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Memorandum

To: Chehalis River Basin Flood Authority and State Technical Review Team

From: WATERSHED Science & Engineering (WSE) and WEST Consultants (WEST)

Date: 04/17/2012

Re: **Response to State team comments on Chehalis River Hydraulic Model**

Watershed Science & Engineering (WSE) and WEST Consultants (WEST) have developed an HEC-RAS hydraulic model of the Chehalis River, including portions of several significant tributaries (e.g. the Wynoochee, Satsop, Black, Skookumchuck, and Newaukum Rivers). Following a meeting on February 23rd, the model and available documentation were provided to a group of State technical staff for review and comment. Three State reviewers provided detailed written comments on the model: Paul Pickett (DOE), Casey Kramer (WSDOT), and Guy Hoyle-Dodson (DOE). These comments were well formed and generally helpful in identifying areas in the hydraulic model that required additional consideration and/or refinement. The three comment letters (attached) were reviewed and discussed by the WSE-WEST team and a number of modifications were made to the model to address significant concerns. In some cases, no changes to the model were necessary, either because the model was already configured appropriately or because the comment raised questions beyond the scope of the current study. Our general responses to the reviewer's comments are provided below. These responses will also be discussed further with the individual reviewers to ensure that we are all comfortable moving ahead with the Chehalis River Basin alternatives analysis using the resulting (refined) model.

RE: Paul Pickett comment letter of 3/30/2012:
Mr. Pickett's comments focused primarily on the hydrologic data proposed for use in the evaluation of flood relief alternatives. He noted that flood events in a basin as large and complex as the Chehalis Basin can come in many different forms and that a comprehensive analysis of flood relief alternatives would require a range of design events to be simulated. However, in our response below we provide data showing that the largest flood events (i.e. the top 10 floods) observed in the Chehalis basin in the past 80 years have similar enough characteristics to make the proposed design event modeling approach reasonable for the current effort. Furthermore, we note that the hydrology for the current study was done and widely reviewed as part of the concurrent Corps project and using the same hydrologic methodology as that study will maintain consistency between the modeling efforts. However, in an effort to provide a more robust and useful analysis, we offer a recommendation to use hydrologic data for the calibration events (1996, 2007, and 2009) to augment the design event evaluation.

In addition to comments on the proposed hydrologic data, Mr. Pickett offered a number of suggestions for improving the evaluation and presentation of "Model Quality" metrics. We have reviewed these comments and find them to be well stated and helpful. We will endeavor to provide additional information on model quality including expanded reporting of model uncertainties, as suggested, when reporting the results of the alternatives analysis.

<u>Detailed Response to “Sensitivity to Hydrology”</u>

Mr. Pickett presented a very useful analysis of the high variability in flood coincidence of contributions from major tributaries in the Upper Chehalis River (above the flow gage near Grand Mound). We agree that multiple hydrologic scenarios of inflows from the major tributaries are possible that would result in a similar magnitude of high flow event for the Chehalis River near Grand Mound.

The hydrologic methodology that WEST used to develop the synthetic flood events for their current U.S. Army Corps of Engineers (USACE) study is similar to the one used by the USACE in the 2003 General Reevaluation Study (updated in 2010). The essential feature of the approach was to develop synthetic flood hydrographs at various locations throughout the basin that together would generate 1.5- to 500-year flood events for the Chehalis River near Grand Mound. The flood magnitude (recurrence frequency) of the basin-wide synthetic events is evaluated using the flow gage site on the Chehalis River near Grand Mound. The coincident relationships for peak flows between the Grand Mound gage and upstream gages were determined using all concurrent annual peaks, which provide a systematic and objective method to define the long-term average coincidence between a synthetic peak discharge near Grand Mound and the coincident inflow from an upstream tributary or from the headwaters of the Upper Chehalis River.

In Mr. Pickett’s comment letter he plots the correlation between the annual peak discharges near Grand Mound and near Doty with and without inclusion of the December 2007 event. The figure shows that for flows in the Chehalis River near Grand Mound less than about 45,000 cfs, roughly the peak discharge of a 10-year event (Table 1), the two regression curves are relatively close to each other. For flows that exceed about 45,000 cfs, the regression curves depart significantly. Mr. Pickett expressed concern that the higher ratio of flows near Doty to flows near Grand Mound might result in unreasonably large contributions from the upper watershed (above Doty), even though this is only seen in some of the observed flood events.

To evaluate and respond to Mr. Pickett’s concern we analyzed data from the top 10 annual peaks at the Grand Mound gage and the corresponding peaks at major upstream gages. Our key finding is that a large flood event near Grand Mound cannot occur if a large event does not occur in the headwaters above Doty. Table 2 summarizes available USGS peak flow data for the Chehalis River basin. This table shows the top 10 flood events recorded by the USGS at the Grand Mound. Of these, two occurred in the 1930s when none of these other major USGS gages in the basin was in operation. Of the remaining eight largest flood events at Grand Mound:

1) All eight had a corresponding flood on the Chehalis at Doty that was in the top 10 of all time at that location.
2) Seven of the eight had a flood on the South Fork Chehalis River that was in the top 10 at that location.
3) Seven of the eight had a flood on the Newaukum River that was in the top 10 at that location.
4) Only four of the eight had a flood on the Skookumchuck River that was in the top 10 at that location.

