American River Watershed Project

Post-Authorization Change Report
And Interim General Reevaluation Report

American River Watershed

Common Features Project
Natomas Basin
Sacramento and Sutter Counties, California

SAFCA
Sacramento Area Flood Control Agency

Central Valley Flood Protection Board
Public Safety and Flood Management
State of California

US Army Corps of Engineers ®
Sacramento District

Appendix B – Hydrology Technical Documentation

August 2010

Appendix B

Appendix B1 – Synthetic Hydrology Technical Documentation

Appendix B2 – American River Hydrology and Folsom Dam Reservoir Operations

Appendix B3 – Dry and Arcade Creeks Flow Frequency Curves and Synthetic 8- Flood Series Hydrographs Upstream of Steelhead Creek

US Army Corps of Engineers
Sacramento District

American River Watershed
Common Features Project
Natomas Post-Authorization Change Report

Appendix B1
Synthetic Hydrology Technical Documentation

September 2008

AMERICAN RIVER WATERSHED COMMON FEATURES PROJECT
NATOMAS POST-AUTHORIZATION CHANGE REPORT
SYNTHETIC HYDROLOGY TECHNICAL DOCUMENTATION

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AMERICAN RIVER WATERSHED COMMON FEATURES PROJECT
NATOMAS POST-AUTHORIZATION CHANGE REPORT
SYNTHETIC HYDROLOGY TECHNICAL DOCUMENTATION

1.0 Documentation for Synthetic Flood Centerings

This chapter cites the documentation used to develop the hydrographs provided to Hydraulic Design Section as input for its calibrated HEC-RAS 4.0 model – the model used to develop water surface profiles for existing conditions (year 2007). Multiple flood centerings were tested to assure that the controlling hydrologic events were used for the hydraulic analysis. Each centering consisted of flow hydrographs developed for the specific frequency events: 50-, 10-, 4-, 2-, 1-, 0.5-, and 0.2 percent exceedence floods (8-Flood Series). The three flood centerings tested were the Sacramento Mainstem, Shanghai Bend-Yuba River, and the American River. The study area includes the Sacramento River from the Natomas Cross Canal down to Freeport and the American River from Folsom Dam down to its confluence with the Sacramento River, as well as the Natomas tributary drainage to the Natomas Cross Canal and to Steelhead Creek. **Plate 1**, the general map, shows the watersheds for the four Natomas tributaries to Steelhead Creek, the five Natomas tributaries to the Natomas Cross Canal, the American River south of the Natomas tributaries, the Feather River at its confluence with the Sacramento River, and the Sacramento River from upstream of Feather River down to its confluence with the American River. **Plate 2** shows where the hydraulic model input locations are for the five hydrographs contributing to the Natomas Cross Canal and the four hydrographs contributing to Steelhead Creek. Steelhead Creek is also known as the Natomas East Main Drainage Canal (NEMDC). The hydrographs are for an unsteady state simulation.

The three different flood centerings mentioned above are being tested in the hydraulic model to see which one produces the highest stages in which locations of the study area. Under certain conditions the American River is the controlling flood event for Steelhead Creek. The Shanghai Bend centering or the Sacramento Mainstem centering may be the controlling flood event for the Natomas Cross Canal. However, which flood centering series will produce the most critical flooding at which locations will not be known without hydraulic analysis.

1.1 <u>Sacramento Mainstem Centering</u>. The flood centering hydrographs were created using the methodology developed in the Comprehensive Study (the "Sacramento and San Joaquin River Basins Comprehensive Study," Technical Studies Documentation, dated December 2002, abbreviated here as Comp Study and described in **Reference 1**). The Comprehensive Study models were developed for use in regional, broad concept studies, such as the Sacramento Common Features General Reevaluation study. **Reference 1, Appendix B**: "Synthetic Hydrology Technical Documentation," describes the development of the unregulated flood hydrographs.

Unregulated flow frequency curves were developed at key mainstem and tributary locations in the Sacramento River basin. The unregulated frequency curves plot historic flood peaks and volumes with the statistical distributions of unimpaired flows (with no reservoir influence). The frequency curves display volumes, or average flow rates, for different time durations over a range of annual exceedence probabilities. These curves are used to translate: 1) hydrographs to frequencies; and 2) frequencies to flood volumes. As part of the Comprehensive Study (Comp Study), flow frequency curves were developed for 1-, 3-, 7-, 15-, and 30-day durations. A routing model was developed to route the unregulated daily flows from the tributary locations to downstream locations for use in constructing mainstem "index" frequency curves. Mainstem locations include the Sacramento River at the Latitude of Sacramento (including flows down the Yolo Bypass) and the Feather River downstream of the Yuba River (at Shanghai Bend). The maximum flows for each winter at the mainstem locations were used to develop flow frequency curves (for 1-, 3-, 7-, 15-, and 30-day durations) for those mainstem locations. No synthetic precipitation events were needed for the hydrology. This paragraph and the paragraphs below explain the development of the synthetic flood centerings for the latitude of Sacramento; the flood centerings for Shanghai Bend were developed similarly.

Based on analysis of historic floods over the Sacramento watershed, synthetic mainstem flood centerings were developed to stress widespread valley areas. The flow frequency curves for the Latitude of Sacramento (used for the Sacramento Mainstem Centering) provide the hypothetic flood volumes that the basin will produce during simulations of each of the eight synthetic exceedence frequency flood events (50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2percent). The role of the mainstem centering is to distribute these flood volumes back into the basin, tributary by tributary, in accordance with patterns visible in historic flood events. **Reference 1, Appendix C**: "Reservoir Operations Modeling, Existing Design Operations and Reoperation Analysis," describes the development of the reservoir operations models to route the unregulated hydrographs through the headwater and major flood management reservoirs for input into the hydraulic model.

The Sacramento Mainstem flood hydrographs were developed using the flood patterns shown on **Table 1** to produce flood runoff hydrographs centered at the Latitude of Sacramento. **Table 1** shows the set of synthetic exceedence frequencies assigned to the set of tributaries listed in column 1 such that the regulated and routed hydrographs have the volumes for a flood series centered at the Latitude of Sacramento. The hydrographs have a duration of 30 days, with six 5-day waves. The pattern hydrograph used for the 5-day waves at each upstream tributary is that of the unregulated flood hydrograph for 30 December 1996 to 3 January 1997 (New Year 1997 flood) at that tributary index point. This flood pattern was used because, of the large historical floods over the Sacramento Basin, it is the flood event for which hourly hydrographs were available for the largest number of upstream tributary gages used for the Comp Study. The American River flood hydrographs are different from those used in the Comp Study. See **Section 1.3** for an explanation of the changes made for the American River centering.

Table 1
Sacramento River Mainstem Synthetic Flood Centering

Index Point	Percent Chance Exceedence						
	50%	10%	4%	2%	1%	0.50%	0.20%
Sacramento River at Shasta	84.42	17.03	8.09	4.41	2.21	1.13	0.44
Clear Cr. at Whiskeytown	80.91	17.03	10.79	6.47	3.24	1.66	0.65
Cow Cr. near Millville	80.91	16.18	9.71	5.39	2.70	1.38	0.60
Cottonwood Cr. near Cottonwood	80.91	17.03	10.79	6.47	3.24	1.66	0.65
Battle Cr. Below Coleman FH	80.91	16.18	9.71	5.39	2.70	1.38	0.60
Mill Cr. near Los Molinos	80.91	16.18	9.71	4.22	2.35	1.23	0.51
Elder Cr. near Paskenta	88.26	19.42	10.79	4.85	2.70	1.38	0.58
Thomes Cr. at Paskenta	88.26	19.42	10.79	4.85	2.70	1.38	0.58
Deer Cr. near Vina	88.26	16.18	9.71	4.22	2.35	1.23	0.51
Big Chico Cr. near Chico	88.26	16.18	9.71	4.22	2.35	1.23	0.51
Stony Cr. at Black Butte	88.26	19.42	10.79	4.85	2.70	1.38	0.58
Butte Cr. near Chico	66.70	13.63	6.08	2.75	1.38	0.71	0.30
Feather River at Oroville	53.60	11.78	4.42	2.41	1.20	0.62	0.24
Yuba R. at New Bullards Bar	55.09	12.52	4.86	2.10	1.05	0.54	0.21
Yuba R. at Englebright	55.09	12.52	4.86	2.10	1.05	0.54	0.21
Deer Cr. near Smartsville	55.12	12.52	4.86	2.10	1.05	0.54	0.21
Bear River near Wheatland	53.60	11.13	4.42	2.10	1.05	0.54	0.21
Cache Cr. at Clear Lake	52.19	12.52	6.95	4.45	2.22	1.14	0.45
N.F. Cache Cr. at Indian Vy.	52.19	12.52	6.95	4.45	2.22	1.14	0.45
American River at Folsom	55.09	12.52	4.86	2.51	1.26	0.64	0.25
Putah Cr. at Berryessa	52.19	12.52	6.95	4.45	2.22	1.14	0.45

The process of preparing flood hydrographs begins by using unregulated frequency curves to translate all of the exceedence frequencies in the synthetic patterns to average flow rates. The unregulated frequency curves were prepared using 1-, 3-, 7-, 15-, and 30-day durations. Values for the 5-, 10-, 20-, and 25-day durations were obtained through interpolation. The values from the frequency curves represent the average flow anticipated over a specific time interval. For instance, the 5-day value is the average flow expected during the highest 5-days of flooding during any of the eight synthetic exceedence events. Likewise the 10-day value is the average over the highest 10 days of flooding. Flood volumes were computed by multiplying the average flows by their respective durations. These values represented the total volumes of water anticipated during the highest 5, 10, 15, 20, 25, or 30 days of flows. Furthermore, these flood volumes were portioned into time segments by subtracting volumes of the shorter durations from the next longer duration. For example, the 5-day volume was subtracted from the 10-day volume and the remainder was equal to the amount of flood volume that is produced by the tributary between the 5-day and 10-day maximum periods. This procedure was repeated for the 10-, 15-, 20-, 25-, and 30-day durations and resulted in a set of eight synthetic exceedence frequency flood volumes produced by the tributary.

The basic pattern of all synthetic flood hydrographs was a 30-day hourly time series consisting of 6 waves, each 5 days in duration. Volumes were ranked and distributed into the basic pattern. The highest wave volume was always distributed into the fourth, or main, wave.

The second and third highest volumes preceded and followed the main wave, respectively. The fourth highest volume was distributed into the second wave and the fifth highest was distributed into the final of the six waves. The sixth and smallest wave volume was distributed into the first wave of the series. The shape of each wave is identical and the magnitude is determined by the total volume that the wave must convey. The process of converting flow frequency curves into the synthetic series of 30-day hydrographs is depicted on **Plate 3**.

There are several reasons for using a 30-day duration for the synthetic flood hydrographs. The Sacramento River watershed is so large that 5 days is not long enough for a flood wave to travel from the most distant headwater down to the mouth of the Sacramento River. The multi-wave flood hydrograph includes the smaller antecedent waves from storms that prime the watershed for the highest wave. Also, the multi-wave hydrograph is needed to (1) provide the extra flood volume needed to simulate reservoir operation during an extended period of wet weather, and (2) fill the floodplains with enough flood volume to run levee failure scenarios.

Figure 1 shows an example of the 30-day hydrograph with the 5-day waves, for unregulated and regulated conditions. The figure shows the 1 percent exceedence hydrographs, for unregulated and regulated conditions, for the Sacramento River at the confluence with the Feather River, for the Sacramento Mainstem Centering. The hydrograph for unregulated conditions is not a true representation of the hydrograph with six 5-day waves; it is the result from routed contributions of upstream tributaries. See **Figure 2** for an example of a tributary hydrograph with six 5-day waves – the Comp Study hydrograph for Folsom Lake inflow.

Figure 1

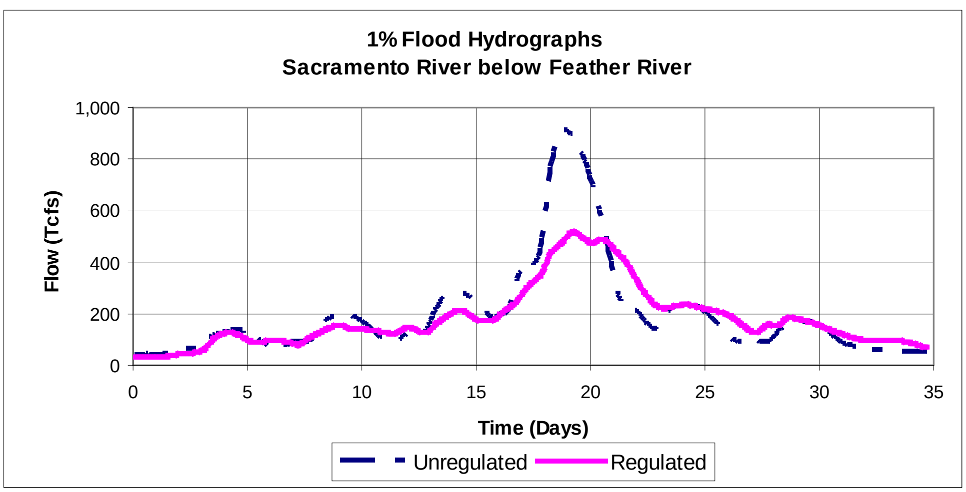
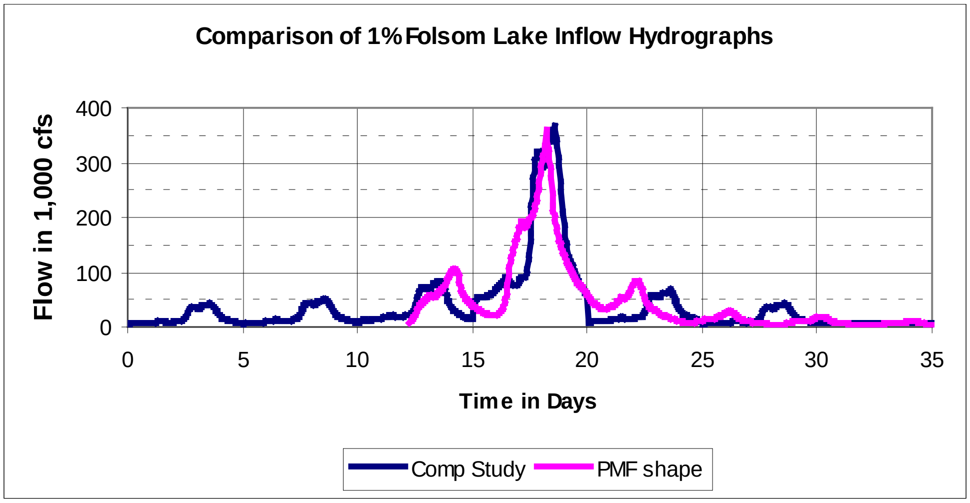


Figure 2



1.2 <u>Shanghai Bend-Yuba River Centering</u>. This flood centering, with a specific centering on the Yuba River and slightly more frequent concurrent event on the Feather River above Oroville, produces the maximum inundation areas along the lower reaches of the Feather and Yuba rivers. It also produces the maximum inundation area at Verona, near the confluence of the Feather River with the Sacramento River. This flood centering was not developed as part of the original Comp Study, but the Comp Study methodology described in **Reference 1** was used to develop the storm centering and flood hydrographs, which were routed through the reservoir system. **Reference 2**, the "Yuba River Basin Project General Reevaluation Report," App. A, Synthetic Hydrology and Reservoir Operations Technical Documentation, dated August 2004, corrected June 2008, documents the hydrology and modeling efforts conducted for the Feather and Yuba rivers using the Comp Study methodology. **Table 2** shows the flood patterns for the Shanghai Bend-Yuba River centering. The American River flood hydrographs are different from those used in the Comp Study. See **Section 1.3** for an explanation of the changes made.

Table 2
Feather River above Shanghai Bend Synthetic Flood Centering A
With a Specific Centering on the Yuba River

Index Point	Percent Chance Exceedence						
	50%	10%	4%	2%	1%	0.50%	0.20%
Sacramento River at Shasta	101.01	20.20	8.08	5.77	2.89	1.44	0.58
Clear Cr. at Whiskeytown	344.83	68.97	27.59	19.70	9.85	4.93	1.97
Cow Cr. near Millville	196.08	39.22	15.69	11.20	5.60	2.80	1.12
Cottonwood Cr. near Cottonwood	344.83	68.97	27.59	19.70	9.85	4.93	1.97
Battle Cr. Below Coleman FH	196.08	39.22	15.69	11.20	5.60	2.80	1.12
Mill Cr. near Los Molinos	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Elder Cr. near Paskenta	140.85	28.17	11.27	8.05	4.02	2.01	0.80
Thomes Cr. at Paskenta	140.85	28.17	11.27	8.05	4.02	2.01	0.80
Deer Cr. near Vina	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Big Chico Cr. near Chico	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Stony Cr. at Black Butte	140.85	28.17	11.27	8.05	4.02	2.01	0.80
Butte Cr. near Chico	76.34	15.27	6.11	4.36	3.18	1.09	0.44
Feather River at Oroville	54.95	10.87	4.35	2.17	1.06	0.53	0.21
Yuba R. at New Bullards Bar	50.00	10.00	4.00	2.00	1.00	0.5	0.20
Yuba R. at Englebright	50.00	10.00	4.00	2.00	1.00	0.5	0.20
Deer Cr. near Smartsville	125.00	25.00	10.00	5.00	2.50	1.25	0.50
Bear River near Wheatland	125.00	25.00	10.00	5.00	2.50	1.25	0.50
Cache Cr. at Clear Lake	153.85	30.77	12.31	6.15	3.08	1.54	0.62
N.F. Cache Cr. at Indian Vy.	153.85	30.77	12.31	6.15	3.08	1.54	0.62
American River at Folsom	76.34	15.27	6.11	3.05	1.53	0.76	0.31
Putah Cr. at Berryessa	153.85	30.77	12.31	6.15	3.08	1.54	0.62

1.3 American River Centering. The flood patterns for the American River specific tributary centering are shown on **Table 3**. The concurrent flood hydrographs for this centering were developed using the Comp Study methodology and hydrograph shapes, based on the January 1997 New Years flood event. However, the American River specific flood hydrographs were developed using a different shape and different volumes. For consistency with the ongoing American River Watershed Study, the Folsom Dam inflow hydrograph shape used for the American River Common Features GRR is based upon the Probable Maximum Flood (PMF) for Folsom Dam. Use of this PMF-shape flood hydrograph predates the Comp Study. Development of the revised Folsom Dam PMF is discussed in **Reference 3**, "Folsom Dam and Lake Revised PMF Study," American River Basin, California, Hydrology Office Report, dated October 2001. The PMF was computed using the most recent Probable Maximum Precipitation criteria, presented in **Reference 4**, "Hydrometeorological Report No. 59, Probable Maximum Precipitation for California," U.S. Dept. of Commerce, NOAA, U.S. Dept of the Army Corps of Engineers, Feb 1999).

