from the time of the historic floods to present**. While we would not expect there to have been much change in channel geometry given the stability of The Dalles, we would expect this issue to be at least acknowledged.

5. Review of recent LiDAR data from Skagit County suggests that the elevation at which spill would occur may be higher than indicated by 1976 FIS. Based on surveys and mapping from the 1976 FIS, PIE assumed that spill would occur if water levels exceed an elevation of 180 ft. NGVD at the entrance to the spill channel. The LiDAR data, which remain to be ground-truthed, indicate that the spill elevation may be as high as 186 ft. NGVD. **Confirmation of the spill elevation is needed before a firm threshold discharge can be established.**

6. The assumption that overflow to the right bank spill channel did not occur in the historic events is apparently based on a similar assumption by Stewart. We would expect Stewart's 1923 field inspections to have identified spill if it had occurred in the 1922 flood. But we would not necessarily expect Stewart to have identified spill that occurred in the earlier and larger floods of 1898 and 1910. **PIE's reliance on Stewart's assumption that spill did not occur during the earlier historic floods seems misplaced. Irrespective of uncertainties with hydraulic modeling, spill must have occurred in 1898 and 1910 if the reported gauge heights are correct. A paleoflood investigation could perhaps determine whether or not spills occurred through this route during those events.**

7. **Repeating PIE's analysis using a roughness of 0.0315 upstream from the gauge site but maintaining the spill elevation at 180 ft. NGVD results in an increase in the threshold discharge at which spill would start to 193,000 cfs, still significantly lower than published peak discharges for the historic events.**

Hydraulic Modeling in the Vicinity of Hamilton

PIE undertook HEC-RAS hydraulic modeling of flows and water levels in the vicinity of Hamilton to determine the threshold flow at which the Smith House would have flooded for an approximation of the channel conditions of around 1911. PIE's model was calibrated to water levels and discharges from November 1995 and October 2003. The calibrated model was then modified to represent historic conditions using channel planform information from a 1911 Corps survey. PIE's analysis indicates that for the 1911 geometry, the Smith House would have flooded at a discharge of about 188,000 cfs. On this basis, PIE set a 188,000 cfs upper threshold on peak flows for the historic events of 1910, 1918 and 1922.

8. Hydraulic conditions below Hamilton are complex, involving flow through a pronounced meander loop, confinement of flow by the Cockreham levee, overtopping and failure of the levee at some point in the model calibration events, and escapement of flow through slough openings (such as Muddy Creek). Additionally, when comparing current and historical conditions, there have been significant changes over time in both channel width and planform geometry. **All these factors lead to uncertainty in the results of the HEC-RAS simulations.**

9. PIE's model is based on channel geometry data from 1976 with the addition of two recent (2007) cross-sections within Hamilton (at RM 40.0) and just upstream from Hamilton (RM 40.5). Cross-section data are, compared with current standards, widely spaced, with just one cross-section (at RM 39.00) in the hydraulically critical reach in the first meander bend below Hamilton. The cross-section data for the critical reach downstream from Hamilton were thus 20 years old at the time of the 1995 flood. While the current planform geometry of the river channel is very similar to that of 1976, **PIE (2007, Appendix J, Figure 1) shows "low flow channel bank lines" which are appreciably narrower in 2001 than in 1976**. It is not known whether this implies lower channel conveyance today than in 1976, however if PIE's model has been calibrated using channel widths which are too large, this would result in an overestimate of Manning's roughness and possible underestimate of the 1911 threshold discharge for flooding of the Smith House.

10. PIE calibrated their existing conditions model of the Skagit River in the Hamilton vicinity, to the Smith House (RM 40.0) water level observations of November 1995 (peak discharge at Concrete of 160,000 cfs) and October 2003 (peak discharge at Concrete of 166,000 cfs). The calibration effort appears reasonable, though it favors the 1995 event which it matches right on while predicting water levels 0.4 ft. too high for the 2003 event.