Furthermore, review of the concurrent USGS gage records for Doty and Grand Mound shows that of the top 10 historical flood events at Doty, eight were also in the top 10 events of all time at Grand Mound. Similarly, of the top 10 events on the South Fork Chehalis River and the Newaukum River 7 were also among the top 10 events at Grand Mound. However, it can be seen that of the top 10 flood events on the Skookumchuck River only four were in the top 10 flood events at Grand Mound. Looking in more detail at the Skookumchuck gage records it can also be seen that the 2nd highest flow of all time on the Skookumchuck was only the 24th highest flow at Grand Mound and the 4th highest flow on the Skookumchuck was only the 23rd highest flow in the USGS record at Grand Mound.

From these data, we can make the following observations:

1) A large flow (herein defined as among the top 10 highest peaks recorded) on the Chehalis at Grand Mound has never happened without a correspondingly large flow on the Chehalis River at Doty.

2) A large flow at Doty is a reliable (although not perfect) indicator of a large flow at Grand Mound.

3) A large flow on the Chehalis at Grand Mound can happen with or without a significant flow contribution from the Skookumchuck River.

4) A large flow on the Skookumchuck is not a very good indicator of large flows at Grand Mound.

5) Peak flows on the Newaukum and South Fork are similarly correlated to the flows at Grand Mound, less so than the Doty flows but more so than the Skookumchuck flows.

Using the top 10 flows at Grand Mound as a representative and sufficiently large sample of basin wide flood events, we see that the average contributions from Doty, South Fork, Newaukum, and Skookumchuck during these events are 45%, 17%, 19%, and 14% of the Grand Mound peak. In his comments Mr. Pickett noted that the preliminary proposed design flow hydrology had ratios of 44%, xx% (South Fork is under review), 17%, and 14%, respectively, for these locations. The proposed design flow ratios appear to be very reasonable given the data in Table 2 and the observations listed above. Figure through Figure show the distributions of flood return periods across the entire basin for the February 1996, December 2007, and January 2009 events. For the January 2009 event, a flood event greater than the 100-year peak discharges occurred in the Skookumchuck and Newaukum Rivers. However, the corresponding flows near Doty and near Grand Mound are only a 12-, and 15-year event, respectively. Thus, while this event is a good example that portions of the basin can see extreme floods while other portions see smaller flood events it also supports the conclusion that a basin-wide extreme flood (as determined using the gage at Grand Mound) is only possible with a large contribution from the Upper Chehalis basin.

We feel that these additional analyses indicate that the coincident relationships determined from all concurrent annual peaks between the Grand Mound gage and the upstream gages provide a reasonable representation of the large flood events in the Upper Chehalis River basin. However, we agree with Mr. Pickett that a high variability in storm timing and magnitude exists in the Chehalis River basin. To evaluate the sensitivity of storm variability, we recommend that the hydraulic model evaluations of flood relief alternatives be run for both the synthetic hydrographs and for the observed February 1996, December 2007, and January 2009 flood events. While we believe that the design event does a reasonable job of characterizing large, basin wide, floods the addition of the historical flood events provides a range of alternative hydrologic conditions that have been seen in the recent past and are useful for a more robust evaluation of flood relief alternatives.

Table 1. Expected Probability Flood Frequency Natural or Unregulated Peak Discharges (in cfs) at Fully Gaged Active Sites

Recurrence Interval (yrs)		Chehalis River nr Doty	Newaukum River nr Chehalis	Skook. River nr Centralia	Chehalis River nr Grand Mound	Chehalis River at Porter	Satsop River nr Satsop	Wyn. River above Save Ck nr Aberdeen	Wyn. River above Black Ck nr Montesano
		12020000	**12025000**	**12026000***	**12027500**	**12031000**	**12035000**	**12036000**	**12037400**
Annual Peak Recurrence (yrs)	**1.5**	8,155	5,160	3,400	21,519	25,109	21,751	11,300	15,100
	2	9,900	6,206	4,230	25,659	29,651	25,936	13,000	17,700
	5	15,110	8,674	6,390	36,917	42,160	35,644	17,500	23,900
	10	19,412	10,253	7,920	45,352	51,678	41,742	20,700	28,000
	20	24,281	11,732	9,450	54,239	61,840	47,382	24,000	31,900
	50	31,906	13,607	11,500	67,091	76,794	54,432	28,400	37,000
	100	38,775	14,995	13,200	77,844	89,514	59,588	32,100	40,800
	200	46,828	16,370	15,000	89,674	103,733	64,642	36,000	44,800
	500	59,627	18,187	17,400	107,184	125,153	71,242	41,600	50,100

*A substitute for Station 12026150 for unregulated flood flow statistics only

Table 2: Comparison of USGS Recorded Peak Flows for Key Gages in the Chehalis River Basin