Table 3
American River Tributary Synthetic Flood Centering

Index Point	Percent Chance Exceedence						
	50%	10%	4%	2%	1%	0.50%	0.20%
Sacramento River at Shasta	250.00	50.00	20.00	10.00	5.00	2.50	1.00
Clear Cr. at Whiskeytown	555.56	111.11	44.44	22.22	11.11	5.56	2.22
Cow Cr. near Millville	178.57	35.71	14.29	7.14	3.57	1.79	0.71
Cottonwood Cr. near Cottonwood	555.56	111.11	44.44	22.22	11.11	5.56	2.22
Battle Cr. below Coleman FH	178.57	35.71	14.29	7.14	3.57	1.79	0.71
Mill Cr. near Los Molinos	121.95	24.39	9.76	4.88	2.44	1.22	0.49
Elder Cr. near Paskenta	138.89	27.78	11.11	5.56	2.78	1.39	0.56
Thomes Cr. at Paskenta	138.89	27.78	11.11	5.56	2.78	1.39	0.56
Deer Cr. near Vina	121.95	24.39	9.76	4.88	2.44	1.22	0.49
Big Chico Cr. near Chico	138.89	27.78	11.11	5.56	2.78	1.39	0.56
Stony Cr. at Black Butte	121.95	24.39	9.76	4.88	2.44	1.22	0.49
Butte Cr. near Chico	138.89	27.78	11.11	5.56	2.78	1.39	0.56
Feather River at Oroville	92.59	18.52	7.41	3.7	1.85	0.93	0.37
Yuba R. at New Bullards Bar	69.44	13.89	5.56	2.78	1.39	0.69	0.28
Yuba R. at Englebright	69.44	13.89	5.56	2.78	1.39	0.69	0.28
Deer Cr. near Smartsville	116.28	23.26	9.30	4.65	2.33	1.16	0.47
Bear River near Wheatland	116.28	23.26	9.30	4.65	2.33	1.16	0.47
Cache Cr. at Clear Lake	192.31	38.46	15.38	7.69	3.85	1.92	0.77
N.F. Cache Cr. at Indian Vy.	192.31	38.46	15.38	7.69	3.85	1.92	0.77
American River at Folsom	50.00	10.00	4.00	2.00	1.00	0.50	0.20
Putah Cr. at Berryessa	192.31	38.46	15.38	7.69	3.85	1.92	0.77

Also, the American River Watershed Study unregulated flow frequency curves for the American River were revised when the period of record was updated through 2004. See **Reference 5**, “Rain Flood Flow Frequency Analysis, American River California,” Office Report, U.S. Army Corps of Engineers, Sacramento District, dated August 2004. Revision of the flood frequency curves changed the flood volumes used for the American River hydrographs for the 8-Flood Series. **Figure 2** is a graphical presentation of the flood inflow hydrographs to Folsom Lake, comparing the Comp Study 1 percent flood with the PMF-shape 1 percent flood. The graph presents the maximum 72-hour period as coincident for the two flood hydrographs for days 17 through 19.

Because the PMF-shape hydrographs for the Folsom Lake inflow are different from the Comp Study hydrographs, a volume comparison was made between the hydrographs for various exceedence events. This comparison was made to ensure that use of the PMF-shape hydrographs would not cause problems and inconsistencies. **Table 4** presents a volume comparison between the two different hydrograph shapes for the American River flood series above Folsom Dam. The table shows that the differences in volume are minor.

Table 4
Hydrograph Volume Comparison for
Inflow Hydrographs to Folsom Lake

% Event Flood	1-Day Volume (in day cfs)	3-Day Volume (in day cfs)	7-Day Volume (in day cfs)
10% (PMF Shape)	101,000	71,000	43,000
10% (Comprehensive Study)	113,000	70,000	46,000
% Difference	12%	-1%	7%
4% (PMF Shape)	156,000	110,000	66,000
4% (Comprehensive Study)	174,000	108,000	67,000
% Difference	10%	-2%	1%
2% (PMF Shape)	207,000	145,000	87,000
2% (Comprehensive Study)	229,000	142,000	86,000
% Difference	10%	-2%	-1%
1% (PMF Shape)	266,000	187,000	112,000
1% (Comprehensive Study)	292,000	181,000	107,000
% Difference	9%	-3%	-5%
0.5% (PMF Shape)	334,000	235,000	141,000
0.5% (Comprehensive Study)	363,000	226,000	131,000
% Difference	8%	-4%	-8%
0.2% (PMF Shape)	440,000	309,000	185,000
0.2% (Comprehensive Study)	475,000	300,000	169,000
% Difference	7%	-3%	-9%

The flow comparison is presented in Table 4 in "% Difference", which shows how much the Comprehensive Study hydrograph volume differs from the PMF shape hydrograph volume. Hydrographs are for unregulated inflow conditions.

The PMF-shape hydrographs were routed through Folsom Dam for three without-project alternatives. In preparation for routing the PMF-shape hydrographs through Folsom Dam, the maximum 72-hour period of the PMF-shape was lined up to occur at the same time as the Comp Study American River hydrograph. See **Figure 2** above. For the PMF-shape hydrographs, the maximum 3-day flow occurs closer to the beginning of the hydrograph. As a result, outflow from Folsom Dam for the PMF-shape hydrographs does not begin until 6 p.m. of day 12 after the start of the Comp Study hydrographs for the other Sacramento River tributaries. A constant flow of 2,000 cfs was used for outflow from Folsom Dam for days 1 through 6pm of day 12 for the PMF shape flood hydrographs.

2.0 Development of Historical Flood Hydrographs for Natomas Tributaries

Historical flow hydrographs for the Natomas tributaries were developed as upstream boundary conditions on the Natomas Cross Canal and Steelhead Creek (also known as Natomas East Main Drainage Canal), for testing of the hydraulic model. The upstream boundary locations for the Natomas tributaries are shown on **Plate 2.** Six large historical flood events were chosen for which Natomas tributary flood hydrographs would be developed. The six flood events are 15 - 19 February 1986, 8 - 12 January 1995, 29 December 1996 - 3 January 1997, 22 - 26 January 1997, 2 - 6 February 1998, and 30 December 2005 - 3 January 2006. The selection of flood events was based on the amount of available precipitation data and whether any flow data, either a hydrograph or mean day flow, were available for the Dry Creek at Roseville gaging station. Hydrographs for the six floods on the Sacramento, Feather, and American rivers were available for use in the hydraulic model. The effect of any additional contribution from the Natomas tributaries could then be tested in the model. Also, from the frequency analysis presented in the Natomas General Reevaluation Report Hydrology Appendix (**Reference 6**), frequencies could be assigned to these flood events for the Natomas tributaries, which could then be compared with the magnitudes of these events on the mainstem Sacramento and American rivers for the Coincident Frequency Analysis.

This chapter discusses the computation of historical flood hydrographs first for the Steelhead Creek tributaries and then for the Natomas Cross-Canal tributaries. The historical flood hydrographs were easier to develop for Steelhead Creek because calibrated HEC-1 models had been developed in previous studies for the tributaries, an extensive network of precipitation gages covers the watershed, and hydrographs or mean day flows exist for the six flood events for the Dry Creek at Roseville gage. A mean day flow record is available for four of the six floods at the Arcade Creek near Del Paso Heights gage. **Table 5** shows what flow data are available for which storm events. Station locations are shown on **Plate 1**.

Table 5
Available Flow Data for 6 Historical Flood Events

Stream---> Gage Location--->	Dry Cr Royer Park	Dry Cr Vernon St.	Magpie Cr Del Paso Hghts	Arcade Cr Del Paso Hghts
CDEC Code or USGS Number	CDEC RYP	CDEC VRS	USGS 11447330	CDEC ACK
FLOOD EVENT	D.A. (sq.mi.) 58.63*	D.A. (sq.mi.) 77.75*	D.A. (sq.mi.) 2.30*	D.A. (sq.mi.) 31.83*
15-19 February 1986	N/A	Hydrograph	N/A	N/A
8-12 January 1995	N/A	Hydrograph	N/A	N/A
29 Dec 96 - 3 Jan 97	N/A	Mean Day	Mean Day	Mean Day
22-26 January 1997	N/A	Mean Day	Mean Day	Mean Day
2-6 February 1998	N/A	Mean Day	N/A	Mean Day
30 Dec 05 - 3 Jan 06	hydrograph	Hydrograph	N/A	Mean Day

N/A = Not Available

* = drainage area in HEC-1 model, not drainage area associated with DWR or USGS gage

Some of the precipitation gages used for the December 2005 storm isohyetal map were not available for the earlier flood events. These are mostly the stations on the Wunderground Web site and are not included in **Table 6**. **Table 6** below lists the National Climatic Data Center (NCDC) stations and California Data Exchange Center (CDEC) stations used to develop the storm isohyetal maps for one or more of the six historical flood events. **Table 6** also lists the station precipitation amounts for the 6 storms. **Plate 4** shows the locations of the precipitation gages listed in **Table 6** and the streamflow gages listed in **Table 5**.

Table 6

Precipitation Gages - Storm Totals for 6 Historical Storm Events

STATION	DATA SOURCE	CDEC STATION CODE	STORM EVENT AND PRECIPITATION (INCHES)					
			1986 15-19 FEB	1995 8-12 JAN	1996 - 97 29 DEC - 2 JAN	1997 22-26 JAN	1998 2-6 FEB	2005 - 06 30 DEC - 3 JAN
Arcade Cr-Winding Way	CDEC	AMC	N/A	N/A	** 3.93	** 6.34	** 5.79	** 4.93
Arden	CDEC	ARW	** 9.09	5.74	** 3.34	** 5.59	** 5.00	4.49
Auburn	NCDC	---	12.83	8.96	7.28	7.95	5.70	N/A
Auburn Dam Ridge	CDEC	ADR	N/A	N/A	** 6.93	** 7.84	** 5.55	4.60
CSUS	CDEC	CSU	N/A	N/A	N/A	N/A	N/A	4.80
Camp Far West	CDEC	CFW	N/A	N/A	N/A	N/A	N/A	4.63
Caperton Reservoir	CDEC	CPR	N/A	N/A	** 4.65	** 5.67	** 5.63	** 4.64
Chicago	CDEC	CHG	** 7.96	N/A	3.82	5.75	2.68	4.69
Cresta Park	CDEC	CRP	9.37	N/A	3.86	6.50	4.88	4.49
Englebright Dam	CDEC	ENG	N/A	5.48	6.20	6.56	4.83	N/A
Folsom Dam	CDEC	FLD	9.53	N/A	2.13	3.58	3.03	4.72
Folsom WTP	CDEC	FWP	N/A	N/A	N/A	N/A	5.94	N/A
Grass Valley #2	NCDC	---	** 14.9	9.51	14.73	10.77	8.69	N/A
Grass Valley	CDEC	GVY	N/A	N/A	N/A	N/A	N/A	10.72
Hurley	CDEC	HUR	N/A	N/A	2.78	3.56	3.91	4.55
Lincoln	CDEC	LCN	N/A	** 5.19	N/A	3.46	** 5.15	4.34
Loomis Observatory	CDEC	LMO	N/A	N/A	3.74	6.38	4.89	3.89
Navion	CDEC	NVN	** 9.54	N/A	N/A	6.07	5.94	N/A
Newcastle-Pineview Sch.	CDEC	NCS	N/A	N/A	** 4.96	** 6.74	** 5.94	4.93
Orangevale	CDEC	ORN	** 6.67	N/A	3.94	5.67	6.26	4.85
Rancho Cordova	CDEC	RNC	7.76	N/A	3.54	5.50	5.24	4.61
Represa	NCDC	---	7.03	5.24	3.52	4.47	4.53	3.89
Rio Linda	CDEC	RLN	** 7.28	N/A	** 2.92	** 4.77	** 5.32	** 3.90
Roseville City Hall	#	---	9.34	N/A	N/A	N/A	N/A	N/A
Roseville Fire Stn	CDEC	RSV	N/A	N/A	3.62	** 5.63	N/A	3.76
Roseville WTP	CDEC	RTP	** 8.76	N/A	** 4.30	** 6.30	** 5.95	** 5.01
Royer Park	CDEC	RYP	N/A	N/A	** 3.86	** 6.50	** 6.10	** 4.08
Sac Exec AP	NCDC	---	6.72	5.11	2.79	5.65	4.69	4.70
Sac Metro AP	CDEC	SMF	N/A	4.30	5.51	5.74	3.70	3.56
Sacramento 5 ESE	NOAA	---	7.68	5.89	2.22	4.71	4.54	5.02
Sacramento City	#	---	8.12	N/A	N/A	N/A	N/A	N/A
Sacramento Post Office	CDEC	SPO	N/A	5.89	2.46	4.75	4.60	N/A
Sierra College	#	---	9.05	N/A	N/A	N/A	N/A	N/A
Sunrise Blvd	#	---	6.82	N/A	N/A	N/A	N/A	N/A
Van Maren	CDEC	VNM	** 8.90	N/A	** 3.98	** 5.95	** 5.98	N/A
Wheatland 2NE	NCDC	---	4.90	4.40	N/A	N/A	N/A	N/A

N/A = Not Available or Missing Record

** = Recording Rain Gage pattern used to distribute this storm in HEC-1 Model

= Data from Dry Creek Basin Hydrology Report dated April 1988

2.1 Steelhead Creek Historical Flood Hydrographs.

a. December 2005 Flood. The December 2005 – January 2006 rainflood event was used to validate the HEC-1 models for Dry and Arcade creeks in **Reference 6**, the Natomas GRR Hydrology Appendix, dated October 2006. **Plate 5** shows the December 2005 – January 2006 storm isohyetal map, and **Figure 3** shows the comparison between the observed and computed hydrographs for Dry Creek at Vernon Street. The HEC-1 model was used to compute flood hydrographs at the streamgage locations, route the flows down to the downstream index locations, add the local flow above Steelhead Creek, and compute flood hydrographs for Upper NEMDC and Old Magpie Creek above and below their respective pumping stations. The computed flood hydrographs for Dry Creek at Steelhead Creek, Arcade Creek at Steelhead Creek, Upper NEMDC above and below the NEMDC Stormwater Pumping Station, and Old Magpie Creek above and below Pump 157, were provided to Hydraulic Design Section as historical flood input for this flood event. The pumping station locations are shown on **Plate 1**.

Figure 3 presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the observed hydrograph. **Table 7** presents a comparison for the peak, and 1-, 3-, and 5-day volumes between the computed hydrographs and the observed hydrographs for the Dry Creek and Arcade Creek gaging stations.

Figure 3

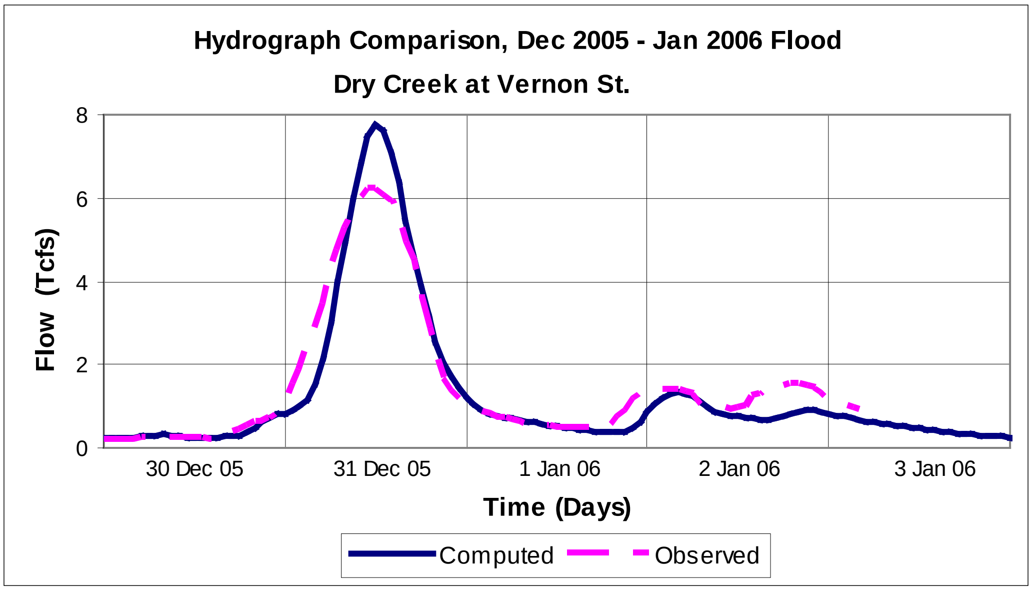


Table 7
30 December 2005 - 3 January 2006 Flood Volume Comparison
For Three Steelhead Creek Tributary Streamflow Gaging Stations

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Dry Creek at Royer Park				
Observed Hydrograph	5,240	3,040	1,620	------
2006 HEC-1 Run	6,230	2,870	1,330	916
% Difference	18.9%	-5.6%	-17.9%	------
Dry Creek at Vernon St.				
Observed Hydrograph	6,250	3,820	1,930	1,424
2006 HEC-1 Run	7,760	3,920	1,810	1,252
% Difference	24.2%	2.6%	-6.2%	-12.1%
Arcade Cr. near Del Paso Heights				
Observed Hydrograph	3,460	1,900	835	536
2006 HEC-1 Run	3,240	1,870	846	561
% Difference	-6.4%	-1.6%	1.3%	4.6%

b. February 1986 Flood**.** According to **Reference 7,** Dry Creek, Placer and Sacramento Counties, California, Hydrology Office Report, revised April 1988, runoff from a large storm event like that of February 1986, can only be estimated, due to a lack of adequate streamflow data. The Dry Creek gage does not function correctly for flows above 2,000 cfs. Peak flows above that are estimated using highwater marks and slope-area measurements by the State of California. The peak flow of 13,100 cfs and associated one-day flow of 5,800 cfs listed in **Reference 7** for the February 1986 flood for Dry Creek at the Vernon Street gage are based upon a flood reconstitution, using the HEC-1 model and rainfall recording data. The flood reconstitution HEC-1 run could not be located, but available data included the reconstituted flood hydrograph for Dry Creek at Roseville, 5-day storm totals, and rainfall recording data for several stations.

Plate 6 shows the isohyetal map created for the 15 - 19 February 1986 storm, based on the station precipitation totals listed on **Table 6**. **Plate 6** may not necessarily be an accurate isohyetal map of the storm, but it shows approximate isolines of the 5-day storm amounts used in the HEC-1 model to develop the flood hydrographs for the Natomas tributaries. Eight precipitation gages used for storm distribution patterns are identified with “**” in the February 1986 rainfall column of **Table 6**. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 9 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.05

No base flow was used for the lower elevation subbasins in the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss. The watershed was wet from three days of rain prior to 15 February, the start of the maximum five-day flow.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. **Figure 4** presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the previously reconstituted flood hydrograph from **Reference 7**. **Table 8** presents a comparison for the peak, and 1-, 3-, and 5-day volumes for the two hydrographs.

Figure 4

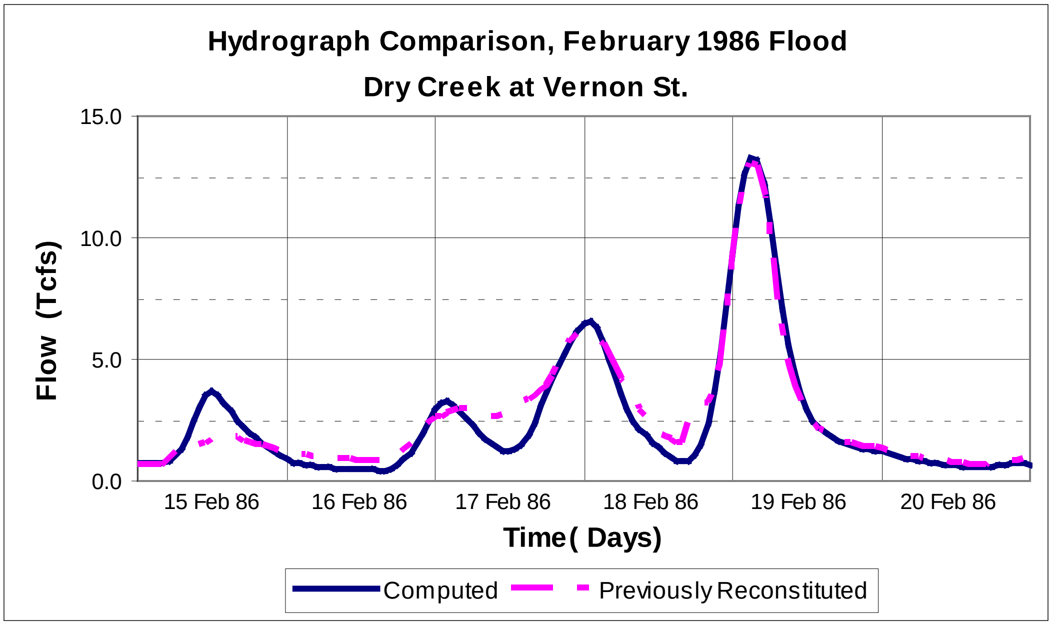


Table 8
15 – 19 February 1986 Flood Volume Comparison
Dry Creek at Roseville Gage

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Ref 7 Hydrograph (1988)	13,100	5,930	4,160	2,980
2008 HEC-1 Run	13,000	5,980	3,810	2,850
% Difference	-0.8%	0.8%	-8.4%	-4.4%

c. <u>January 1995 Flood</u>. The 8 - 12 January 1995 storm had a very intense 6-hour period of rainfall the evening of 9 January that produced the peak flow of record on Dry Creek. **Reference 8**, “Use of Radar-Rainfall Estimates to Model the January 9 - 10, 1995 Floods in Sacramento, CA,” paper presented October 1995, explains how data from a network of rain gages were combined with radar-rainfall estimates from the National Weather Service WSR-88D radar observations to reconstitute the flood hydrograph for Dry Creek at Roseville and estimate flood hydrographs for other locations in the watershed. The HEC-1 model used a 5-minute time increment for one hundred small subbasins above the Dry Creek at Roseville gage for a 3-day hydrograph. Each subbasin or small group of subbasins had its own rainfall distribution pattern.