Recalibration of PIE's existing conditions model was undertaken by **nhc** to instead favor the 2003 event. Lowering Manning's n-values from 0.035 to 0.033 along Cockreham levee to Hamilton (RM 37.34 to 41.1) resulting in computed water levels matching the 2003 observation but now being about 0.4 ft. too low for the 1995 observation at Smith House. **Applying these lowered n-values to the circa-1911 model in the same manner as PIE's analysis then results in a higher threshold discharge of about 197,000 cfs.**

11. The PIE model makes consistent use of both the levee and permanent ineffective area options within HEC-RAS. Even in locations where there are no levees, the levee option is commonly used for high banks to prevent overbank conveyance (and storage) from occurring until flood levels exceed the bank elevation – or possibly some other lower threshold elevation at which overflows may begin to occur from upstream and inundate the given overbank. We feel this is a reasonable and appropriate modeling technique. The PIE model also makes extensive use of the permanent option for ineffective area to block out lower areas from flood conveyance (but not storage). Sometimes this appears to relate to localized floodplain depressions but at other times it's applied to all area below the levee/bank threshold elevation. An argument could be made that these areas should not be permanently ineffective, that once the threshold bank or levee elevation is exceeded, the overbank should be allowed to convey these flows. Specifically the cross-section at RM 39.8 shows a high bank along the riverfront street with lower elevations further landward. The PIE model sets both a levee and permanent ineffective area at and landward of this location/elevation, resulting in no landward conveyance or storage below this bank elevation and only storage for flood levels above this elevation. It appears in looking at the topography within Hamilton that the river bank upstream (RM 40.0 and above) would overtop prior to this and convey overbank flows into Hamilton at RM 39.8. Therefore **nhc** performed further test runs in which the permanent option for ineffective flow was disabled, allowing conveyance once the "levee" threshold elevation was exceeded. With this change it was necessary to recalibrate the existing condition model, with n-values between the Cockreham levee and Smith House increased from 0.035 to 0.040. The recalibrated model favors neither the November 1995 nor the October 2003 event, and more or less splits the difference in targeting the observed Smith House water levels. Applying the revised permanent ineffective assumption and n-values to the circa-1911 model results in an increased threshold discharge of about 196,000 cfs. Furthermore, if for the circa-1911 simulation we also removed the permanent ineffective options along Cockreham Island (RM 37.34 and 38.15), since pre-Cockreham levee it can be assumed flow could freely convey across the island, the threshold discharge would increase to about 201,000 cfs. Thus, **simply using an alternate representation of hydraulic conditions around Hamilton increases the threshold discharge to 200,000 cfs.**

12. Another further simple sensitivity test performed consisted of reducing the main channel n-values in the Cockreham-Hamilton area (RM 37.34 to 41.1) for the circa-1911 model. **Lowering the channel n-values from 0.035 to 0.030 reduces the water level at Smith House by about 1 ft and raises the threshold discharge to about 212,000 cfs.** By way of comparison, the Corp of Engineers' most recent hydraulic routing of the 100-year flood assumed a channel n-value for this reach of 0.04.

13. In the course of this review we obtained a legible copy of the 1911 Corps of Engineers survey of the Skagit River. Using soundings from the 1911 map to define actual low-flow channel geometry, **nhc** modified PIE's circa-1911 model by inserting two new cross-sections on either side of RM 39.0 and refining sections 38.15 and 39.80. The datum of the soundings is uncertain, but elevations on the Corps map sheets are referenced to "extreme low water of Puget Sound". Using tidal data at various NOAA stations within Puget Sound, a datum shift of about 5 ft would convert mean lower low water (MLLW) to NGVD. Minimum recorded tidal levels at several NOAA stations are on the order of 4 ft below MLLW, which would result in a datum shift of 9 ft to NGVD. Taking the 1911 "extreme low water of Puget Sound" to be somewhere within this range (below MLLW but above the lowest ever recorded) results in an average datum shift on the order of 7 ft, which is also generally consistent with comparison of bank elevations between the 1911 and current maps. **This "refined" 1911 model, using otherwise the same input and assumptions (n-values, ineffective areas, etc.), yields similar results to PIE's 1911 model with a similar threshold discharge.**

14. On the basis of a recent report commissioned by the City of Burlington[1], we accept the claim that the Smith House was not flooded above its main floor level in the historic floods of 1910, 1918 and 1922. As a result of this review, we are also in general agreement with PIE's assertion that the USGS published flows for the historic events are too high. As for a maximum discharge threshold flow for these historic events, the 188,000 cfs threshold determined by PIE is one possibility. However, **on the basis of our review and modeling of alternate representations of the Hamilton reach, a threshold flow in the range 200,000 to 210,000 cfs is equally plausible.**

Consistency with Discharge Estimates at Sedro-Woolley

The peak discharge values at Concrete ultimately "selected" for the historic events by PIE were determined by decreasing by 2% the peak discharge estimates recommended in an internal USGS memo for those events at Sedro-Woolley[2]. The values selected by PIE at Sedro-Woolley and Concrete are summarized, along with the published values, in Table 1 below. The values at Sedro-Woolley were "selected" from a number of alternative estimates to ensure that flows would not exceed the upstream thresholds of 188,000 cfs at Hamilton (the discharge above which the Smith House would have flooded) and 180,000 cfs at The Dalles (the discharge at which flow would have spilled to the right bank spill channel).