Chehalis at Porter				Chehalis near Grand Mound			Skookumchuck at Bucoda				Newaukum				SF Combined (extended w Doty)				Chehalis near Doty			
Date	Flow (cfs)	Rank [1]	% [2]	Date	Flow (cfs)	Rank [1]	Date	Flow (cfs)	Rank [1]	% [2]	Date	Flow (cfs)	Rank [1]	% [2]	Date	Flow (cfs)	Rank [1]	% [2]	Date	Flow (cfs)	Rank [1]	% [2]
12/05/2007	102000	1	129%	12/04/2007	79100	1	12/03/2007	3600	55	5%	12/03/2007	12900	3	16%	12/03/2007	20710	1	26%	12/03/2007	63100	1	80%
02/09/1996	80700	2	108%	02/09/1996	74800	2	02/08/1996	11300	1	15%	02/08/1996	13300	1	18%	02/08/1996	9540	4	13%	02/08/1996	28900	2	39%
01/11/1990	60400	4	88%	01/10/1990	68700	3	01/10/1990	8540	8	12%	01/09/1990	10400	6	15%	01/09/1990	9880	3	14%	01/09/1990	27500	3	40%
11/25/1986	45900	9	89%	11/25/1986	51600	4	02/01/1987	6470	22	13%	11/24/1986	10700	5	21%	11/24/1986	6430	12	12%	11/24/1986	17900	9	35%
01/09/2009	68100	3	134%	01/08/2009	50700	5	01/08/2009	10500	3	21%	01/07/2009	13000	2	26%	01/08/2009	11660	2	23%	01/08/2009	20100	7	40%
01/22/1972	55600	5	113%	01/21/1972	49200	6	01/21/1972	8190	11	17%	01/21/1972	9770	10	20%	01/20/1972	6540	10	13%	01/20/1972	22800	4	46%
Data not available				12/29/1937	48400	7									Data not available							
11/26/1990	43000	11	90%	11/25/1990	48000	8	11/25/1990	8400	9	18%	11/24/1990	10300	7	21%	11/24/1990	7400	7	15%	11/24/1990	20600	6	43%
Data not available				12/21/1933	45700	9									Data not available							
12/05/1975	48100	7	107%	12/05/1975	44800	10	12/04/1975	6110	27	14%	12/04/1975	8020	17	18%	12/04/1975	6590	9	15%	12/04/1975	17400	10	39%
		42	**107%**			**39**			**136**	**14%**			**51**	**19%**			**48**	**17%**			**42**	**45%**
01/27/1971	49600	6	11	01/26/1971	40800	11	12/09/1953	10930	2	24	11/07/2006	11200	4	26	11/06/2006	8130	5	26	02/07/1945	21400	5	32
01/02/1997	46000	8	12	12/30/1996	38700	12	12/11/1955	10150	4	23	12/02/1977	10300	7	17	11/25/1998	7420	6	17	01/18/1986	18100	8	27
01/13/2006	43200	10	15	01/23/1935	38000	13	01/25/1964	9760	5	22	11/26/1998	10000	9	17	01/30/2006	7080	8	15	12/16/2001	16600	11	
02/26/1999	42000	12		02/10/1951	38000	13	02/17/1949	9400	6	17	12/29/1996	9700	11		01/18/1986	6500	11		02/24/1999	16300	12	
12/19/2001	41200	13		01/31/2006	37900	15	12/28/1949	8710	7	36	01/31/2003	8940	12		12/15/1999	6350	13		01/30/2006	16000	13	
01/07/1954	40800	14		01/17/1974	37400	16	12/30/1996	8380	10	12	01/30/2006	8720	13		02/07/1945	5700	14		02/09/1951	15700	14	
01/17/1974	39100	15		02/18/1949	36500	17	12/13/1966	7270	12		01/15/1974	8440	14		12/16/2001	5620	15		12/20/1994	15300	15	
12/23/1955	38900	16		12/03/1977	36500	17	12/22/1964	7200	13		01/26/1971	8390	15		12/20/1994	5500	16		12/03/1982	15200	16	
12/15/1977	38900	16		11/26/1998	36500	17	12/02/1977	7170	14		12/16/1999	8100	16		12/03/1982	5460	17		12/15/1939	15100	17	
01/27/1964	38500	18		01/15/1936	36300	20	11/12/1958	6940	15		01/25/1964	7970	18		12/15/1939	5430	18		11/06/2006	14500	18	
12/17/1999	38100	19		12/21/1994	35900	21	11/20/1960	6680	16		02/23/1986	7960	19		12/09/1987	4960	19		12/09/1987	13800	19	
02/11/1951	36100	20		01/26/1964	35700	22	01/30/2006	6640	17		12/17/2001	7920	20		02/17/1949	4920	20		12/13/1966	13400	20	
12/15/1966	35700	21		12/22/1955	35100	23	01/26/1971	6630	18		12/09/1953	7880	21		12/13/1966	4650	21		02/22/1949	12800	21	
12/22/1994	35600	22		01/06/1954	34700	24	02/08/1955	6530	19		12/04/1982	7820	22		03/19/1997	4530	22		12/09/1956	12600	22	
01/31/1965	34000	23		12/14/1966	34400	25	11/20/1962	6520	20		01/18/2005	7740	23		01/25/1964	4330	23		03/19/1997	12600	22	
02/24/1949	33500	24		11/08/2006	32700	26	02/09/1951	6480	21		01/30/2004	7460	24		12/26/1980	4310	24		11/25/1962	12400	24	
01/26/1982	33300	25		01/20/1986	32100	27	12/11/1946	6320	23		01/14/1975	7400	25		12/30/1970	4250	25		12/15/1999	12400	24	
02/27/1950	32500	26		12/18/2001	31900	28	03/22/1948	6320	23		02/07/1979	7280	26		01/31/2003	4240	26		12/26/1980	12000	26	
01/16/1975	32100	27		12/17/1999	31000	29	11/22/1959	6290	25		12/12/1955	7200	27		11/27/1949	4040	27		12/07/1970	11700	27	
02/23/1961	32000	28		11/21/1962	29800	30	12/19/1941	6190	26		11/20/1962	6960	28		12/09/1956	3940	28		11/27/1949	11400	28	
12/28/1980	32000	28		01/25/1982	27300	31	12/17/2001	6060	28		02/17/1949	6950	29		12/23/1964	3780	29		02/04/1968	11200	29	
11/28/1962	31600	30		02/09/1945	27000	32	02/01/2003	5990	29		01/25/1984	6760	30		12/11/1955	3720	30		12/11/1955	11000	30	
11/23/1959	30100	31		02/22/1961	27000	32	01/16/1974	5950	30		04/01/1931	6750	31		02/09/1951	3690	31		02/02/1947	9980	31	
11/09/2006	29400	32		12/20/1941	26900	34	12/09/1956	5520	31		01/14/1998	6580	32		01/18/2005	3650	32		10/30/1997	9920	32	
01/28/1970	29200	33		01/15/1975	26900	34	01/24/1982	5250	32		12/23/1964	6500	33		10/30/1997	3560	33		11/17/2009	9460	33	
12/19/1979	28600	34		02/26/1950	26300	36	01/08/2007	5240	33		11/20/1960	6460	34		02/03/1963	3460	34		01/25/1964	9450	34	
12/28/1972	28100	35		12/24/1964	26200	37	03/09/1966	5160	34		12/11/1946	6350	35		02/04/1968	3450	35		02/04/1952	9320	35	