The Natomas GRR study is more concerned with 5-day volumes than those of shorter duration, so the rainfall period was extended back one day, to include 8 January. The Natomas GRR HEC-1 model listed in **Reference 6,** Attachment 1 was used instead of the 5-minute HEC-1 model described in **Reference 8**. The **Reference 6** model has 28 subbasins above the Dry Creek at Roseville gage instead of the 100 subbasins in the **Reference 8** model. The nearly one hundred 5-minute rainfall distribution patterns in the **Reference 8** HEC-1 model were reduced to eight patterns to distribute the January 1995 storm for the Natomas GRR HEC-1 model. The 5-minute rainfall distribution patterns were converted to hourly increments, and extended back to 8 January using the CDEC rainfall gage for Lincoln (LCN). **Plate 7** is not an accurate isohyetal map of the storm, but it shows approximate isolines of the 5-day storm amounts used in the HEC-1 model to develop the flood hydrographs for the Natomas tributaries. The isolines were based on the station precipitation totals listed on **Table 6** and subbasin storm totals in the **Reference 8** HEC-1 model. Very little rain fell on 11-12 January. The HEC-1 model for this American River GRR study was run for a 5-day time period. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 3 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.10

No base flow was used for the rest of the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. **Figure 5** presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the observed flood hydrograph shown on Figure 12 of **Reference 8**, the radar-rainfall report. The rainfall distribution patterns used in the HEC-1 model produced a hydrograph with two peaks flows, not one. The higher peak is still similar in magnitude and timing to the observed peak, and the three-day volumes are nearly the same. **Table 9** presents a comparison for the peak, and 1-, and 3-day volumes for the two hydrographs. The computed Dry Creek hydrograph has only a single peak by the time it is routed down to Steelhead Creek and added to the local flow.

Figure 5

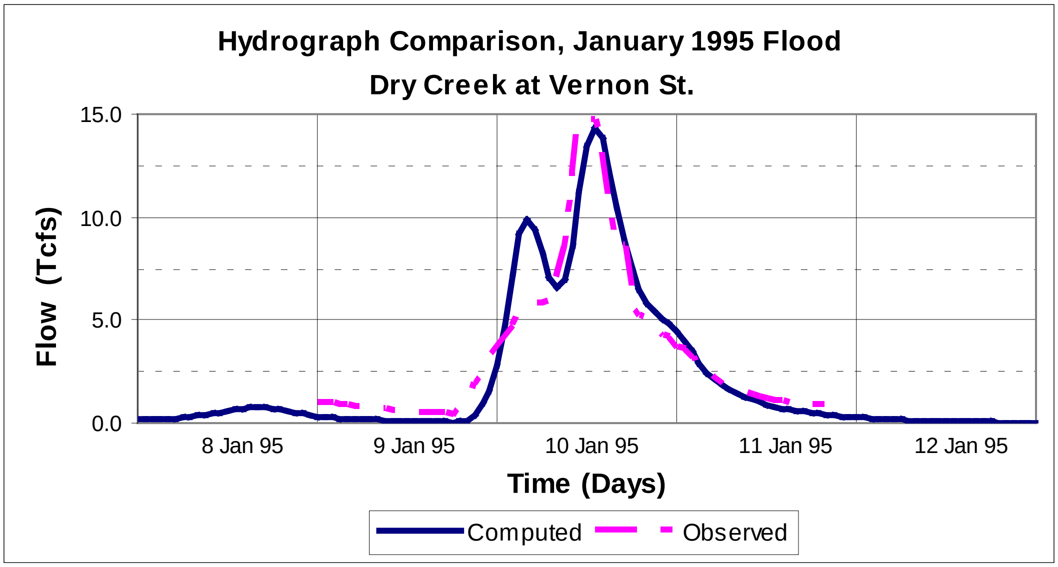


Table 9
8 – 12 January 1995 Flood Hydrograph Comparison
Dry Creek at Roseville Gage

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Observed Hydrograph	14,800	7,580	3,380	------
2008 HEC-1 Run	14,400	8,390	3,360	2,120
% Difference	-2.7%	10.7%	-0.6%	------

d. 29 Dec 1996 – 3 Jan 1997 Flood. Recording rainfall data for numerous stations were available on the CDEC website for January 1997. **Table 6** lists the storm totals for these and the daily rainfall stations. The 5-day storm period for the 1997 New Years storm is from 29 December 1996 to 2 January 1997. An isohyetal map was created, based on the storm amounts for this time period, shown on **Table 6**, and subbasin storm amounts were estimated for the HEC-1 model. Nine precipitation stations, identified with "**" in the Dec '96 – Jan '97 rainfall column of **Table 6**, were used as rainfall distribution patterns in the HEC-1 model. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 3 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.05

No base flow was used for the rest of the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. These hydrographs are of greater importance than merely as reconstituted hydrographs for this flood event. The shapes of these computed hydrographs for the 5-day period 30 Dec 1996 to 3 Jan 1997 are used as the 5-day pattern hydrographs in the Coincident Frequency Analysis. The 5-day flood hydrograph patterns used in the Comprehensive Study as Sacramento River tributary input hydrographs, prior to their re-distribution to the upstream reservoirs for the Comp Study reservoir operations modeling, are either the observed or computed unregulated tributary hydrographs for that 5-day period, 30 Dec 1996 to 3 Jan 1997. With all the tributary hydrographs for the same 5-day period, timing for high flows on the Natomas tributaries should historically match their actual timing with respect to timing of the other streams, including the Sacramento River at Verona flood hydrograph for the New Year 1997 flood event.

The observed flows for this flood event at the stream gages on Dry and Arcade creeks and the flood hydrographs routed to the downstream index points showed the flood to be a 30 percent chance or more frequent event for Natomas, compared with the large, low frequency flows occurring on many other Sacramento River tributaries. It would be difficult to justify basing the shapes of floods up to the 0.2 percent event upon a 30 percent chance event, so the HEC-1 model was revised. The observed storm amounts were raised by between 15 and 45 percent, to compute a somewhat rarer flood event, on which to base the synthetic flood hydrographs. With enhanced rainfall and higher runoff, the 8-Flood Series flood patterns are based on a 15 percent chance 5-day flood event. Exceedence estimates of the 5-day volumes for the six historic floods are discussed in **Section 2.1.g**. **Plate 8** shows the revised isohyetal map with the higher rainfall amounts used to develop subbasin storm totals in the HEC-1 model to develop Natomas tributary flood hydrographs

Figure 6 presents the flood hydrograph from the HEC-1 run with the increased rainfall for Dry Creek at Roseville compared with the observed mean day flow hydrograph for the Vernon Street gage. **Figure 7** presents the flood hydrograph from the HEC-1 run for Arcade Creek near Del Paso Heights USGS gage compared with the observed mean day flow hydrograph for the gage. The bars on **Figures 5 and 6** represent the observed peak flows for Dry and Arcade creeks at their respective gaging stations. **Table 10** presents a comparison for the peak, and 1-, and 3-day volumes between the computed hydrograph and the mean day flow hydrograph published for the gage. The 5-day period, 30 December 1996 to 3 January 1997, is the period for which the computed 5-day hydrographs for Dry and Arcade creeks at their confluences with Steelhead Creek and Upper NEMDC and Old Magpie Creek above their respective pumping stations are the pattern hydrographs used for the 8-Flood synthetic series.

Figure 6

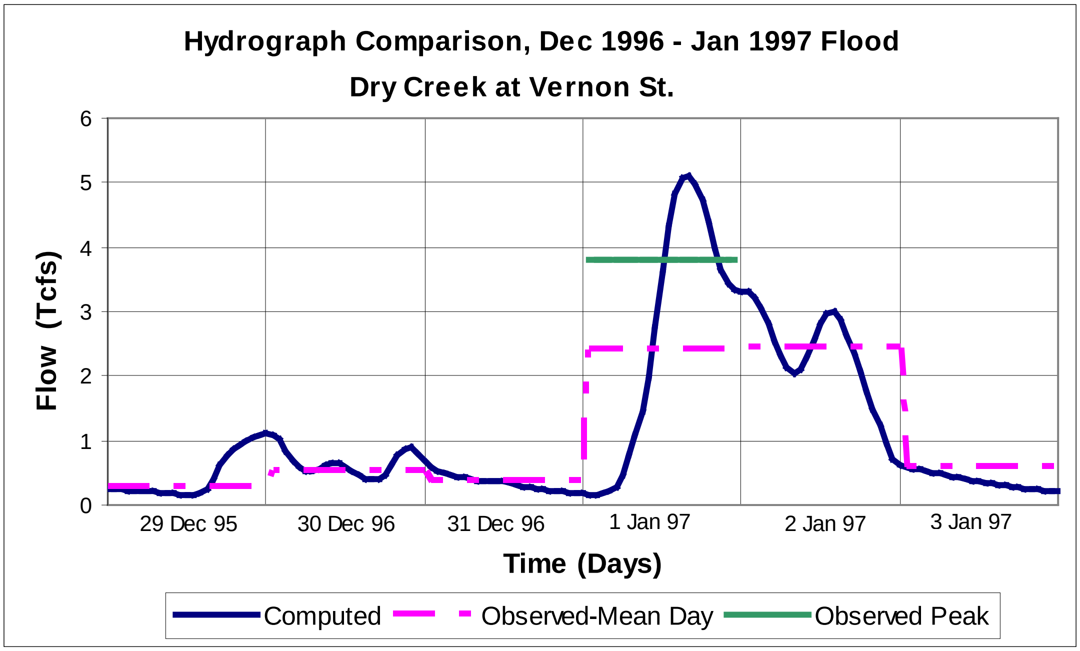


Figure 7

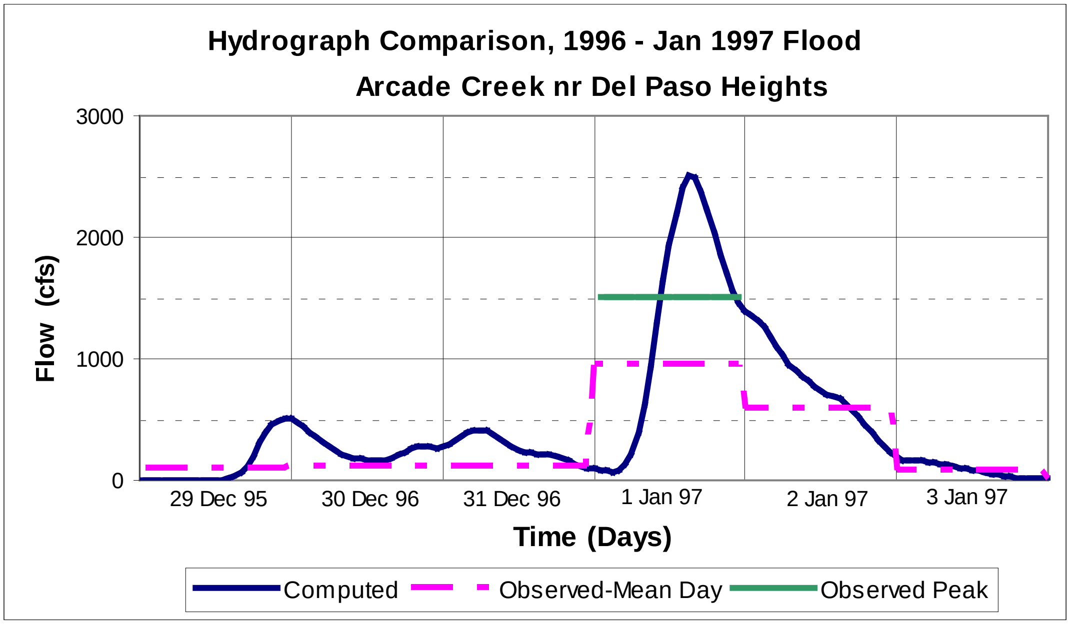


Table 10
29 December 1996 – 3 January 1997 Flood Volume Comparison
For Three Steelhead Creek Tributary Streamflow Gaging Stations

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Dry Creek at Vernon St.				
Observed Hydrograph	3,800	2,440	1,810	1,262
2008 HEC-1 Run	5,120	3,470	1,770	1,303
% Difference	34.7%	42.2%	-2.2%	3.3%
Magpie Cr. near Del Paso Heights				
Observed Hydrograph	N/A	81	35	25
2008 HEC-1 Run	320	108	47	31
% Difference	------	33.3%	35.6%	22.0%
Arcade Cr. near Del Paso Heights				
Observed Hydrograph	1,510	945	551	373
2008 HEC-1 Run	2,507	1,630	778	558
% Difference	66.0%	72.5%	41.2%	49.5%

e. Mid-January 1997 Flood. The mid-January 1997 flood was not an especially rare flood event for the higher elevation tributaries to the Sacramento River. However, for the Natomas tributaries, the mid-January rainfall was greater than for the New Year 1997 storm a few weeks earlier. The greater mid-January rainfall is reflected in the higher peak flows and runoff volumes for this event on the Natomas tributaries. Compare the difference between the Dry Creek hydrographs shown on **Figure 6** and **Figure 8**. The peak flow on Arcade Creek was 150 percent of the peak flow there three weeks earlier. The rainfall from **Table 6** for the 22-26 January 1997 storm was used to develop a storm isohyetal map for the HEC-1 model. **Plate 9** may not necessarily be an accurate isohyetal map of the storm, but it shows approximate isolines of the 5-day storm amounts used in the HEC-1 model to develop the flood hydrographs for the Natomas tributaries. The observed mean day flood hydrographs for Vernon Street, Magpie Creek and Arcade Creek near Del Paso Heights were used as the observed hydrographs for the comparison between observed and computed flood hydrographs in **Table 11**. Ten precipitation stations, identified with "**" in the 22-26 January 1997 rainfall column of **Table 6**, were used as storm distribution patterns. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 3 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.05

No base flow was used for the rest of the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. **Figure 8** presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the mean day hydrograph observed for the Vernon Street gage. Timing of the observed peak flows of 7,950 cfs and 7,250 cfs is based on the time that the highest stages occurred. The computed peak flows are not the same as the observed peak flows, but the observed peak flows are only one hour earlier than the computed peak flows, which is better timing than for the New Year 1997 flood hydrograph reproduction. There is not much difference between the computed and the observed 5-day flood volumes for Dry Creek. **Table 11** presents a comparison for the peak, and 1-, 3-, and 5-day volumes for the three gaging stations.

Figure 8

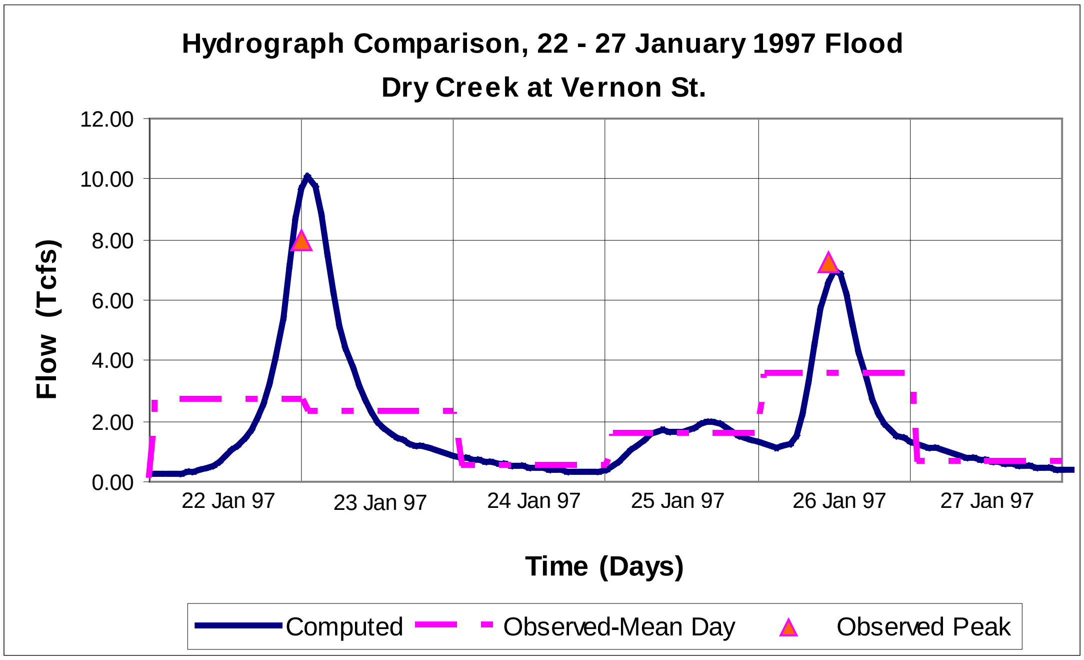


Table 11
22 - 26 January 1997 Flood Volume Comparison
For Three NEMDC Tributary Streamflow Gaging Stations

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Dry Creek at Vernon St.				
Observed Hydrograph	7,950	3,550	1,886	2,142
2008 HEC-1 Run	10,060	4,810	2,200	2,204
% Difference	26.5%	35.5%	16.6%	2.9%
Magpie Cr. near Del Paso Heights				
Observed Hydrograph	560	128	47	47
2008 HEC-1 Run	570	107	45	49
% Difference	1.8%	-16.4%	-4.5%	3.2%
Arcade Cr. near Del Paso Heights				
Observed Hydrograph	2,270	1,090	591	679
2008 HEC-1 Run	3,410	1,730	714	748
% Difference	50.2%	58.7%	20.8%	10.2%

f. February 1998 Flood. Another large storm occurred over the Natomas tributaries watershed in February 1998. The storm amounts for 2 - 6 February 1998 on **Table 6** were used to create a storm isohyetal map for the event, and subbasin storm amounts were used in the HEC-1 model. **Plate 10** may not necessarily be an accurate isohyetal map of the storm, but it shows approximate isolines of the 5-day storm amounts used in the HEC-1 model to develop the flood hydrographs for the Natomas tributaries. The observed mean day flood hydrographs for the Vernon Street and Arcade Creek near Del Paso Heights gages were used for the comparison between the observed and computed flood hydrographs. Ten precipitation stations, identified with "**" in the 2-6 February 1998 rainfall column of **Table 6**, were used as storm distribution patterns. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 3 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.05

No base flow was used for the rest of the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. **Figure 9** presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the mean day hydrograph observed for the Vernon Street gage. The observed peak flow at Vernon Street gage occurred two hours earlier than the computed peak flow in the HEC-1 run. There is not much difference between the computed and the observed 5-day flood volumes for the Dry and Arcade creek gages. **Table 12** presents a comparison for the peak, and 1-, 3-, and 5-day volumes for the two gaging stations.