[1] Wiss, Janney, Elstner Associates, Inc. Letter report to City of Burlington, 17 August 2007

[2] Riggs, H.C. and W.H. Robinson, *Proposed Revision of Skagit River Flood Peaks*, USGS internal memorandum, 16 November 1950.

Table 1: Historic Peak Discharges (cfs)

	Sedro-Woolley		Concrete	
Year	**PIE (2007) (from Riggs & Robinson)**	**Published**	**PIE (2007)**	**Published**
1898	170,000	190,000	166,600	265,000
1910	190,000	220,000	186,200	245,000
1918	160,000	195,000	156,800	210,000
1922	170,000	210,000	166,600	228,000

15. Estimates of discharges at Sedro-Woolley are known to be generally quite uncertain. Reliance on Riggs and Robinson's estimates for flows at Sedro-Woolley results in a circular argument. Riggs and Robinson's proposed revisions to flows at Sedro-Woolley originated from their proposed revisions to the peak flows at Concrete. Riggs and Robinson state "On the basis of the revised rating curve extension and revised discharges near Concrete all the Sedro-Woolley flood peaks are reduced in value." **PIE is now seeking to reduce flows at Concrete on the basis of unpublished and unaccepted flows at Sedro-Woolley which in turn were based on unpublished and unaccepted reductions in discharge estimates at Concrete. In our opinion, it is extremely unlikely that this approach would be accepted by the Federal agencies.**

16. In recommending use of Sedro-Woolley data to estimate historic peak flows at Concrete, it is not clear that PIE is aware of the basis for the various discharge estimates. **Stewart undertook detailed calculations for the 1922 flood at Sedro-Woolley and given the closeness of Stewart' peak discharge estimate to the discharge threshold at Hamilton, we see no reason to modify that or the 1918 value**. The basis for discharge estimates prior to 1911 (i.e. prior to the Sterling Bend cutoff) is quite weak and there is certainly much greater uncertainty and room for adjustment to the peak flows for 1898 and 1910 (both higher and lower) than for the 1918 and 1922 floods.

17. **PIE states that "Comparison of flood peaks for recent recorded floods in 1990, 1995 and 2003, demonstrates that flows at USGS Concrete gage average 1.6% lower than flows at the USGS Sedro-Woolley gage". This statement is misleading in that there are no recorded flows at Sedro-Wolley for these recent events**. The data quoted by PIE are in fact from computer simulations. Furthermore, these figures come from simulation of flood events under regulated conditions, which generally result in lowering the peak but extending the duration of flood hydrographs. Under unregulated conditions, flood hydrographs can be expected to have a sharper peak and undergo some attenuation in routing from Concrete to Sedro-Woolley. **In our opinion it would be more appropriate to assume a modest increase (say 4%) in the peak unregulated discharges at Concrete relative to Sedro-Woolley.**

Unregulated Discharge Record

Analysis of unregulated conditions presented by PIE includes observed peak flow data from the Concrete gage for the period 1925 -1943. These data are taken to be representative of unregulated conditions on the assumption that "Lower Baker Dam and Diablo Dam had only insignificant incidental regulation effects on the flood flows ....". The period 1925-1943 had few large flood events and hence incorporation of this period in PIE's frequency analysis results in some reduction in the estimate of flood quantiles.