Notes: [1] Rank is the rank among the events at each individual gage, highlighted cells show events that were in the top 10 at Grand Mound but not in the top 10 at another gage

[2] % refers to percent of corresponding flow at Grand Mound seen at each of the other gages

[3] The table was truncated to show only events above a 2-year flow at Grand Mound

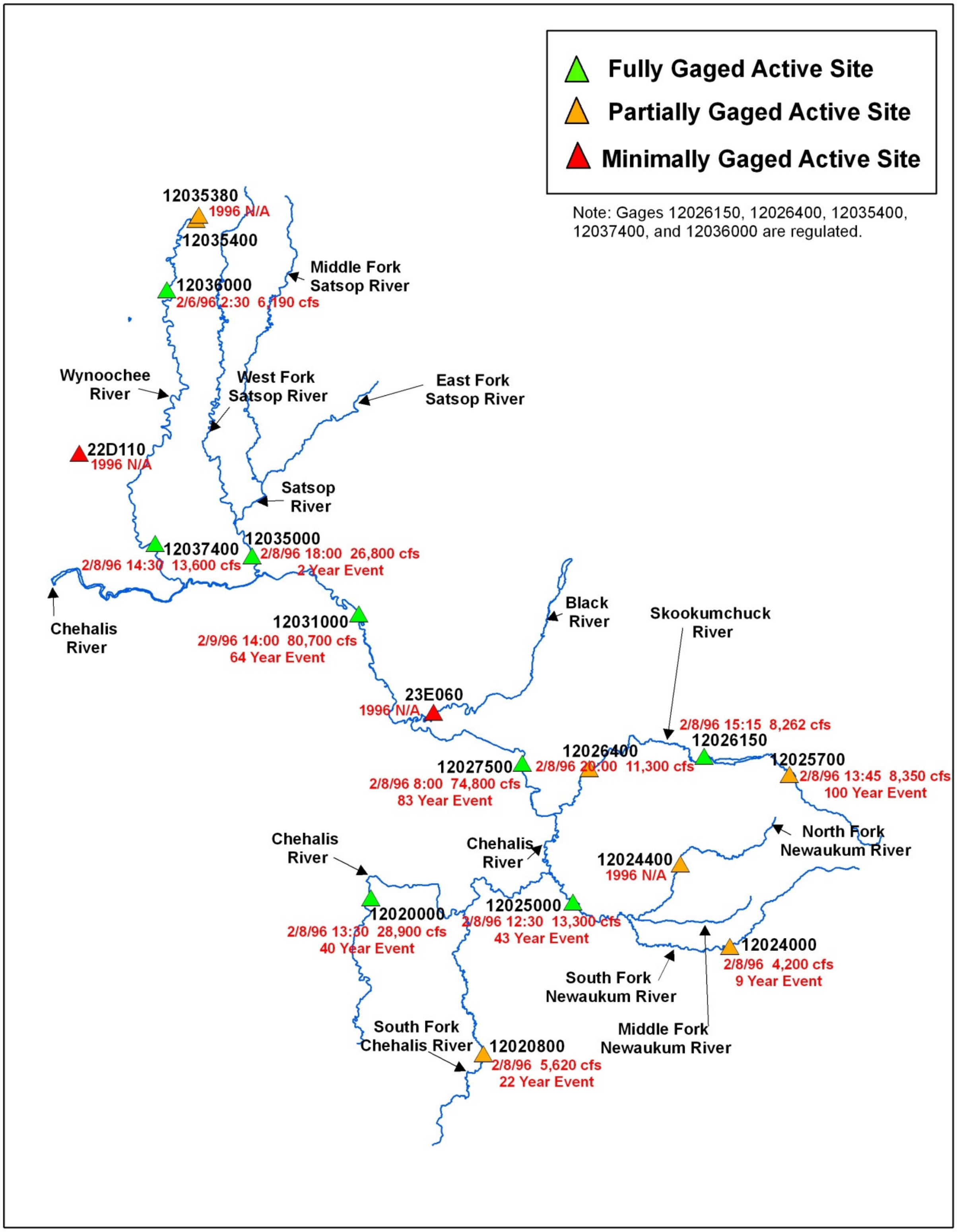

Figure 1. Flood Return Periods at Various Gaged Sites for the February 1996 Event

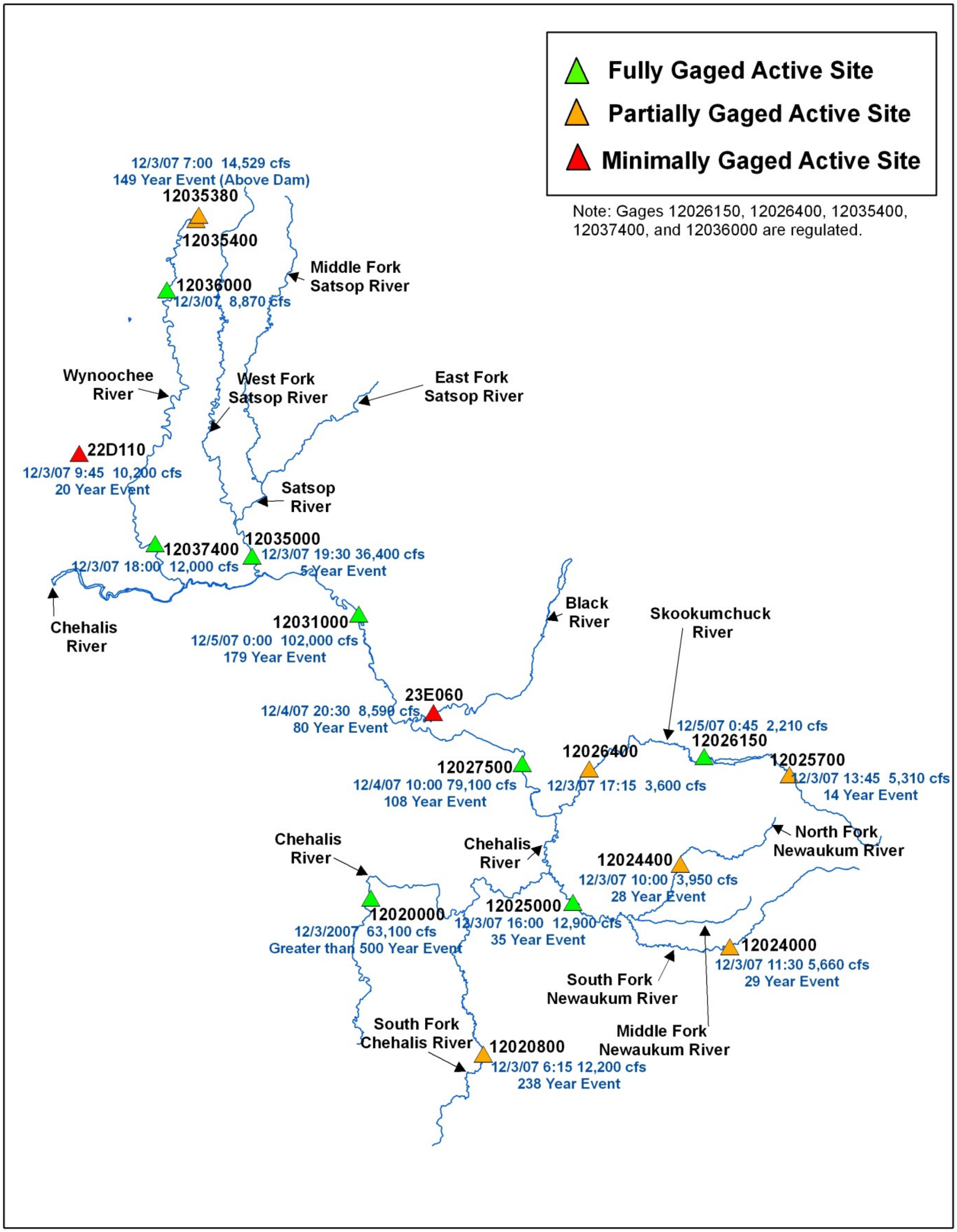

Figure 2. Flood Return Periods at Various Gaged Sites for the December 2007 Event