Figure 9

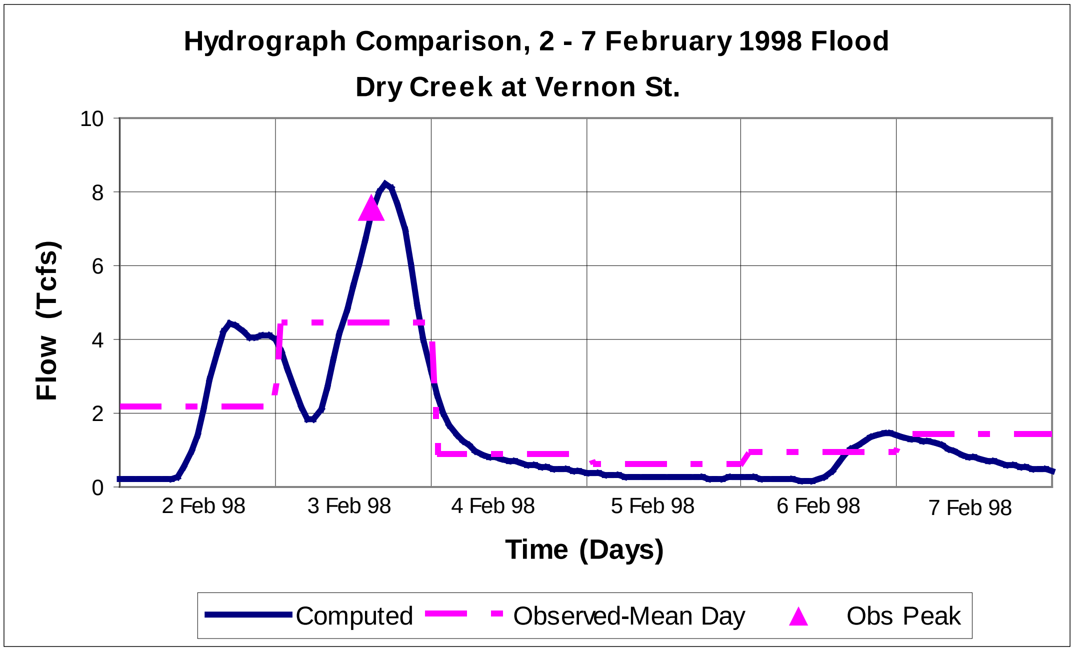


Table 12
2 - 6 February 1998 Flood Volume Comparison
For Two Steelhead Creek Tributary Streamflow Gaging Stations

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Dry Creek at Vernon St.				
Observed Hydrograph	7,549	4,420	2,489	1,791
2008 HEC-1 Run	8,240	4,840	2,620	1,822
% Difference	9.2%	9.5%	5.2%	1.7%
Arcade Cr. Near Del Paso Heights				
Observed Hydrograph	3,320	1,910	1,069	715
2008 HEC-1 Run	3,190	2,100	1,120	718
% Difference	-3.9%	9.9%	4.7%	0.4%

g. <u>5-Day Volume Frequency Relationships</u>. **Table 13** lists the 5-day flood volumes for the 8-Flood Series for the Steelhead Creek and Natomas Cross Canal tributaries at their downstream index points. The NEMDC Sum in **Table 13** below is the maximum 120 hours of the Steelhead Creek hydrograph developed by adding the 4 tributary hydrographs together at their respective downstream index points. The NEMDC Sum is not necessarily the sum of the four tributary hydrograph volumes, because the maximum 120 hours for the tributary hydrographs do not have the exact same starting and ending times. The 5-day volume frequency curves for Steelhead Creek and Natomas Cross Canal are shown on **Plates 11 and 12**.

Table 13

Summary Table - 8-Flood Series - Five-Day Duration Volumes

Stream at at Mouth	D.A. (sq.mi.)	8-Flood Series Five-Day Volumes (in Acre-Feet)							
		50%	20%	10%	4%	2%	1%	0.50%	0.20%
Steelhead Cr									
Dry Cr. at NEMDC	116.48	9,250	15,450	19,800	26,600	31,000	35,600	39,800	47,200
Upper NEMDC	27.13	2,010	3,230	4,110	5,300	6,190	7,120	7,980	9,360
OldMag at NEMDC (5-DAY)	4.57	380	594	747	952	1,103	1,260	1,410	1,640
Arcade Cr. At NEMDC	40.14	3,400	5,310	6,650	8,430	9,710	11,050	12,300	14,260
NEMDC Sum	188.32	14,970	24,600	31,340	41,320	48,020	54,980	61,360	71,750
Cross Canal									
Coon Creek at WPRR	112.61	8,760	15,640	20,360	29,430	34,360	39,410	44,040	51,430
Markham Rav. at WPRR	32.36	1,840	3,310	4,370	5,660	6,700	7,760	8,810	10,480
Auburn Rav. at WPRR	79.97	6,770	11,250	14,290	19,460	22,500	25,660	28,600	33,250
Pl.Grove Cr. at WPRR	46.69	4,140	6,500	8,110	10,360	11,880	13,390	15,080	17,420
Curry Creek at WPRR	16.59	1,190	2,000	2,560	3,300	3,850	4,420	4,950	5,810
Cross Canal Sum	288.22	22,690	38,710	49,680	68,160	79,230	90,580	101,420	118,320

The 5-day volumes in **Table 13** and the volume frequency curves on **Plate 11** were used to estimate the percent exceedence of the 5-day volumes for Steelhead Creek for the six historical flood events described above. **Table 14** lists the 5-day volumes for the Steelhead Creek tributaries computed using the HEC-1 program and the storm isohyetal maps for the 6 historical floods, along with the estimated percent exceedence of the 5-day volume for Steelhead Creek hydrographs.

Table 14

5-Day Volume Frequency Relationships for Six Historical Storms

Steelhead Creek Tributaries

Steelhead Cr Index Pt	5-Day Volume (ac-ft)	5-Day Volume % Chance Event (%)	Steelhead Cr Index Pt	5-Day Volume (ac-ft)	5-Day Volume % Chance Event (%)
Feb 1986 Storm			Mid-Jan 1997 Storm		
Dry Cr. At Mouth	38,400	0.6%	Dry Cr. At Mouth	28,500	2.6%
Arcade Cr.-Del Paso Hghts	10,700	0.6%	Arcade Cr.-Del Paso Hghts	7,420	4.6%
Arcade Cr. at Mouth	12,200	0.6%	Arcade Cr. At Mouth	8,300	4.4%
Upper NEMDC abv. Pump	7,090	1.0%	Upper NEMDC abv. Pump	4,230	9.3%
Old Magpie Cr. abv. Pump	1,420	0.6%	Old Magpie Cr. Abv. Pump	810	8.0%
Steelhead Sum	58,300	0.7%	Steelhead Sum	41,600	3.6%
Jan 1995 Storm			Feb 1998 Storm		
Dry Cr. At Mouth	29,800	2.2%	Dry Cr. At Mouth	24,100	5.1%
Arcade Cr.-Del Paso Hghts	8,300	2.7%	Arcade Cr.-Del Paso Hghts	7,380	5.7%
Arcade Cr. at Mouth	9,540	2.3%	Arcade Cr. At Mouth	8,100	4.9%
Upper NEMDC abv. Pump	5,430	3.6%	Upper NEMDC abv. Pump	4,540	7.3%
Old Magpie Cr. abv. Pump	930	4.6%	Old Magpie Cr. Abv. Pump	780	9.0%
Steelhead Sum	45,700	2.4%	Steelhead Sum	37,500	5.4%
New Year 1997 Storm			New Year 2006 Storm		
Dry Cr. At Mouth	17,400	14.5%	Dry Cr. At Mouth	17,700	13.8%
Arcade Cr.-Del Paso Hghts	5,300	15.6%	Arcade Cr.-Del Paso Hghts	5,430	14.6%
Arcade Cr. at Mouth	6,100	13.5%	Arcade Cr. At Mouth	6,370	11.8%
Upper NEMDC abv. Pump	3,370	18.4%	Upper NEMDC abv. Pump	2,820	28.0%
Old Magpie Cr. abv. Pump	600	19.5%	Old Magpie Cr. Abv. Pump	700	13.0%
Steelhead Sum	27,500	14.6%	Steelhead Sum	27,600	14.4%

A sensitivity analysis of storm centerings and runoff discussed in the Natomas GRR Hydrology Appendix showed there was less than a 5 percent difference in runoff on Steelhead Creek for a 1 percent storm centering on the Steelhead drainage and a concurrent storm on Steelhead Creek with the specific centering on Cross Canal drainage. The difference in runoff was also less than 5 percent for the Natomas Cross Canal. To simplify Natomas flood centerings for the Coincident Frequency Analysis, an n-percent chance flood is assumed to be centered on the combined drainages of Steelhead Creek and Natomas Cross Canal. So, if the 5-day flood hydrograph for Steelhead Creek for the New Year 1997 flood is a 15 percent exceedence event, it is assumed to be a 15 percent exceedence event for the Natomas Cross Canal 5-day runoff volume as well. Based on the flood volumes listed in **Table 13**, the 5-day volume of the New Year 1997 flood for the Natomas Cross Canal should be about 43,300 acre-feet. Based on this combined 5-day flood volume for the Cross Canal, 5-day flood hydrographs needed to be computed for the five Cross Canal tributaries for the New Year 1997 flood, to be used in the Coincident Frequency Analysis. Computation of the Natomas Cross Canal tributary hydrographs for the New Year 1997 flood and other five historic floods is discussed in **Section 2.2**.

2.2 Natomas Cross-Canal Historical Flood Hydrographs.

a. Computing 5-Day Volumes for 6 Historical Floods on Natomas Cross Canal. There are several problems with developing historical flood hydrographs for the Natomas Cross Canal tributaries. One is the lack of precipitation stations in the Cross Canal watershed. See **Plate 2**, the watershed map showing the precipitation station locations. Also, there are no flow gages – only a few stage gages on Pleasant Grove Creek at and upstream of Fiddyment Road, and in the upper watersheds of Coon Creek and Auburn Ravine. Coon Creek and Auburn Ravine stage gage locations can be found at **Reference 9,** on the map of Sacramento County ALERT gages. The Pleasant Grove Creek stage gage locations can be found at **Reference 10**, the map of City of Roseville Flood Alert gages. The isohyetal lines on the isohyetal maps for the six historic storms (**Plates 5 through 10**) were extended from Steelhead Creek drainage north through the Cross Canal drainage.

The Civil Engineering Solutions HEC-1 models and the isohyetal maps (**Plates 5 through 10**) were used to compute preliminary runoff hydrographs for the Cross Canal tributaries for the six historical floods. The storm isohyetal maps and subbasins storm amounts for the Cross Canal tributaries were adjusted until the 5-day runoff volumes for the Cross Canal tributaries matched the percent exceedence of the 5-day Steelhead Creek tributary volumes for the same event. (See **Table 14**.) The Pleasant Grove Creek and Markham Ravine drainages are similar to Arcade Creek in east-to-west alignment, drainage area, and elevation range (below 300 feet), so that the percent exceedence event for the Arcade Creek 5-day flood volumes were used as guidance to estimate the flood volumes for those two Cross Canal tributaries. For the larger tributaries, Coon Creek and Auburn Ravine, with large contributing drainage above 300 feet (extending up to 2,000 feet for Coon Creek), the percent exceedence 5-day volumes for the six historical floods were based on the percent exceedence flood volumes for Dry Creek at Steelhead Creek. Curry Creek is adjacent to Upper NEMDC, which was used as a model in case the 5-day volumes on Curry Creek needed adjustment.

Table 15 lists the computed 5-day flood volumes from the above adjusted modeling runs for the Natomas Cross Canal tributaries, as well as the ratios of peak-to-5-day-volume for the computed hydrographs on the Steelhead Creek and Cross Canal tributaries. The HEC-1 models developed by Civil Engineering Solutions, Inc., for the Natomas Cross canal tributaries, discussed in the Natomas GRR Hydrology Appendix (**Reference 6**), assumed that future housing and urbanization projects were in place. At the present time, they have yet to be constructed. One review comment on the Hydrology Appendix was that the Cross Canal tributary peak flows computed for the Hydrology Appendix had much higher peak flows in proportion to their flood volumes and contributing drainage areas. The relationship for Cross Canal peak flows should be more in line with the ratios of peak flow to flood volume and to drainage area for the Steelhead Creek tributaries.

Table 15
Ratio of Peaks to 5-Day Volumes
for 6 Historical Floods on Natomas Tributaries

Stream at Mouth	D.A. (sq.mi.)	8-Flood Series - Peaks, Volumes and Ratios: Peak to Volume						Average Peak to Volume
		Feb-86	Jan-95	NY 1997	MidJan 97	Feb-98	NY 2006	
Steelhead Cr								
Dry Cr. At Steelhead Cr.	Peak (cfs)	10,040	12,080	5,110	7,830	7,350	6,900	
5-day Vol. (ac-ft)		38,400	29,800	17,400	28,500	24,100	17,700	
Drainage Area 116.48 sq.mi.	PK/Vol.	0.26	0.41	0.29	0.27	0.30	0.39	0.32
Upper NEMDC	Peak (cfs)	3,830	3,840	2,610	2,610	1,610	2,110	
5-day Vol. (ac-ft)		7,090	5,430	3,370	4,230	4,540	2,820	
Drainage Area 27.13 sq.mi.	PK/Vol.	0.54	0.71	0.77	0.62	0.35	0.75	0.62
Old Magpie Cr. above Pump	Peak (cfs)	831	918	603	673	389	573	
5-day Vol. (ac-ft)		1,420	930	600	810	780	700	
Drainage Area 4.57 sq.mi.	PK/Vol.	0.59	0.99	1.01	0.83	0.50	0.82	0.79
Arcade Cr. At Steelhead Cr.	Peak (cfs)	3,720	4,950	2,640	3,470	3,200	3,360	
5-day Vol. (ac-ft)		12,200	9,540	6,100	8,300	8,100	6,370	
Drainage Area 40.14 sq.mi.	PK/Vol.	0.30	0.52	0.43	0.42	0.40	0.53	0.43
Steelhead Cr. Sum	Peak (cfs)	14,060	17,840	8,470	11,300	11,050	10,860	
5-day Vol. (ac-ft)		58,300	45,700	27,500	41,600	37,500	27,600	
Drainage Area 188.32 sq.mi.	PK/Vol.	0.24	0.39	0.31	0.27	0.29	0.39	0.32
Cross Canal		Feb-86	Jan-95	NY 1997	MidJan 97	Feb-98	NY 2006	Average
Coon Creek at WPRR	Peak (cfs)	11,700	26,500	8,250	13,700	10,150	9,970	
5-day Vol. (ac-ft)		35,500	29,100	17,600	20,700	18,050	13,460	
Drainage area 112.61 sq.mi.	PK/Vol.	0.33	0.91	0.47	0.66	0.56	0.74	0.61
Markham Rav. At WPRR	Peak	6,510	4,830	2,520	4,810	2,550	4,120	
5-day Vol. (ac-ft)		8,620	4,850	3,700	5,280	5,130	3,440	
Drainage Area 32.36 sq.mi.	PK/Vol.	0.76	1.00	0.68	0.91	0.50	1.20	0.84
Auburn Rav. At WPRR	Peak (cfs)	11,700	10,200	4,290	6,840	5,490	5,700	
5-day Vol. (ac-ft)		26,450	21,000	12,500	16,360	14,100	10,200	
Drainage Area 79.97 sq.mi.	PK/Vol.	0.44	0.49	0.34	0.42	0.39	0.56	0.44
Pl.Grove Cr. At WPRR	Peak (cfs)	7,870	9,100	4,550	7,360	4,610	5,470	
5-day Vol. (ac-ft)		14,900	11,400	6,560	9,090	9,330	6,160	
Drainage Area 46.69 sq.mi.	PK/Vol.	0.53	0.80	0.69	0.81	0.49	0.89	0.70
Curry Creek at WPRR	Peak (cfs)	2,520	2,500	1,570	1,680	1,020	1,290	
5-day Vol. (ac-ft)		4,650	3,330	2,130	2,890	3,000	1,730	
Drainage Area 16.59 sq.mi.	PK/Vol.	0.54	0.75	0.74	0.58	0.34	0.75	0.62
Cross Canal Sum	Peak (cfs)	30,700	43,600	16,100	23,200	20,800	21,300	
5-day Vol. (ac-ft)		89,800	72,900	42,500	54,300	49,500	35,000	
Drainage Area 288.22 sq.mi.	PK/Vol.	0.34	0.60	0.38	0.43	0.42	0.61	0.46

Upper NEMDC (Steelhead tributary) and Curry Creek (Cross Canal tributary) are adjacent basins on the valley floor and have similar ratios of computed peak to 5-day volume for each of the six flood events. The 6-event averaged ratio of peak/5-day volume (**Table 15**, right-hand column) is the same, 0.62, for Upper NEMDC and Curry Creek.

Arcade Creek (Steelhead tributary) and Pleasant Grove Creek and Markham Ravine (Cross Canal tributaries) are similar in orientation and elevation. However, because of the highly urbanized HEC-1 models used for Pleasant Grove Creek and Markham Ravine, the 6-event averaged ratio of peak/5-day volume for Pleasant Grove Creek is 60 percent higher than for Arcade Creek and for Markham Ravine is nearly two times that of Arcade Creek.

Dry Creek (Steelhead tributary) and Coon Creek and Auburn Ravine (Cross Canal tributaries) have larger drainage areas as well as headwaters at much higher elevations than the other Natomas tributaries. Because of the highly urbanized HEC-1 models used for Auburn Ravine and Coon Creek, the 6-event averaged ratio of peak/5-day volume for Auburn Ravine is 38 percent higher than for Dry Creek and is 91 percent higher for Coon Creek than for Dry Creek.

Table 16 shows the ratios of peak-to-drainage-area for the computed hydrographs on the Steelhead Creek and Cross Canal tributaries.