18. PIE's assumption that "Lower Baker Dam and Diablo Dam had only insignificant incidental regulation effects on the flood flows ...." is likely true for many of the relatively minor flood peaks in the period 1925 -1943. However, there is very little qualitative or quantitative information available to support the assumption. Furthermore, **the assumption of "insignificant incidental regulation" may be incorrect for the major flood of 1932.** The USGS (Stewart and Bodhaine, 1961) presents the observed (i.e. regulated) hydrograph and an estimated natural or unregulated hydrograph for the 1932 flood. The instantaneous observed peak flow (treated by PIE as the unregulated peak) was 147,000 cfs compared with the USGS-estimated instantaneous unregulated peak flow of 182,000 cfs. Comparison of the observed and USGS unregulated hydrographs implies regulation by some 105,000 acre-feet of storage in the 36 hours before the observed peak flow at Concrete. Of this, some 65,000 acre-ft is attributed to storage in Lake Shannon (Lower Baker). A contemporary newspaper report (Courier Times, 3 March 1932) states "At Baker River before the flood, water was 36 feet below the top, and at the peak of high water flowed nine feet deep over the top of the dam." That 45-foot change in elevation during the flood represents about 84,000 acre-feet of storage. We have been unable to locate any comparable information on storage change at Diablo Dam during this flood, but note that the project was not operating for hydro-power production at the time. While construction of Diablo Dam was completed in 1930, construction of the power house was not completed until 1935, with power production starting in 1936. During construction of the power house, releases from Diablo would have been via low level outlets to the extent possible, in which case Diablo could easily have stored in excess of 40,000 acre-ft of water during the 1932 flood. We therefore conclude that incidental flood control storage of 105,000 acre-ft could easily have been available at the start of the 1932 flood, consistent with the USGS reconstruction of the unregulated hydrograph.

19. The January 1935 flood is another large event in the 1925-1943 time frame under discussion here. Contemporary newspaper records cited by PIE (Burlington Journal, 8 February 1935) indicate that Lake Shannon stored only about 12,000 acre-ft of water during the event, which, as PIE indicates, would have minimal effect on the peak flow at Concrete. However no information is available for Diablo and **PIE's assumption that Diablo had an insignificant incidental flood control effect in the 1935 flood seems to be speculative.** The USGS analysis for the 1932 flood (Stewart and Bodhaine, 1961), estimated that flood storage at Diablo during that event reduced the peak flow at Concrete by 26,400 cfs. Since the Diablo power plant was still not operational in January 1935, it is possible that Diablo's effect on the 1935 flood was similar to that of 1932. This

assumption, admittedly, is speculative, but PIE's assumption that the effect of regulation is insignificant also seems to be largely speculative.

20. Two other moderately large floods occurred in the 1925-1943 time frame, in 1933 and 1934. These events again occurred after completion of Diablo Dam but before the commencement of power production in 1936 and, as with comment 17 above, it is possible that Diablo's effect on the 1933 and 1934 floods was similar to that of 1932. **On balance, we are of the opinion that PIE's inclusion of observed peak flow data for the period 1925-1943 in their analysis of unregulated flows is inappropriate.** Exclusion of these data from frequency analyses would result in a modest increase in estimates of flood quantiles.

21. We find it worrying that PIE's report fails to mention the USGS reconstruction of the 1932 unregulated hydrograph and was selective in quoting information from the Courier Times article of 3 March 1932 (PIE noted that "water at the peak of high water flowed nine feet deep over the top of Lower Baker Dam", but they failed to note that the Courier Times also reported, in the same sentence, that the water level was 36 feet below the top of the dam before the flood event.). **Failure to cite information that does not support PIE's analysis diminishes the credibility of the report and leaves it open to criticism.** PIE is fully aware of the USGS reconstruction of the 1932 unregulated flood hydrograph but contends that the USGS reconstruction is incorrect (Liou, personal communication, September 2007). PIE's reasoning for discounting the USGS estimate for the 1932 flood should be presented in the report.

Regulated Hydrographs

22. Regulated hydrographs are developed assuming the availability of 120,000 acre ft of flood control storage at Ross and 74,000 acre ft at Upper Baker. As noted in **nhc**'s review of the U.S. Army Corps of Engineers' hydrology report[3], provision of the full amount of flood control storage is not required until December 1 of each flood control season. Many of the large floods in the historical record have occurred early in the flood control season when the required flood control storage is less than that assumed by PIE. **PIE's (and the Corps') analysis likely overstates flood control benefits for current reservoir operating policies.**

23. **PIE's synthetic regulated peak instantaneous discharges at Concrete (PIE 2007, Figure 9) appear to be low relative to historical experience.** For example, the 10-year peak discharge at Concrete for the synthetic regulated hydrograph is 116,000 cfs. This discharge has been exceeded 7 times in the 30 year period since 1978 when current flood control operating policies were adopted. The synthetic data points plot below frequency curves based on the observed data for both the instantaneous (PIE, Figure 9) and one-day (PIE, Figure 10) peak discharges. Since peak flows from the observed

[3] Northwest Hydraulic Consultants, *Skagit River Hydrology Independent Technical Review Final Report,* Report to Skagit County Department of Public Works, April 2007.

record likely benefit from the effects of incidental regulation, the underestimation of peak discharges by the synthetic hydrographs is probably larger than suggested by PIE's Figures 9 and 10.