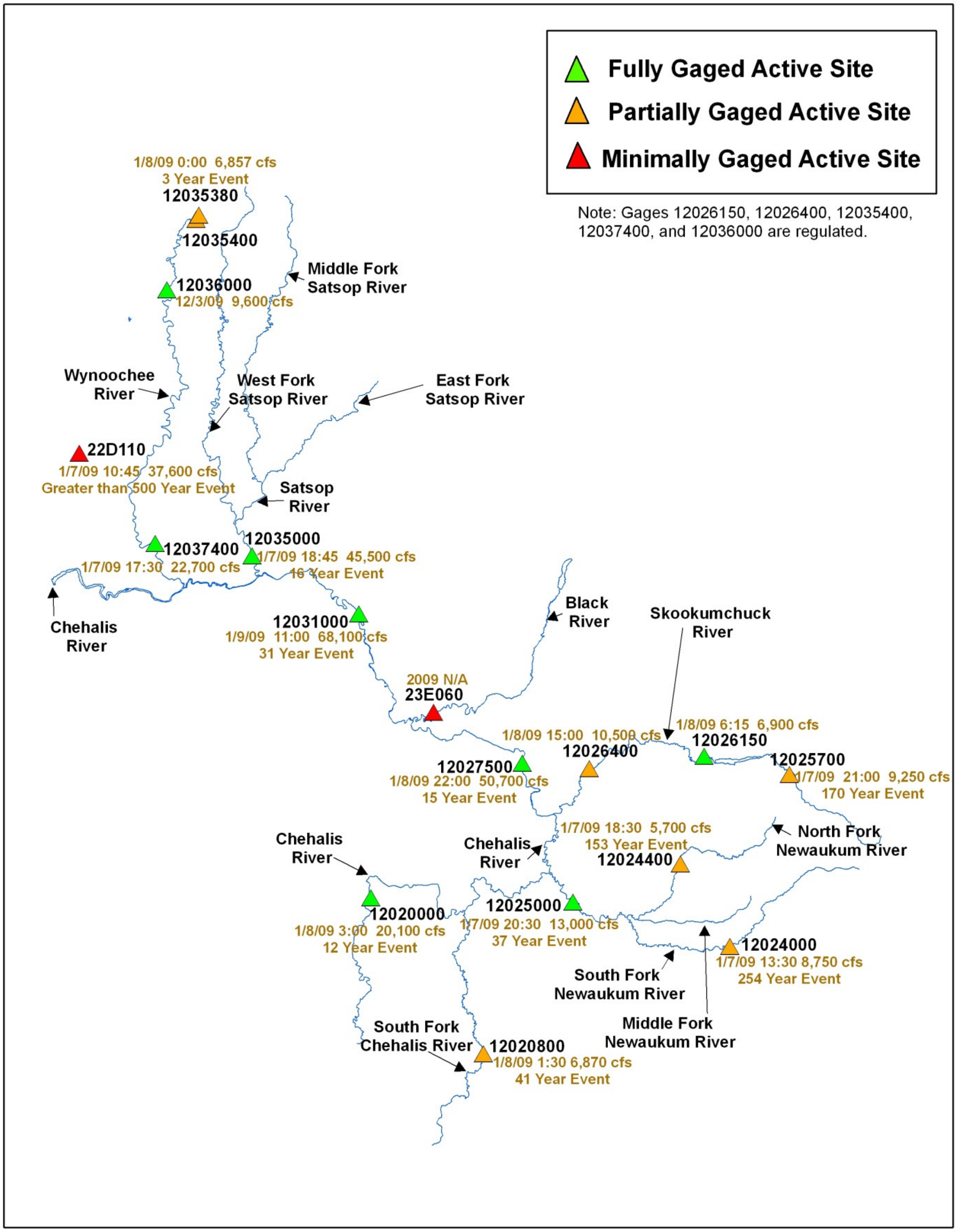

Figure 3. Flood Return Periods at Various Gaged Sites for the January 2009 Event

RE: Guy Hoyle-Dodson comment letter of 4/1/2012:
Mr. Hoyle Dodson's comments on the HEC-RAS model were particularly comprehensive including comments on general modeling approaches as well as a number of specific areas of concern or question. While many of these related to the new portions of the model being developed for this study, a large number were specifically related to the "Twin Cities" portion of the model previously developed by others. That said, and in an effort to make the model as robust and useful as possible, we have reviewed all of the comments and will attempt to address all of them as appropriate in refining the model. In addition to refinements to the model configuration we offer the following responses to key comments made by Guy:

- Regarding contraction and expansion losses, at bridges and elsewhere, note that the momentum equation which is solved under unsteady flow implicitly accounts for losses due to flow transitions. The original modeling by PIE and then by NHC, was carried out using unsteady-flow versions of UNET and HEC-RAS, that did not allow inputs of additional contraction and expansion losses. With the current HEC-RAS version 4.1, the USACE has now added a table to allow modeling of additional losses, for example at bridges with a particularly sharp contraction or expansion zone. For typical bridges, however, these losses are already accounted for in the unsteady (momentum) equation of motion. See HEC-RAS version 4.1 release notes, page 4: http://www.hec.usace.army.mil/software/hec-ras/documents/HEC-RAS_4.1_Release_Notes.pdf
- Regarding reach lengths, it should be noted that this model was developed (by PIE) generally following the 6 cross-section bridge modeling approach commonly called the Normal Bridge methodology in HEC-2 parlance. The two middle cross-sections were cut typically along the top of the roadway. The immediate upstream and downstream cross-sections were then cut close to the roadway but along natural ground (sometimes referred to as full valley sections). These are not intended to be the fully expanded or contracted sections, but are included so that floodplain storage is properly accounted for in the unsteady model. These should have appropriate ineffective areas to keep the majority of the floodplain from conveying flow, and have been checked accordingly. The fully expanded/contracted sections are generally the next downstream/upstream sections from the "full valley" sections, i.e. sections 1 and 6. These are further away from the bridge at a more acceptable distance for the flow transition.
- Regarding divided flow, it was generally assumed that this issue was dealt with appropriately in the original Twin Cities model. The current project did not include scope or budget to review or revise these in the existing FEMA model. That said, we took a quick look at the sections identified, and in some instances examined the amount of flow simulated on the floodplain to see if it would make any significant difference in the simulation results. Revisions were made to ineffective areas at some locations, as noted further below.
- On the Lower Chehalis tidal portion, the divided flow is more complex due to the tidal nature of this reach. Water does not have to exceed the channel bank elevation for flow to be in the side channels, as it comes up the channels from downstream due to the tide. Regarding the two bridges in the tidal reach, the Monte Bridge does not really have any flow contraction or expansion, in part because the upstream reach parallels the highway and does not overtop. The Hwy 101 bridge could have some ineffective areas added upstream and downstream, but it is not going to change the results any this close to the Aberdeen tidal boundary.
- Interpolated cross-sections on the Newaukum River were removed. These were added to reduce reach length and improve model stability, but HEC-RAS is unable to interpolate the blocked ineffective areas. Upon further review, the interpolations are not necessary for stability.

- Ineffective area limits (station, elevation) were revised at Newaukum cross-sections 9.84, 5.01, 2.97 and 1.03, as suggested. At other locations on the Newaukum, review of topography indicates ineffective area limits are appropriately set; i.e., divided flow would exist based upon upstream conditions.
- Regarding divided flow and ineffective limits on the main stem Chehalis in Reaches 19, 21, 23, and 24: These reaches downstream of Grand Mound tend to have significant remnant channels in some overbank areas. In addition to the general adjustments to ineffective limits discussed previously, in the areas where divided flow was noted and remnant channels are picked up in the cross section geometry, blocked, permanent ineffective areas were used where appropriate to make cut-off remnant channels ineffective.
- Regarding Right Overbank Manning's n values at cross sections 82.61 through 82.57: The overbank n values of 0.08 were a carryover from the Corps modeling. Although the aerial imagery shows what appear to be fields in the overbanks, there are also rows of trees in the right overbank at these cross sections. A Mannings n value of 0.08 does not seem to be overly conservative in this area.
- Regarding lateral structures where bounding channel cross sections have been recommended: HEC-RAS uses a linear interpolation of water surfaces between modeled cross sections to calculate flows over lateral structures. We believe the cross sections currently in the model appropriately estimate the overflows at the level of detail warranted in a regional model and that the addition of cross sections to refine the overflow estimates would not create large changes in water surface elevations in the modeled storage areas and the Chehalis River.
- Regarding Rainbow Falls Inline Weir (Reach 1): We will add a cross section closer to the upstream face of the weir to more accurately model the upstream head on the weir.
- Regarding comments related to the Skookumchuck River: Under the original Flood Authority contract, non-georeferenced areas of the Skookumchuck River model (Reach 14 of the PIE model above RS 6.44) were georeferenced by West, and 2002 LiDAR was used to update overbank geometry. The contract did not include time to investigate (or refine) modeling assumptions made during the original model development. The subsequent tributaries modeling amendment included budget for WSE to update cross section data and refine the model near the town of Bucoda (RS 9.69 to 11.8) While we agree that additional refinement to the remainder of the model would be beneficial, such refinement is generally outside the scope and budget of the current project. That said, the following summarizes the changes made to the Skookumchuck reach of the model to address Mr. Hoyle-Dodson's comments:
    - NHC Reach (River Mile 0.0 to River Mile 6.44) – this reach was refined by Northwest Hydraulic Consultants as part of the Lewis County FEMA study (2010). As such we did not feel that additional model changes, without detailed supporting investigations, were advisable.
    - Intermediate Reach (River Mile 6.44 to 9.39) – this reach, between the NHC reach and the Bucoda reach had some unusual ineffective flow and levee limits in the original PIE model (as georeferenced by WEST). In response to Mr. Hoyle-Dodson's comments and our own review of the topographic information for this reach we adjusted several ineffective and levee boundaries to better simulate expected conditions in this reach.
    - Bucoda Reach (River Mile 9.69 to 11.8) – The HEC-RAS configuration in this reach was developed and calibrated by WSE using new cross section surveys and available high

water marks. Comments on this reach were reviewed and minor changes were made to levee and ineffective flow limits.

- Upstream Reach (River mile 11.92 to 21.77) – We agree with Mr. Hoyle-Dodson that some of the ineffective limits in the PIE model of this reach appear unusual. However, the hydraulic conditions in this reach are fairly complex with shallow overbank flow in many locations. Without additional high water mark data or detailed field investigations to verify existing conditions we did not feel it was appropriate to make adjustments to the existing model at this time.