Table 16
Ratio of Peaks to Drainage Areas
for 6 Historical Floods on Natomas Tributaries

Stream at Mouth	D.A. (sq.mi.)	8-Flood Series - Ratios of Peaks to Drainage Areas						Average Peak to D.A.
		Feb-86	Jan-95	NY 1997	MidJan 97	Feb-98	NY 2006	
Steelhead Cr								
Dry Cr. At Steelhead Cr.	Peak (cfs)	10,040	12,080	5,110	7,830	7,350	6,900	
Drainage Area (sq.mi.)								
116.48	Pk/D.A.	86.2	103.7	43.9	67.2	63.1	59.2	70.6
Upper NEMDC	Peak (cfs)	3,830	3,840	2,610	2,610	1,610	2,108	
Drainage Area (sq.mi.)								
27.13	Pk/D.A.	141.2	141.5	96.2	96.2	59.3	77.7	102.0
Old Magpie Cr. above Pump	Peak (cfs)	831	918	603	673	389	573	
Drainage Area (sq.mi.)								
4.57	Pk/D.A.	181.8	200.9	131.9	147.3	85.1	125.4	145.4
Arcade Cr. At Steelhead Cr.	Peak (cfs)	3,720	4,950	2,640	3,470	3,200	3,360	
Drainage Area (sq.mi.)								
40.14	Pk/D.A.	92.7	123.3	65.8	86.4	79.7	83.7	88.6
Steelhead Cr. Sum	Peak (cfs)	14,060	17,840	8,470	11,300	11,050	10,860	
Drainage Area (sq.mi.)								
188.32	Pk/D.A.	74.7	94.7	45.0	60.0	58.7	57.7	65.1
Cross Canal		Feb-86	Jan-95	NY 1997	MidJan 97	Feb-98	NY 2006	Average
Coon Creek at WPRR	Peak (cfs)	11,700	26,500	8,250	13,700	10,150	9,970	
Drainage Area (sq.mi.)								
112.61	Pk/D.A.	103.9	235.3	73.3	121.7	90.1	88.5	118.8
Markham Rav. At WPRR	Peak (cfs)	6,510	4,830	2,520	4,810	2,550	4,120	
Drainage Area (sq.mi.)								
32.36	Pk/D.A.	201.2	149.3	77.9	148.6	78.8	127.3	130.5
Auburn Rav. At WPRR	Peak (cfs)	11,700	10,200	4,290	6,840	5,490	5,700	
Drainage Area (sq.mi.)								
79.97	Pk/D.A.	146.3	127.5	53.6	85.5	68.7	71.3	92.2
Pl.Grove Cr. At WPRR	Peak (cfs)	7,870	9,100	4,550	7,360	4,610	5,470	
Drainage Area (sq.mi.)		14,900	11,400	6,560	9,090	9,330	6,160	
46.69	Pk/D.A.	168.6	194.9	97.5	157.6	98.7	117.2	139.1
Curry Creek at WPRR	Peak (cfs)	2,520	2,500	1,570	1,680	1,020	1,290	
Drainage Area (sq.mi.)								
16.59	Pk/D.A.	151.9	150.7	94.6	101.3	61.5	77.8	106.3
Cross Canal Sum	Peak (cfs)	30,700	43,600	16,100	23,200	20,800	21,300	
Drainage Area (sq.mi.)								
288.22	Pk/D.A.	106.5	151.3	55.9	80.5	72.2	73.9	90.0

The 6-event averaged ratio of peak/drainage area (**Table 16**, right-hand column) is nearly the same for the adjacent stream drainages, Upper NEMDC and Curry Creek, with ratios of 102 and 106.3, respectively. These basins are in close agreement for ratios of both peak to 5-day volume and peak to drainage area. The computed historical reproduction hydrographs for Curry Creek do not appear to need adjustment.

The 6-event averaged ratio of peak/drainage area for Arcade Creek is 88.6. While Markham Ravine and Pleasant Grove Creek are the tributaries to the Natomas Cross Canal most similar to Arcade Creek, the 6-event averaged ratio of peak/drainage area for Markham Ravine is 47 percent higher than for Arcade Creek and for Pleasant Grove Creek is 57 percent higher than for Arcade Creek. These higher ratios for the Cross Canal tributaries can be explained by the HEC-1 models that included future urbanization on those watersheds. The peak flows for present conditions on Markham Ravine and Pleasant Grove Creek should be lower.

The 6-event averaged ratio of peak/drainage area for Dry Creek is 70.6. The Cross Canal tributaries most similar to Dry Creek are Auburn Ravine and Coon Creek. The 6-event averaged ratio of peak/drainage area for Auburn Ravine is 31 percent higher than that for Dry Creek while the averaged ratio for Coon Creek is 68 percent higher than for Dry Creek. The peak flows for present conditions on Auburn Ravine and Coon Creek should be lower.

Based on the differences in the ratios presented in **Tables 15 and 16**, the hydrographs for Auburn Ravine, Coon Creek, Markham Ravine, and Pleasant Grove Creek were reshaped with lower peak flows. This process is explained in **Section 2.2.b**.

b. <u>Re-shaping the Natomas Cross Canal Historical Hydrographs</u>. Once the 5-day runoff volumes for the six historic floods on the Natomas Cross Canal tributaries were determined, the flood hydrographs were re-shaped (except for Curry Creek), with lower peak flows, more in line with the peak to volume and to drainage area ratios for the Steelhead Creek tributaries (**Tables 15 and 16** above). The same Steelhead Creek tributaries were used for the hydrograph patterns: Arcade Creek at Steelhead Creek as a pattern for Pleasant Grove Creek and Markham Ravine at their downstream WPRR index points, and Dry Creek at Steelhead Creek as a pattern for Auburn Ravine and Coon Creek at their downstream WPRR index points. The computed flood volumes for the Cross Canal tributaries remained the same, but volume lost by re-shaping for lower peak flows was offset by the addition of recession flow. The timing of the peak flows on the Cross Canal tributaries was not changed. Examples of re-shaping of the Cross Canal tributary hydrographs for the New Year 1997 flood are shown on **Figure 10**, Pleasant Grove Creek at WPRR, based on Arcade Creek, and **Figure 11**, Coon Creek at WPRR, based on Dry Creek at Steelhead Creek.

The figures show how the high peak flows on the Cross Canal tributaries were reduced by hydrograph re-shaping. Rapid hydrograph fluctuations were filled in. Recession base flow was added to the hydrographs for the Cross Canal tributaries with major contributing drainage above 300 feet (Coon Creek and Auburn Ravine). Minor waves in the flood hydrographs were not adjusted. While the Arcade Creek hydrograph appears to have base flow, the higher flow trailing after the main wave is due to water being pumped from interior drainage areas upstream of the mouth of Arcade Creek.

Figure 10

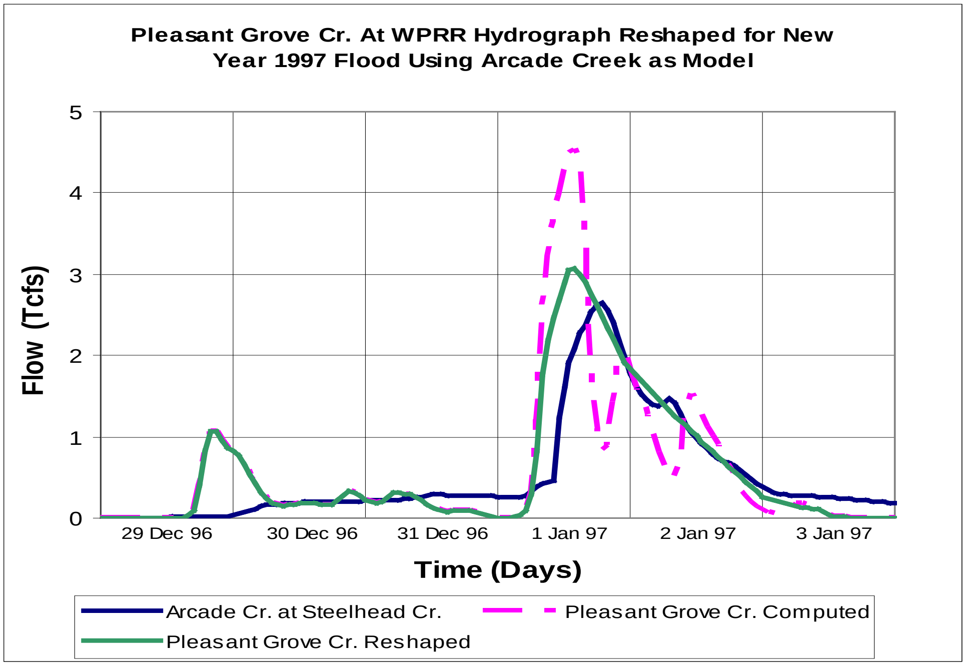
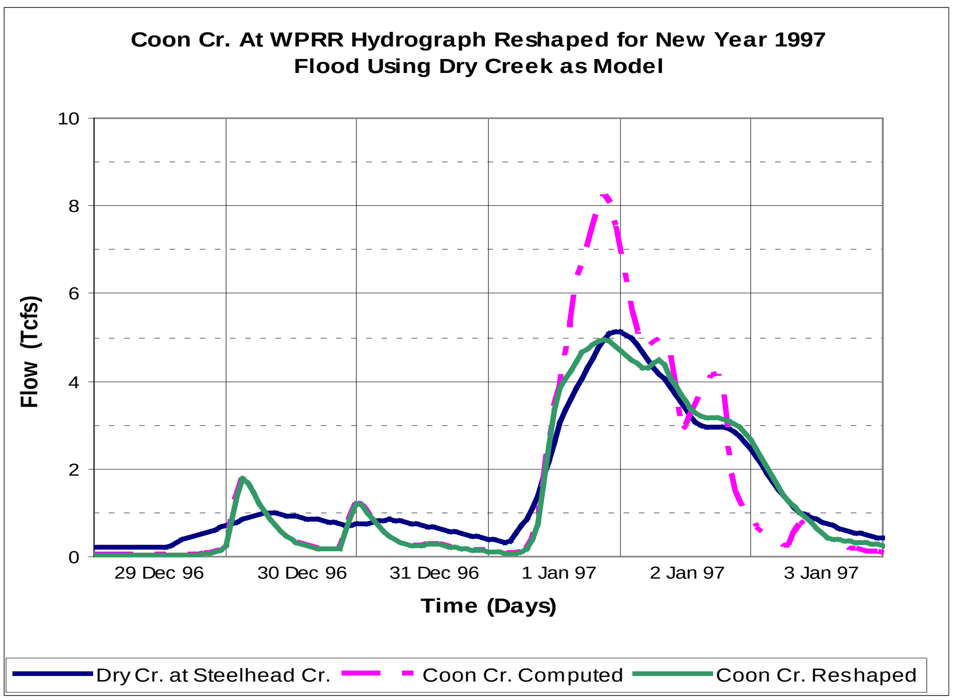
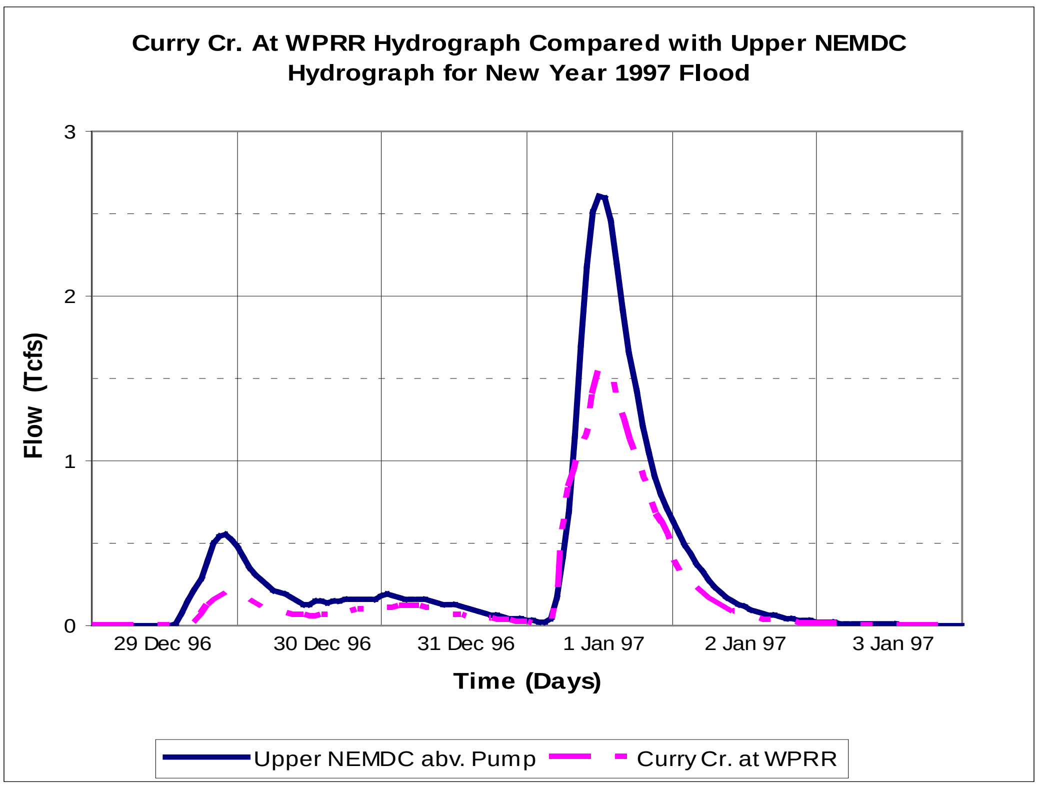


Figure 11



The smaller valley tributaries, Upper NEMDC and Old Magpie Creek, have higher peak flows in proportion to their flood volumes and drainage areas, but those peak flows would not have as much effect on the downstream Steelhead Creek hydrograph, even if they contributed directly to Steelhead Creek instead of being pumped in; their drainage areas and flood volumes are small compared with the larger tributaries, Dry and Arcade creeks. The contribution from Curry Creek to flows at the Natomas Cross Canal does not have a large effect either. The Rio Linda rainfall gage was used to distribute the precipitation over these two drainages for the six historical storms. The ratios of peak to flood volume and to drainage area for Curry Creek are very similar to the ratios for Upper NEMDC. The historical flood hydrograph for Curry Creek was not re-shaped. **Figure 12** presents the flood hydrographs for Curry Creek and Upper NEMDC for the New Year 1997 flood.

Figure 12



2.3 <u>Use of Historical Flood Hydrographs on Natomas Tributaries</u>. The Natomas tributary hydrographs for the six historic floods were provided to Hydraulic Design Section to be used for upstream boundary conditions in the hydraulic modeling. The historic flood hydrographs were at the following locations: Coon Creek at WPRR, Markham Ravine at WPRR, Auburn Ravine at WPRR, Pleasant Grove Creek at WPRR, Curry Creek at WPRR, Upper NEMDC above and below the NEMDC Stormwater Pumping Station, Dry Creek above Steelhead Creek confluence, Old Magpie Creek above and below Pump Station 157, and Arcade Creek above Steelhead Creek confluence. **Plate 13** shows the New Year 1997 computed flood hydrographs for Curry

Creek and the Steelhead Creek tributaries and the reshaped flood hydrographs for Pleasant Grove Creek, Auburn Ravine, Markham Ravine, and Coon Creek.

3.0 Development of 8-Flood Series Hydrographs for Natomas Tributaries

Development of the 8-Flood Series hydrographs for the Natomas tributaries follows Comprehensive Study methodology. The Comprehensive Study used 30-day hydrographs consisting of six 5-day waves, with the 4^{th} wave being the highest. The process includes: 1) obtaining the average flood flow rates from the unregulated frequency curves, 2) separating these average flows into wave volumes, and 3) distributing volumes into the 6-wave series.

All of the Natomas tributaries at their respective downstream index points are unregulated. The index points for Upper NEMDC and Old Magpie Creek are upstream of their respective pumping stations. The 5-day volume frequency curves for the Natomas tributaries are shown on **Plates 11 and 12. Plates 14 and 15** present the 10-day volume frequency curves. The 5-day volumes for the 8-Flood Series for the Natomas tributaries are listed on **Table 13** in **2.1.g. Table 17** below lists the 10-day volumes for the 8-Flood Series.

Table 17

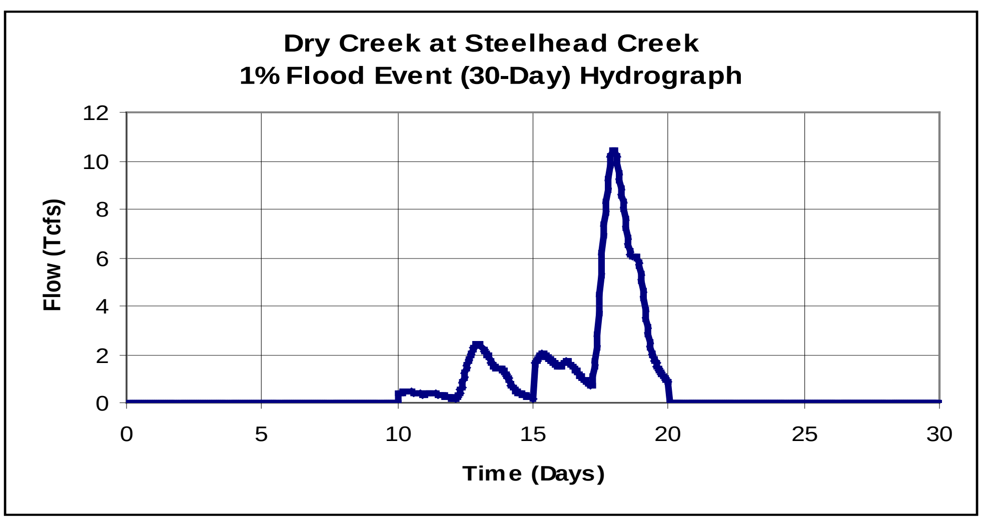
Summary Table - 8-Flood Series - Ten-Day Duration Volumes

Stream at	D.A.	8-Flood Series Five-Day Volumes (in Acre-Feet)							
at Mouth	(sq.mi.)	50%	20%	10%	4%	2%	1%	0.50%	0.20%
Steelhead Cr									
Dry Cr. at NEMDC	116.48	11,000	18,300	23,600	32,700	38,200	43,900	49,100	58,700
Upper NEMDC	27.13	2,400	3,840	4,920	6,400	7,510	8,700	9,760	11,500
OldMag at NEMDC (5-DAY)	4.57	470	724	891	1,200	1,390	1,590	1,770	2,070
Arcade Cr. at NEMDC	40.14	4,220	6,570	8,190	10,300	11,900	13,600	15,100	17,600
NEMDC Sum	188.32	18,090	29,434	37,601	50,600	59,000	67,790	75,730	89,870
Cross Canal									
Coon Creek at WPRR	112.61	10,900	19,500	25,400	38,300	44,700	51,400	57,600	67,300
Markham Rav. at WPRR	32.36	2,380	4,170	5,450	7,320	8,610	9,920	11,200	13,300
Auburn Rav. at WPRR	79.97	8,600	14,200	18,100	25,300	29,300	33,400	37,300	43,400
Pl.Grove Cr. at WPRR	46.69	5,160	8,060	10,200	13,100	15,000	17,000	19,200	22,100
Curry Creek at WPRR	16.59	1,490	2,490	3,180	4,120	4,820	5,540	6,230	7,330
Cross Canal Sum	288.22	28,530	48,420	62,330	88,140	102,430	117,260	131,530	153,430

For consistency with the Comprehensive Study, the computed New Year 1997 flood hydrographs for the Natomas tributaries at their respective downstream index points, or upstream of their respective pumping stations for Old Magpie Creek and Upper NEMDC, were used as the pattern hydrographs for the synthetic 8-Flood Series. For the Comprehensive Study, the basic pattern of all synthetic flood hydrographs was a 30-day hourly time series consisting of six waves, each 5 days in duration. Flood volumes were ranked and distributed into the basic pattern. The highest wave volume was distributed into the fourth, or main, wave. The second highest volume preceded the main wave. So, the two highest waves are in the middle ten days of the 30-day hydrograph. The upstream tributary index points used for the Comprehensive Study are listed on **Table 1**. They flow out of the mountains to the east, west, and north of the Sacramento Valley and have high flows during the rainy season. The Natomas tributaries flow out of the foothills or originate on the valley floor. Flows on these tributaries can be high during and immediately after a rainstorm. Without additional rainfall, the flows drop to base flow or to urban runoff levels. The average flows are a lot lower than for the Comp Study tributaries on **Table 1**. The Natomas tributary flows for the four smaller waves would be so minor, that zero runoff was assumed for the 30-day hydrographs except for the middle 10 days (Waves 3 and 4).

The 1 percent flood hydrograph for Dry Creek at Steelhead Creek was developed in the following way. The 5-day flood pattern hydrograph for 30 Dec 1996 to 3 Jan 1997 for Dry Creek at its downstream index point is shown on **Figure 11** and **Plate 13**. The 5-day flood volume for this pattern hydrograph is 17,400 acre-feet. The 5-day flood volume for the 1 percent flood for Dry Creek is 35,600 acre-feet. The ratio of the 1 percent event 5-day volume to the New Year 1997 5-day volume is 35,600 / 17,400 or 2.046. This ratio was applied to the hourly ordinates of the computed 5-day New Year 1997 hydrograph for Dry Creek at Steelhead Creek, to define the 1 percent flood hydrograph for Wave 4 at the Dry Creek index point. The difference between the 1 percent 5-day volume (35,600 ac-ft) for Dry Creek at Steelhead Creek index point and the 1 percent 10-day volume (43,900 ac-ft) for the Dry Creek index point is 8,300 acre-feet. The ratio of 8,300 ac-ft to the New Year 1997 5-day volume for Dry Creek at Steelhead Creek is 8,300 / 17,400, or 0.477. This ratio was applied to the New Year 1997 flood hydrograph at the Dry Creek index point, to define the hydrograph for Wave 3 of the 30-day 1 percent event flood hydrograph at the Dry Creek index point. **Figure 13** below shows the shape of the 30-day 1 percent event hydrograph for Dry Creek at Steelhead Creek, with zero flow for waves 1 – 2 and 5 – 6. Wave 4 is higher than Wave 3.