24. **The comparison of regulated peak flows at Mount Vernon vs Concrete (PIE 2007, Figure 11) suggests that peak flows at Mount Vernon may be understated.** Firstly, peak flows at Concrete are understated (see comment 21 above); hence peak flows for the synthetic events should plot further to the right than shown on PIE Figure 9. Furthermore, as events become larger (larger volume), we would expect a reduction in peak flow attenuation between Concrete and Mount Vernon. PIE Figure 11 shows an increase in attenuation in both absolute and relative terms. This apparent anomaly should be investigated and either corrected or explained in the report.

Potential Way Forward

We agree with PIE's approach to determining thresholds to set upper limits on peak discharges during the historic events of 1910, 1918 and 1922. It is more difficult to justify application of a threshold to the 1898 flood for the obvious reasons that: (a) the Smith House was not built until 1908, and (b) it is possible that spill occurred above The Dalles in 1898 without such spill being recognized by Stewart. While we agree with PIE's general approach on threshold determination, we are of the opinion that the flow thresholds determined by PIE are of the order of 10,000 to 20,000 cfs too low. Nevertheless, application of a threshold, even if raised by 20,000 cfs, would result in a significant reduction in peak discharge estimates for the 1910, 1918, and 1922 events.

The main difficulty we have with PIE's work is selection of unpublished revisions to the historical peak discharges at Sedro-Woolley, which fall close to or below the flow thresholds at Hamilton and The Dalles, and scaling of those data to then estimate revised peak discharges at Concrete. We see no prospect of Federal agencies accepting this approach.

PIE's work is however helpful in that it suggests that greater emphasis could be put on the Sedro-Woolley data in analysis of flood risk on the Skagit. There are at least two possible additional avenues to what in our opinion would be justifiable downward revisions to design discharges:

a) **Exclude data from the 1898 flood from analyses**. If one takes the 1898 peak flow data as being correct at both Sedro-Woolley and Concrete, the event is clearly unusual and should not be used to provide the basis for synthetic flow hydrographs. The reported attenuation in peak flow between Concrete and Sedro-Woolley in the 1898 event is extreme and implies an event with an unusually large peak and an unusually small volume. This is inconsistent with the Corps' characterization of flood hydrographs, as used in the creation of synthetic flood hydrographs, and suggests that this event should be excluded from the analyses.

b) **Base analysis more directly on the Sedro-Woolley data.** The Corps' FIS analysis gives a 100-year peak discharge for Sedro-Woolley of 283,130 cfs. As can be seen from Table 2 below, this seems unrealistically large compared with the **published** peak discharge for the flood of record (220,000 cfs on 30 November 1909). This apparent discrepancy arises because the Corp's analysis does not use the Sedro-Woolley record but essentially transposes the Concrete data to Sedro-Woolley assuming that the flood hydrographs for the historical events at Concrete have similar peak to volume characteristics as the 1995 and 2003 flood hydrographs. The net effect is that the Corps analysis shows minimal attenuation in historical flood peaks between Concrete and Sedro-Woolley. The work on flow thresholds at Hamilton by PIE indicates that the published Sedro-Woolley data are reasonable, despite significant uncertainties about the gage rating. It may therefore be appropriate to incorporate the Sedro-Woolley data directly into the hydrologic analysis instead of relying exclusively on the record from Concrete.

Table 2: Unregulated Peak Discharges

	Concrete		**Sedro Woolley**	
	Discharge (cfs)	**% 100-yr**	**Discharge (cfs)**	**% 100-yr**
1898	265,000	93%	190,000	67%
1910	245,000	86%	220,000	78%
1918	210,000	74%	195,000	69%
1922	228,000	80%	210,000	74%
100-yr	284,000		283,130	