RE: Casey Kramer comment letter of 4/2/2012:

Mr. Kramer's comments were discussed between Mr. Kramer, WSE, WEST, and NHC staff in a meeting at WSE's office on March 27, 2012. As a group we agreed upon a plan of action for updating the model to address the comments. It is noted that Mr. Kramer's model comments focused on the Twin Cities portion of the model constructed by others and not actually part of the current model development effort. However, in an effort to ensure that all future analyses conducted with the model are as useful as possible the following modifications were made:

1) USGS Chehalis River Near Grand Mound, WA Gage 12027500

No model modifications were necessary to address questions with the USGS gage. WSE confirmed with the USGS that the Grand Mound gage rating curve was extrapolated from the available discharge measurements, none of which were made at a time when there was any overbank flow or flow over Prather Road. An excel plot of the available USGS discharge measurements was prepared by WSE and discussed at the meeting on March 27th. As concurred by the group, the lack of high flow discharge measurements from which to develop the high flow rating means that the upper end of the current rating curve is subject to greater uncertainty than if actual discharge measurements were available. In our opinion, discharges at higher stages (e.g. near the 100-year event) should only be considered accurate to within plus or minus 15% or so. Thus, the "observed" flow in the December 2007 flood event (79,100 cfs) could actually range between about 67,000 and 91,000 cfs.

2) Chehalis River along I-5 Upstream of Mellen Street

As discussed during the March 27th meeting, several changes were made to the model geometry near the Mellen Street Bridge. The small section of Long Road Dike immediately adjacent to I-5 was lowered and a connection was added between SA501 and SA5. Ineffective limits were added in the left overbank upstream of Mellen Street, at RS 67.86 through 67.59. Ineffective limits through the bridge itself were also modified to further constrict the upstream and downstream cross sections.

These changes had only limited effect on simulated water surface elevations upstream of Mellen Street Bridge. When constrictions were added to the Chehalis River, in the form of ineffective limits (changes to Manning's n and contraction/expansion coefficients were also briefly tested), water surface elevations in the vicinity of Mellen Street increased only about one tenth of a foot. However, more flow did overtop the lateral structures in the right overbank, which resulted in less flow in the Chehalis River.

WSDOT also provided new topographic survey data for I-5 and the airport levee. The lateral structure elevations in the model were revised to reflect the new survey data. The revision to the lateral structures resulted in minor changes to the simulated water levels in the main stem of the Chehalis River.

Considering the results of the model investigations in this area it appears that we would either need to make atypical changes to the modeling of the Mellen Street Bridge (such as arbitrary additional head losses) or increase the flows reaching the bridge in order to "hit" the higher of the high water marks upstream of the bridge. Increasing the flows would lead to problems with matching high water marks at other locations in the model so we do not feel that is a reasonable alternative. Similarly, we don't feel it is wise to insert arbitrary losses into the model simply to meet a few high water marks (bearing in mind that there are other, lower high water marks in the same area that we are already overshooting). Thus, we feel that the modeling in this area has been improved as much as possible and do not propose to make any additional changes.

3) Dillenbaugh Creek and Chehalis River Connections near Main Street and I-5

To better approximate December 2007 flood conditions near the Dillenbaugh Creek/Chehalis Junction, two lateral weirs (0.120 and 0.092) were added along Dillenbaugh to model flow entering the north- and southbound lanes of I-5 and flowing under the Highway 6 overpass. Weir elevations were based on 2012 survey completed by WSDOT. Additionally, the weir coefficient (C_d) for Main Street was reduced from 2.0 to 1.5 to approximate losses as water exiting Dillenbaugh flows through vegetation and around buildings on its path to Storage Area #303.

With these changes the model showed peak flow values of:

- 1870 cfs flowing over the Main Street weir (LS 0.187) between Dillenbaugh Creek and Storage Area #303
- 1710 cfs overtopping of the I-5 weir returning to the Chehalis River (LS 74.41, Chehalis Reach 9) and 30 cfs flowing through the culvert under I-5
- 165 cfs flowing from SA #303 to Dillenbaugh Creek via the northbound lanes of I-5
- 145 cfs flowing from Dillenbaugh Creek to the Chehalis River via the southbound lanes of I-5

A section of the I-5 weir (LS 74.41) was then lowered (as discussed during the March 27th meeting) to simulate the portion of I-5 that does not have a jersey barrier along its east side, and the failure of the centerline jersey barrier that occurred during the Dec 2007 flood event. This resulted in peak flow values of:

- 2378 cfs flowing over the Main Street weir
- 2552 cfs flowing over the I-5 weir or through the culvert back into the Chehalis River
- 176 cfs flowing from Dillenbaugh to SA #303 via the northbound lanes of I-5
- 87 cfs flowing from Dillenbaugh to the Chehalis via the southbound lanes of I-5

The maximum simulated depth of flow over I-5 in between SR-6 and NW West Street was about 2.0 ft, which may be somewhat high based on photographs we have seen from the 2007 flood. Additional model refinement might reduce the peak stages over the freeway in this area but it is not clear that there is enough information to definitively state how high the flow may have gotten and/or the direction and magnitude of breakout flows from Dillenbaugh Creek during the event. As such, no additional refinement to the model calibration was attempted.