Figure 13



The rest of the floods in the 8-Flood Series for Dry Creek, as well as the hydrographs for the other eight Natomas tributaries, were developed using the same method. These hydrographs are consistent in shape and timing with the synthetic flood hydrographs for the Sacramento River tributary index points listed on **Table 1**.

The 30-day hydrographs for Upper NEMDC above the NEMDC Stormwater Pumping station and Old Magpie Creek above Pump 157 were routed through their respective pumping stations for each of the 8-Flood Series.

The Natomas tributary 30-day hydrographs for the 8-Flood Series were provided to Hydraulic Design Section for use as upstream boundary conditions for the hydraulic model. For Upper NEMDC and Old Magpie Creek, hydrographs for above and below their respective pumping stations were provided to Hydraulic Design Section.

4.0 Natomas Cross Canal (NCC) and Steelhead Creek (SHC) Coincident Frequency Study

The Comprehensive Study hydrology included coincident flood centerings for the Sacramento River tributaries large enough to have an influence on the flows downstream of their confluences with the mainstem. Flood hydrograph contributions from the tributary Natomas Cross Canal (NCC) and Steelhead Creek (SHC) are negligible in comparison with the mainstem flood flows, such that the tributary flow or stage hydrographs do not need to be considered when developing stage-frequency functions for the mainstem channels. However, the mainstem channel stages still need to be considered when developing stage-frequency functions on the tributaries. For this phase of the analysis, the Sacramento Mainstem flood series is used as the mainstem for the Natomas Cross Canal, and either the American River or the Sacramento Mainstem is used as the mainstem for the Steelhead Creek tributary, depending upon percent exceedence. For low mainstem stage conditions, Steelhead Creek flows directly to the Sacramento River rather than mingling flows with the American River.

4.1 Total Probability Theorem. Instead of the Comprehensive Study concurrent flood centering methodology, a total probability approach was used to evaluate coincident flood stages on the Natomas Cross Canal and Steelhead Creek. The procedure used was an extension of the Total Probability method documented in **Reference 11**, Procedures for Developing Stage-Probability Functions for Tributary Streams, prepared by David Ford Consulting Engineers (Ford) in February 2007.

Tangible benefit of a flood management project is computed, in part, as the expected value of inundation damage reduced. This computation requires a stage-frequency function at the location of interest. If that location is on a tributary stream, development of the function must account properly for the influence of the mainstem stream into which the tributary flows. A systematic, uniform approach is required for development of the stage-frequency functions for the locations of interest. The procedure begins with an assessment of the degree to which the tributary is dependent on the mainstem. An overview flowchart for the tributary analysis procedure is shown on **Plate 16**.

If the tributary is not dependent on mainstem conditions (Case 1), then the necessary information can be developed using typical riverine analyses: estimate the discharge for a specified probability, use that as the upstream boundary condition, and use a rating curve or similar control as the downstream boundary condition for the hydraulics model.

If tributary conditions are hydraulically dependent on mainstem conditions, can the frequency of the stage at the tributary location be predicted, given the mainstem conditions? If so (Case 3), then the Comprehensive Study methodology is used to develop the tributary flow-frequency function and the mainstem stage-frequency function. A channel model is developed for the reach of interest, and a resulting stage-frequency function is derived for the tributary index location.

If tributary conditions cannot be predicted reliably from mainstem conditions (Case 2), then combinations of boundary conditions are applied to the standard watershed and channel models. Using the results from analysis of tributary stages computed with varying downstream boundary conditions, the total probability equation is used to compute the desired stage-frequency function at the tributary location. The equation is:

$$F(stage_{tributary}) = \sum_{\substack{mainstem\\conditions}} (F(stage_{tributary} | stage_{mainstem}) \times F(stage_{mainstem}))$$

If a correlation exists between the tributary and mainstem, but is not definitive (Case 4), then a conditional probability analysis needs to be done. Practical methods to accomplish this have yet to be developed and field-tested.

4.2 Application to Natomas Tributaries. The coincident-frequency procedures that Ford used to develop stage-frequency curves for the Natomas Cross Canal and Steelhead Creek channels are described in the memorandum, "NCC/SHC Coincident Frequency Study: Exposition of Analytical Procedures," dated September 10, 2008, prepared by David Ford Consulting Engineers (**Reference 12**). Primary technical tasks include assessing hydrologic dependence between tributary and mainstem channels and identifying flow regimes where hydrologic independence may be presumed. A secondary task is identifying timing differences between tributary and mainstem peak stages. Total probability methodology relies on historical rainfall and streamflow data. Stage records from the California Data Exchange Center (CDEC, **Reference 13**) were used for the analysis. Due to the lack of stage data on the Natomas Cross Canal, CDEC stage records for the Dry Creek gage at Vernon Street (VRS) were substituted to develop a cross-correlation with the Sacramento River at Verona (VON) records. Records for the Sacramento River at I Street (IST) and at Ord Ferry (ORD) gages were used to supplement/correct the VON stage records. Similarly, due to the unavailability of long-term records for Steelhead Creek, Arcade Creek (AMC) records were cross-correlated with American River at H-Street gage (HST) records. American River at Fair Oaks (AFO) records were used to fill in missing values in the HST record. **Table 18** summarizes the primary stream gages used for this study. Gaging station locations (except for ORD) are shown on **Plate 1**.

Table 18
CDEC Gage Records Used for Hydrologic Dependence Analysis

Gage Name	CDEC gage ID	Period of Record
Sacramento River at Verona	VON	01Jan1984 – Present
Sacramento River at I Street	IST	01Jan1984 – Present
Sacramento River at Ord Ferry	ORD	01Jan1984 – Present
American River at H Street	HST	01Jan1984 – Present
American River at Fair Oaks	AFO	02Nov1998 – Present
Dry Creek at Vernon Street	VRS	19Oct1996 – Present
Arcade Creek at Winding Way	AMC	29Oct1996 – Present

The memorandum, "Cross-Correlation Analysis Results for NCC/SHC Coincident-Frequency Study," dated April 17, 2008, prepared by David Ford Consulting Engineers (**Reference 14**), describes the methods Ford used to assess conditions of hydrologic dependence between (1) Steelhead Creek and the American River, (2) Natomas Cross Canal and the

Sacramento River, and (3) the American River and the Sacramento River. It also identifies peak-stage timing differences between each tributary and the downstream mainstem channel.

Table 19 shows the tributary/mainstem confluence water surface elevations used as input in the Hydraulic Design Section's hydraulic models for the Natomas Cross Canal (NCC) and Steelhead Creek (SHC) tributaries as a function of mainstem annual exceedence probability (AEP) stages. Water surface elevation (WSEL) values are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29). Water surface elevations on SHC and NCC in **Table 19** correspond to stages on the American River and on the Sacramento River, respectively. For the more frequent mainsteam AEP between 0.50 and 0.04, Steelhead Creek stages are affected more by stages on the Sacramento River than by flows down the American River.

An analytical approach based on historical storm event data was used to characterize tributary/mainstem dependencies. Local event Annual Exceedence Probabilities (AEPs) were assigned to individual storm events, based on precipitation records from rainfall gages close to the SHC and NCC drainages. Rainfall frequency data was provided by Rainfall Depth-Duration Frequency Analysis for California Rain Gages (**Reference 15**), assembled by retired California State Climatologist Jim Goodridge. Historical mainstem peak flows were matched to concurrent local rainfall events on an event-by-event basis. Based on local storm magnitudes, the set of historic events was partitioned into return-frequency classes. Distributions for rarer AEP events were based on projected regional meteorologic patterns. Only rainfall and flow/stage records collected after 1980 were used for the analysis. It was assumed that n-year local flow event corresponded to the n-year local rainfall event, and that mainstem/tributary conditional distribution patterns can be extrapolated for rarer events using general knowledge of regional storm patterns and local channel hydraulics.

Table 19

Applied Stage-Frequency Functions for Mainstem AEP Events

Mainstem-event AEP	Steelhead Creek (SHC) Downstream WSEL (ft. NGVD29)	Natomas Cross Canal (NCC) Downstream WSEL (ft. NGVD29)
0.500	24.09	33.08
0.200	24.80	35.10
0.010	25.70	36.34
0.040	30.71	39.34
0.020	32.65*	40.10
0.010	35.43*	41.62
0.005	37.18*	43.00
0.002	42.62*	44.35

Notes:

AEP = Annual Exceedence Probability

WSEL = Water Surface Elevation

* WSEL is stage for American River conditions. All other WSELs are stages on the Sacramento River Mainstem.

The Hydraulic Design models were used to generate peak water surface elevations for the SHC and NCC index points for various combinations of tributary discharge and fixed mainstem stage (per **Table 19**). The tributary discharge rates were characterized by local-event AEP; similarly, the downstream confluence stages were characterized by mainstem AEP. The computed NCC and SHC index point stage values corresponded to regulated mainstem conditions.

4.3 <u>Computational Results</u>. Ford developed stage-frequency functions for the Natomas Cross Canal and Steelhead Creek index points. **Table 20** presents the stage-frequency functions for the NCC and SHC index points based on Ford's coincident-frequency evaluation. The stage values were computed under regulated mainstem conditions. Water surface elevation (WSEL) values are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

Table 20

Computed Stage-Frequency Functions for Local AEP Events

Local-event AEP	Steelhead Creek (SHC) Index Point WSEL (ft. NGVD29)	Natomas Cross Canal (NCC) Index Point WSEL (ft. NGVD29)
0.500	26.3	33.9
0.200	28.6	34.5
0.010	29.9	34.8
0.040	31.4	36.6
0.020	33.4	37.8
0.010	35.5	38.6
0.005	37.4	40.1
0.002	40.1	42.4

Notes:
AEP = Annual Exceedence Probability
WSEL = Water Surface Elevation
SHC index point is located at RM 3.713
NCC index point is located at RM 4.323

Stages listed in **Table 20** are based on UNET modeling, not on the latest HEC-RAS model. The above stages may change when the HEC-RAS model is used for the analyses. The memorandum, "NCC/SHC Coincident Frequency Study: Computational Results," dated September 10, 2008 prepared by Ford (**Reference 16**), provides additional details regarding the results in **Table 20** from the analyses - the special factors considered, the hydraulic profiles and probabilistic relations used in the computations, and the coincident stage-frequency functions.

Table 21 shows the combination of which mainstem flood hydrographs are being used in combination with which Natomas tributary flood hydrographs in the HEC- RAS hydraulic model. These flood hydrograph combinations are being used in preparation for the F3 Conference Milestone. Different combinations of floods may be tested for later analysis.

Preliminary analysis determined that, for the mouth of the Natomas Cross Canal, the flood stages for the Sacramento Mainstem and Shanghai-Yuba centerings were similar. So the Shanghai-Yuba flood series hydrographs are not being used in the current phase (pre-F3 Milestone) of the analysis, but will be tested later.

Table 21
Flood Hydrograph Combinations used in HEC-RAS Hydraulic Model
for Current Phase of Analysis

Sacramento Mainstem Flood-event AEP	Steelhead Creek Flood-event AEP	Natomas Cross Canal Flood-event AEP
0.500	0.500	0.500
0.200	0.500	0.500
0.010	0.200	0.200
0.040	0.010	0.010
0.020	0.040	0.040
0.010	0.020	0.020
0.005	0.010	0.010
0.002	0.005	0.005
American River Flood-event AEP	Steelhead Creek Flood-event AEP	Natomas Cross Canal Flood-event AEP
0.500	0.500	0.500
0.200	0.500	0.500
0.010	0.200	0.200
0.040	0.010	0.010
0.020	0.040	0.040
0.010	0.020	0.020
0.005	0.010	0.010
0.002	0.005	0.005

Notes: AEP = Annual Exceedence Probability

5.0 List of References

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2. Yuba River Basin Project General Reevaluation Report, Appendix A, Synthetic Hydrology & Reservoir Operations Technical Documentation. U.S. Army Corps of Engineers, Sacramento District. Aug 2004, corrected June 2008.

3. Folsom Dam and Lake Revised PMF Study, American River Basin, California, Hydrology Office Report. U.S. Army Corps of Engineers, Sacramento District. October 2001.

4. Hydrometeorological Report No. 59, Probable Maximum Precipitation for California. U.S. Department of Commerce, National Oceanic and Stmospheric administration. U.S. Army Corps of Engineers. February 1999.

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6. Natomas General Reevaluation Report, Natomas Cross Canal and Steelhead Creek Watersheds, Placer, Sacramento, and Sutter Counties, California, Hydrology Appendix. U.S. Army Corps of Engineers, Sacramento District. October 2006.

7. Dry Creek, Placer and Sacramento Counties, California, Hydrology Office Report. U.S. Army Corps of Engineers, Sacramento District. July 1984, revised April 1988

8. "Use of Radar-Rainfall Estimates to Model the January 9-10, 1995 Floods in Sacramento, CA," by David C. Curtis, Ph.D., NEXRAIN Corporation, and John H. Humphrey, Ph.D., P.E., C.C.M., HYDMET, Inc. Paper presented October 1995

9. Sacramento County ALERT System gage map: <http://www.sacflood.org/alrtloc1.htm>.

10. City of Roseville Public Works Web page with access to maps with local gaging station locations: <http://www.roseville.ca.us/pw/engineering/floodplain_management/roseville_current_stream_levels/default.asp>.

11. Procedures for Developing Stage-Probability Functions for Tributary Streams. Prepared by David Ford Consulting Engineers, Inc. February 26, 2007.

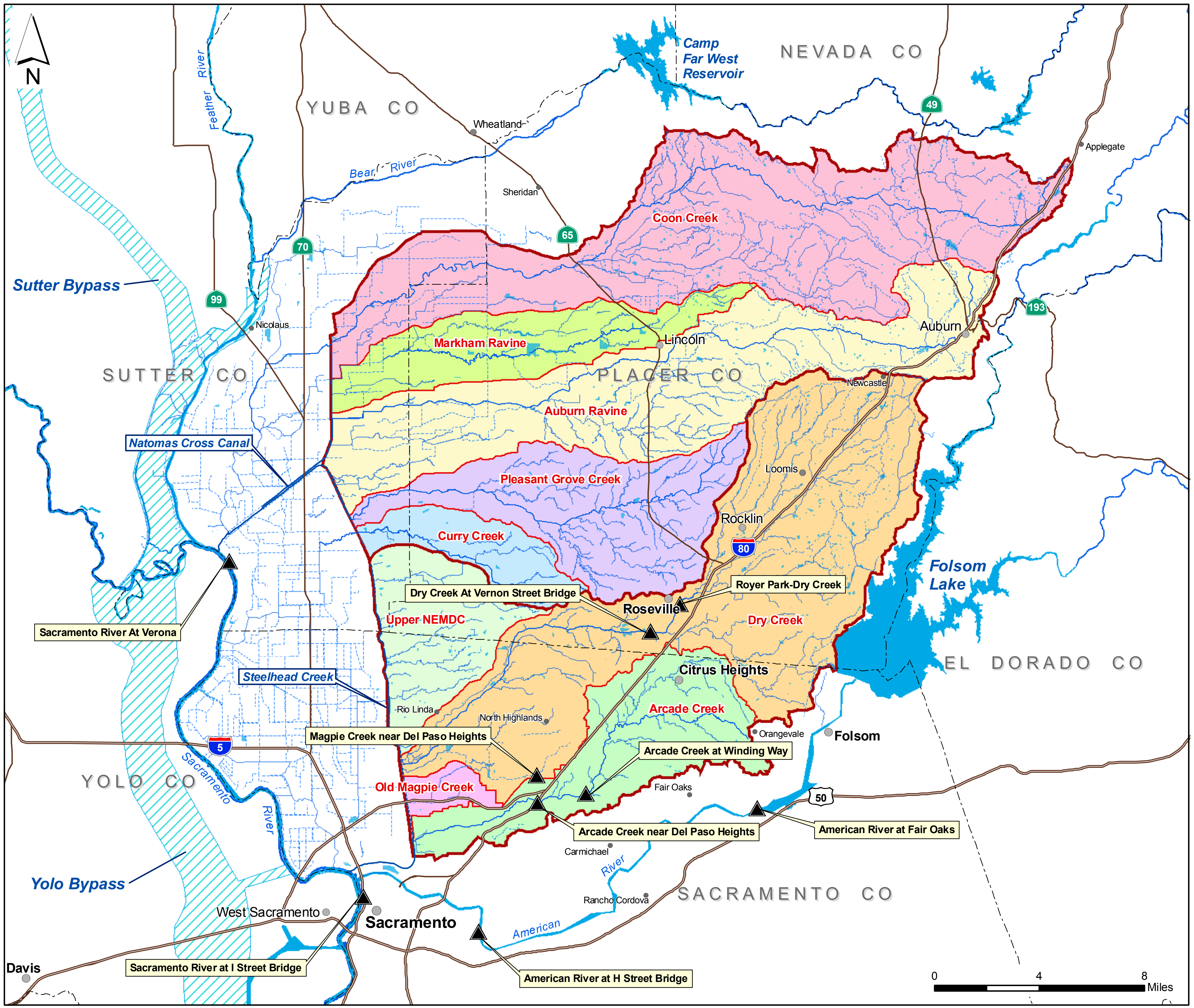
12. Memorandum: NCC/SHC Coincident-Frequency Study: Exposition of Analytical Procedures. Prepared by David Ford Consulting Engineers, Inc. September 10, 2008.

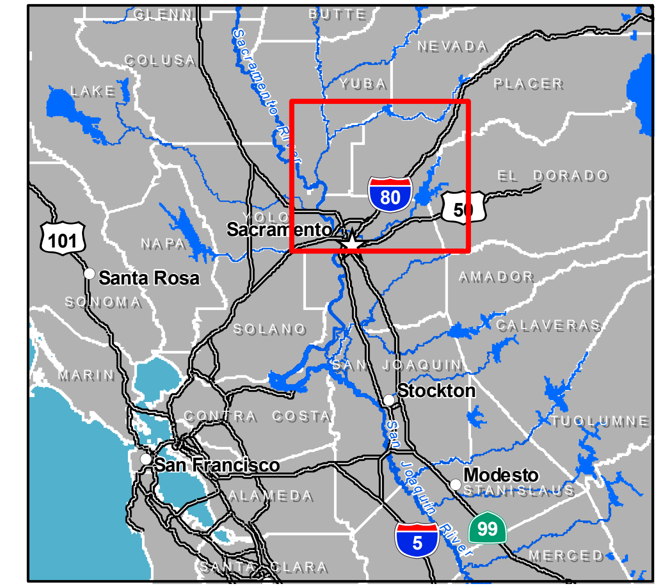
13. California Data Exchange Center (CDEC), the access point to the California Department of Water Resources' operation hydrologic data at: <http://cdec.water.ca.gov>.

14. Memorandum: Cross-Correlation Analysis Results for NCC/SHC Coincident-Frequency Study. Prepared by David Ford Consulting Engineers, Inc. April 17, 2008.

15. Rainfall Depth-Duration Frequency Analysis for California Rain Gages. Mr. James Goodridge, retired State of California climatologist. Revised 2005.

16. Memorandum: NCC/SCH Coincident Frequency Study: Computational Results. Prepared by David Ford Consulting Engineers, Inc. September 10, 2008.





Legend

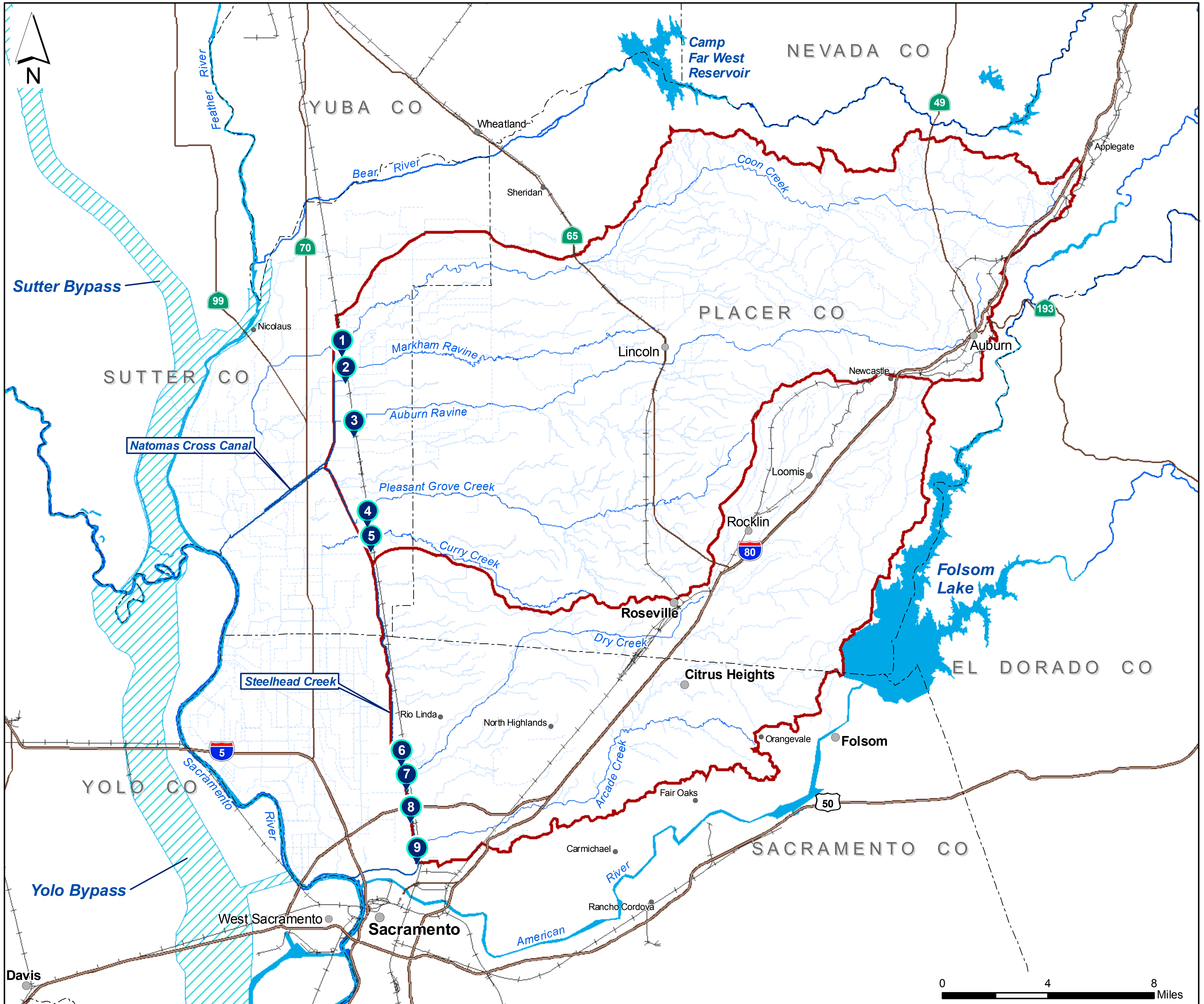
- Stream Gage
- Watershed
- Sub-Watershed
- Lake or Major River
- River or Stream
- Canal or Ditch
- Freeway
- Highway
- County Boundary
- City or Town

American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

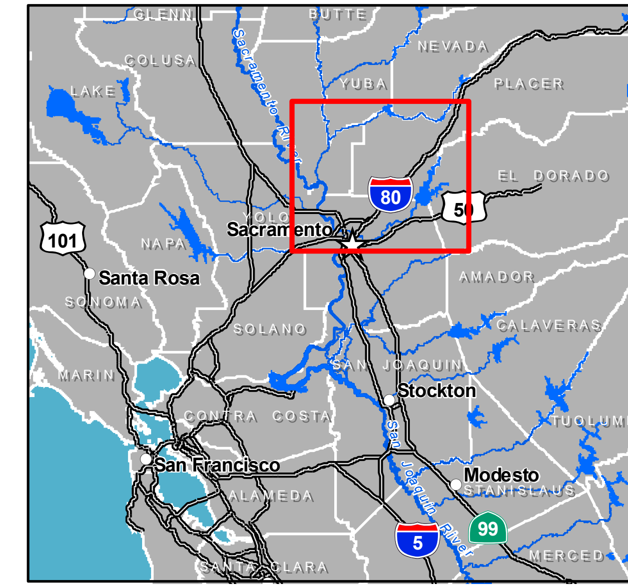
GENERAL MAP

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008 PLATE 1



Prepared by J.S.M



Legend

- Watershed
- Lake or Major River
- River or Stream
- Canal or Ditch
- City or Town
- Freeway
- Highway
- Railroad
- County Boundary

Input Sites

1. Coon Creek
2. Markham Ravine
3. Auburn Ravine
4. Pleasant Grove Creek
5. Curry Creek
6. Upper NEMDC Tribs
7. Dry Creek
8. Old Magpie Creek
9. Arcade Creek

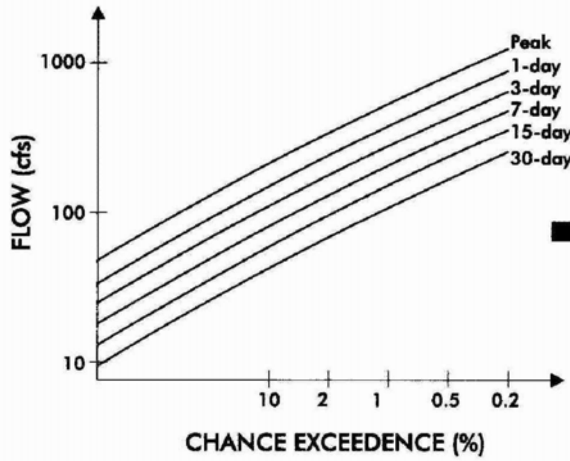
American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

NATOMAS TRIBUTARY INPUT LOCATIONS FOR HYDRAULIC MODEL

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

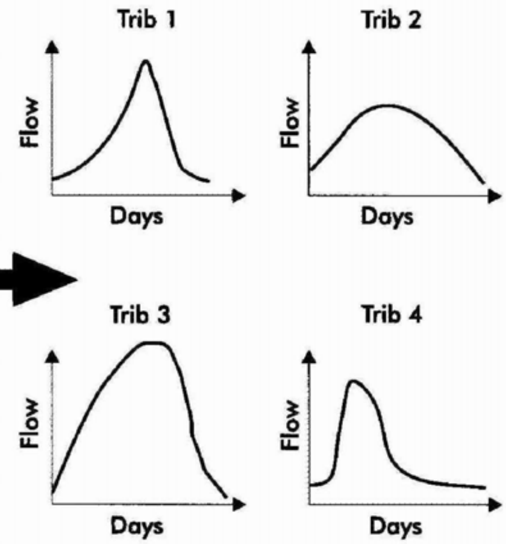
SEP 2008 PLATE 2

Full Natural Flow



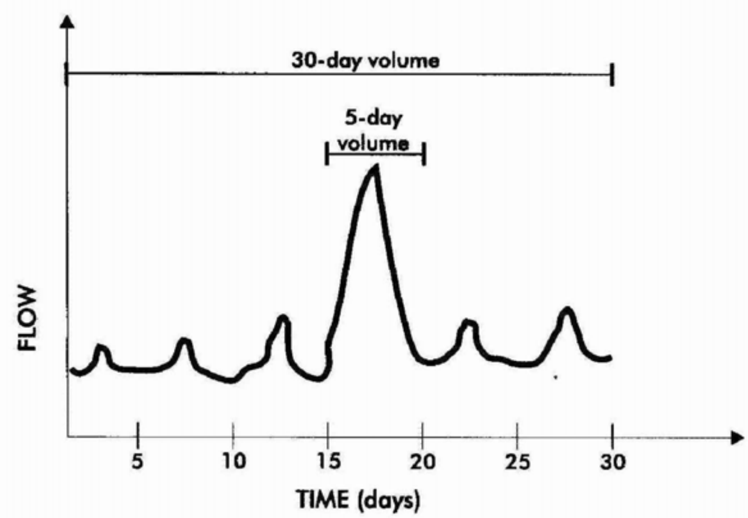
Full natural flow volumes were developed for each of 7 synthetic exceedence events on each tributary

Tributary-Specific Patterns



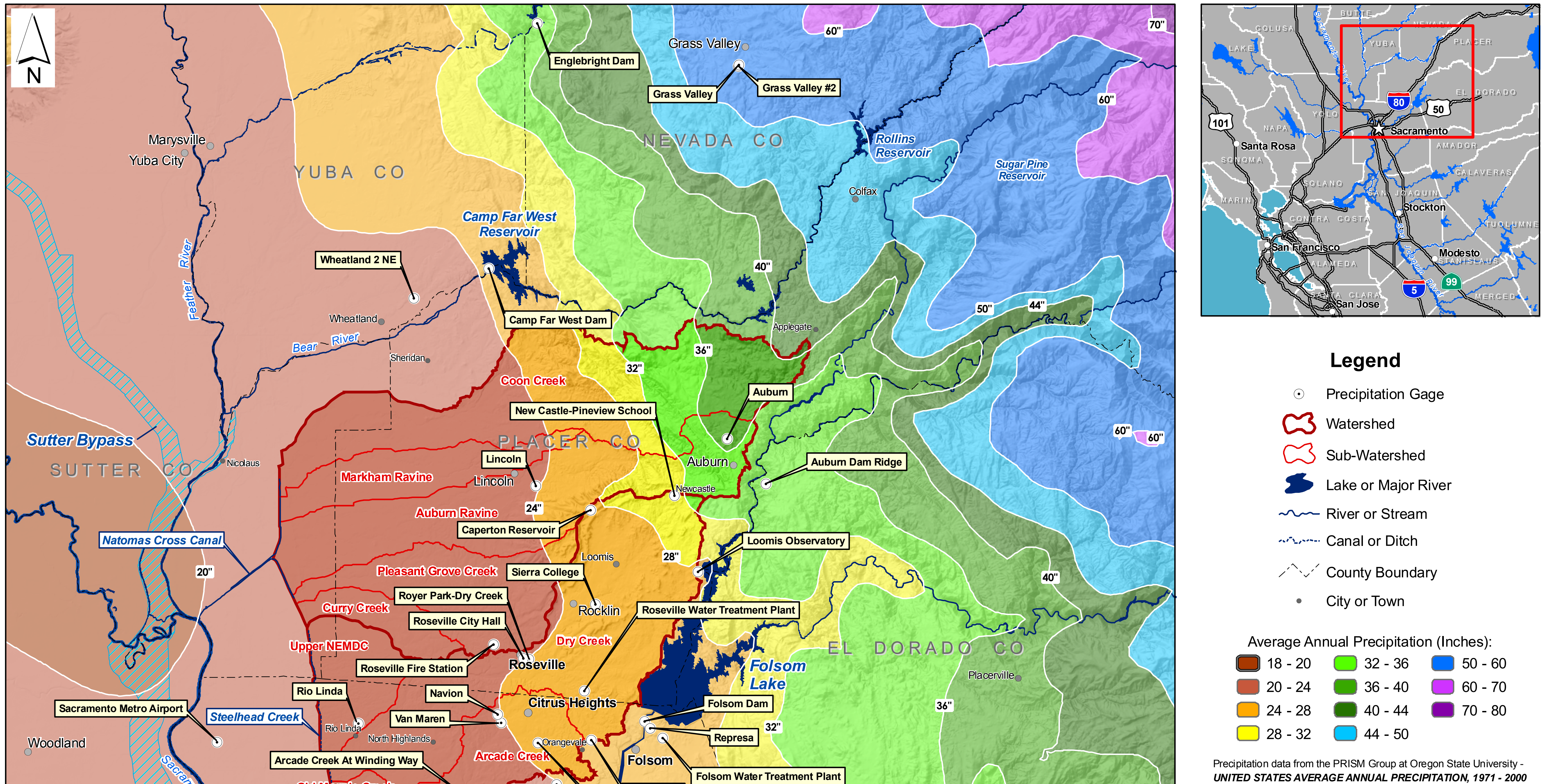
Tributary-specific hydrograph patterns were used to translate 5-day incremental volumes to flow patterns

Flow Patterns were combined to develop a 30-day period



Resulting 30-day hydrographs for each of the 7 exceedence events on each tributary

Sacramento & San Joaquin River Basins Comprehensive Study
HYDROGRAPH CONSTRUCTION
U.S. Army Corps of Engineers Reclamation Board, State of California July 2002



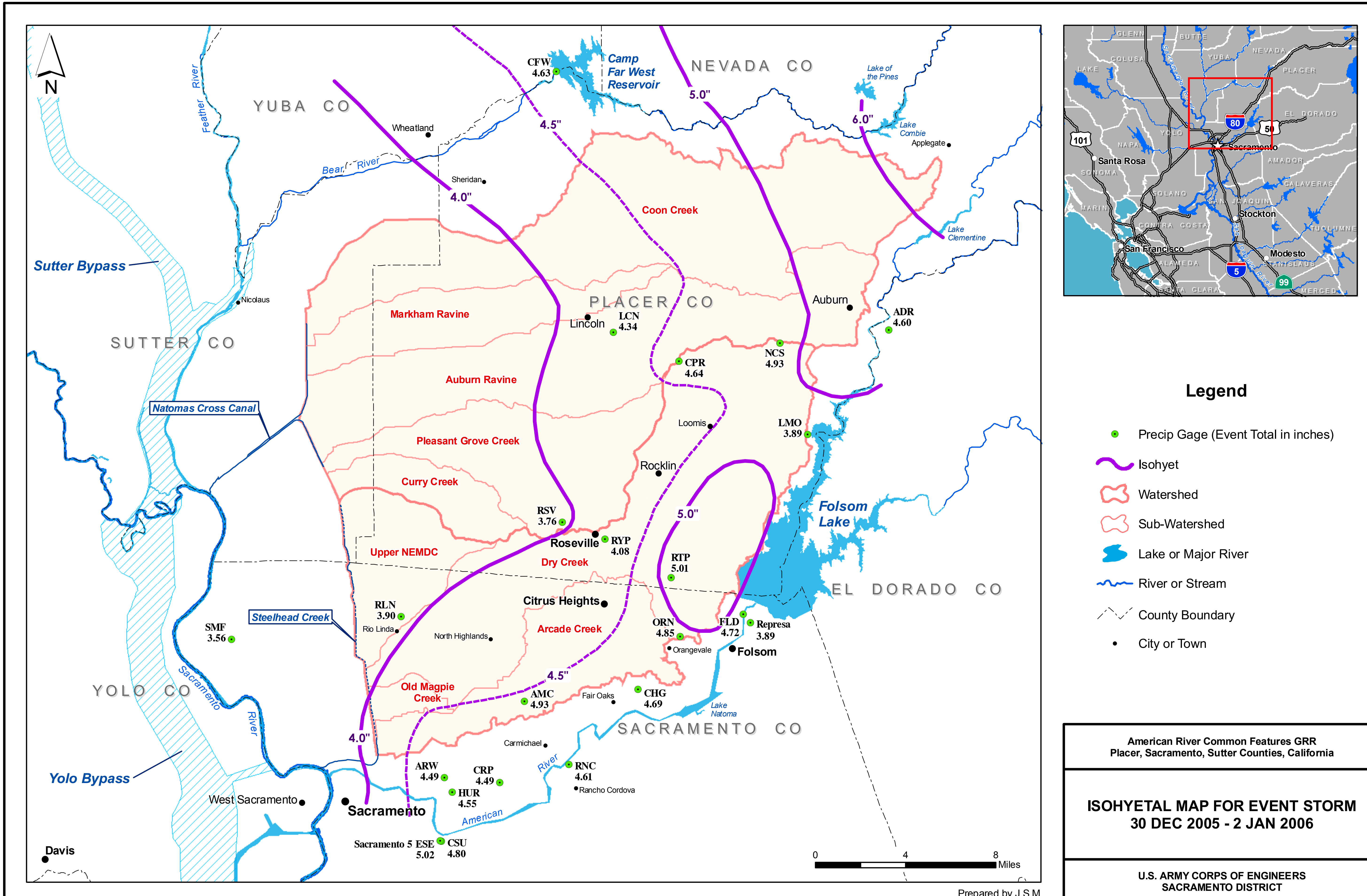
Prepared by J.S.M

American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

AVERAGE ANNUAL PRECIPITATION

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008



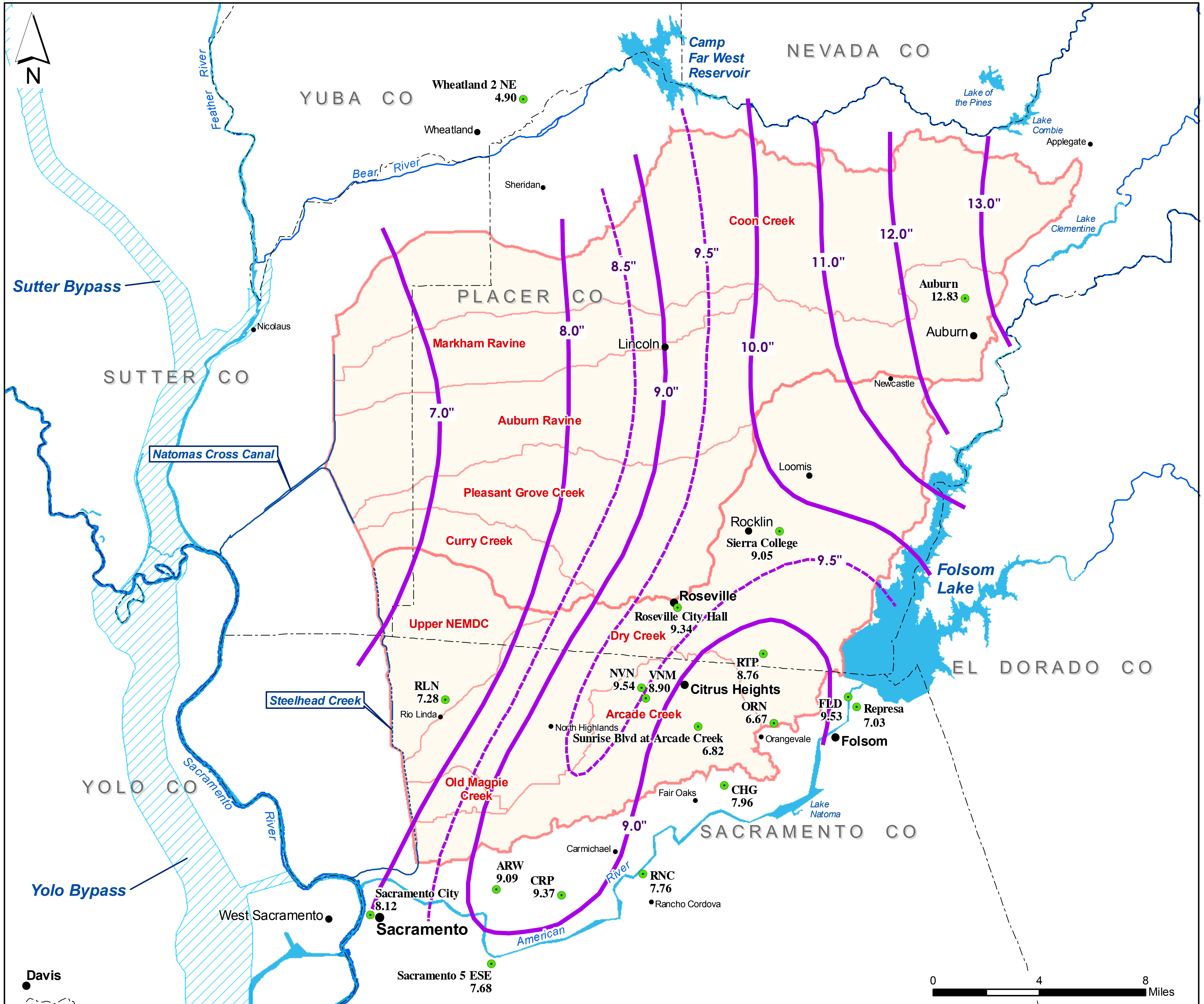
American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

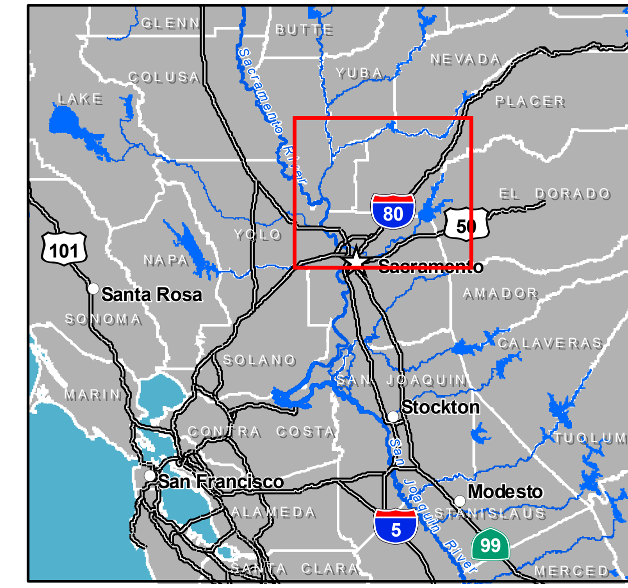
ISOHYETAL MAP FOR EVENT STORM 30 DEC 2005 - 2 JAN 2006

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

PLATE 5





Legend

- Precip Gage (Event Total in inches)
- Isohyet
- Watershed
- Sub-Watershed
- Lake or Major River
- River or Stream
- County Boundary
- City or Town

Prepared by J.S.M

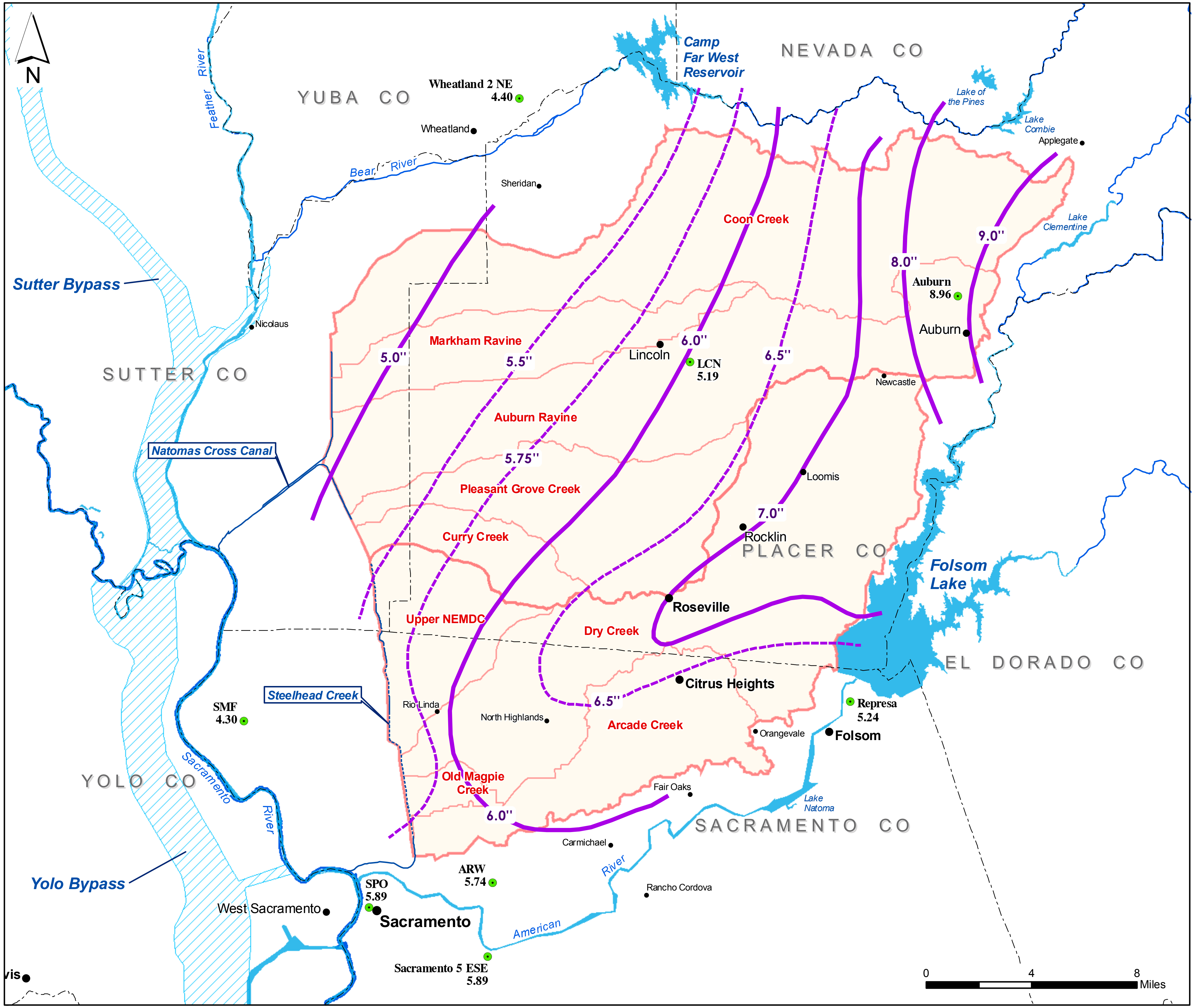
American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

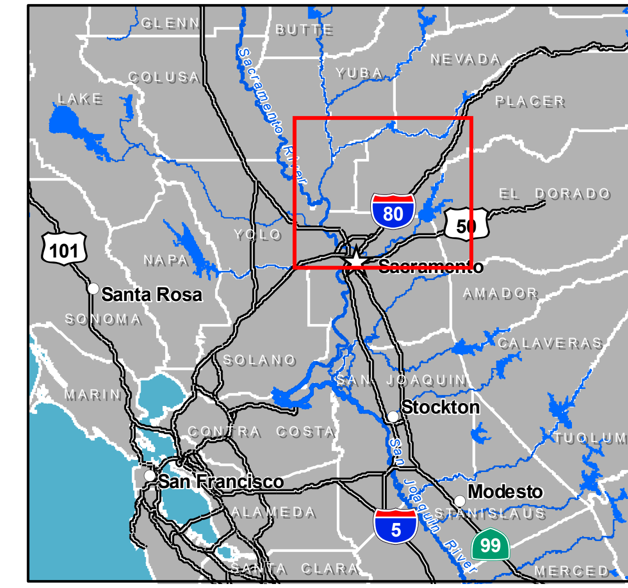
ISOHYETAL MAP FOR EVENT STORM
15 FEB - 19 FEB 1986

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

PLATE 6



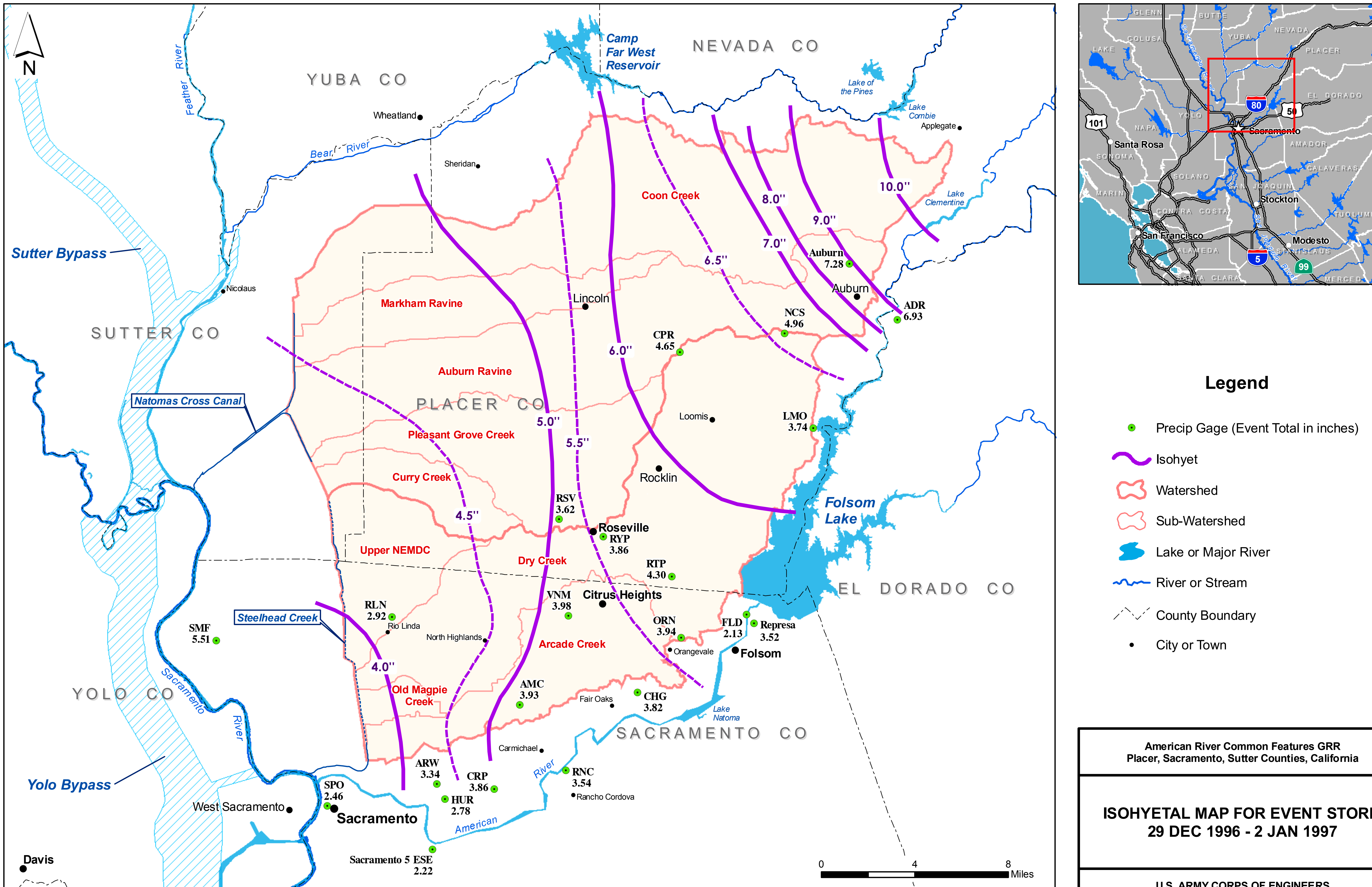


American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

ISOHYETAL MAP FOR EVENT STORM
8 JAN - 12 JAN 1995

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008 PLATE 7

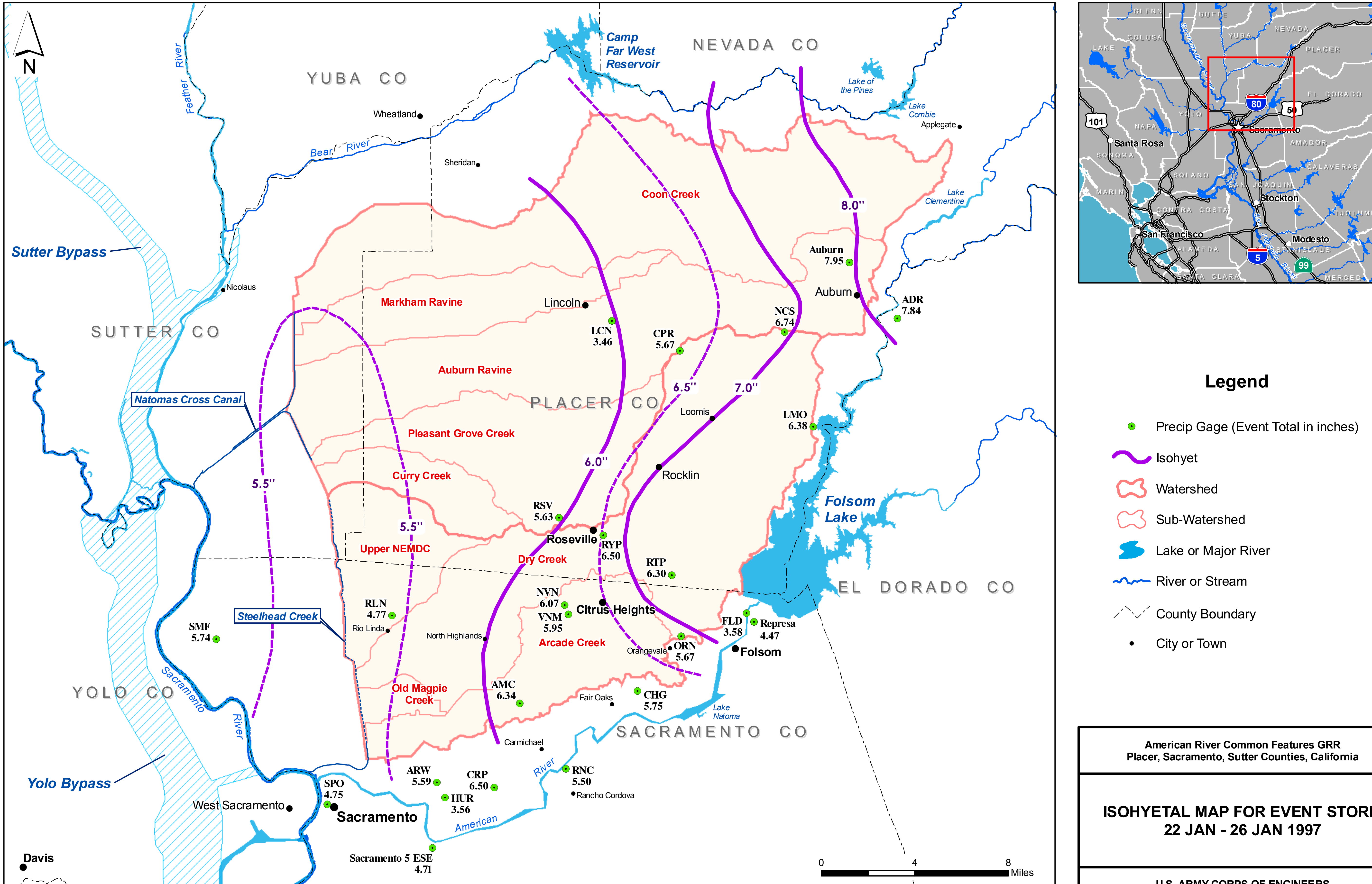


American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

ISOHYETAL MAP FOR EVENT STORM
29 DEC 1996 - 2 JAN 1997

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008 PLATE 8



Legend

- Precip Gage (Event Total in inches)
- Isohyet
- Watershed
- Sub-Watershed
- Lake or Major River
- River or Stream
- County Boundary
- City or Town

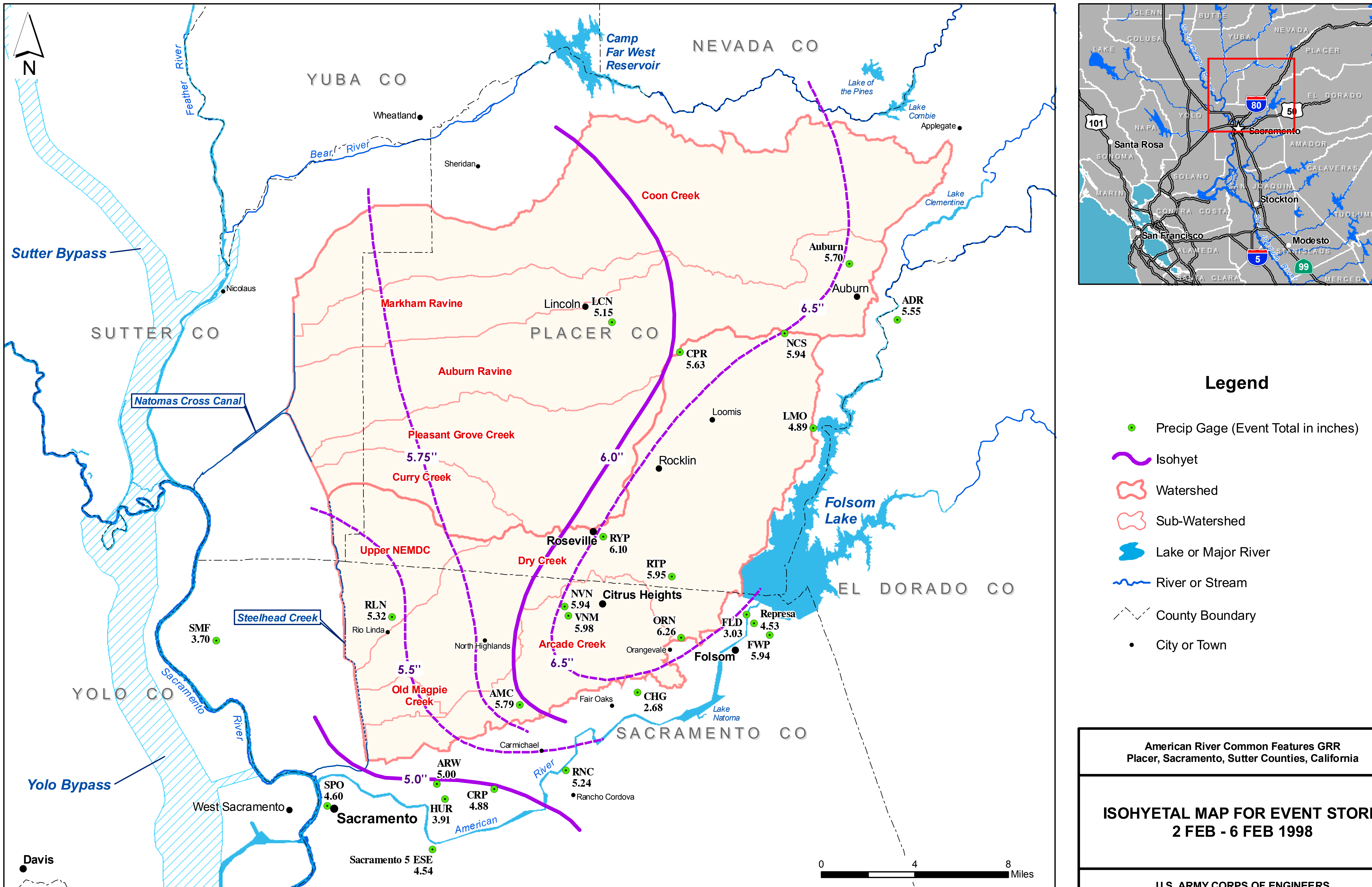
Prepared by J.S.M

American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

ISOHYETAL MAP FOR EVENT STORM 22 JAN - 26 JAN 1997

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

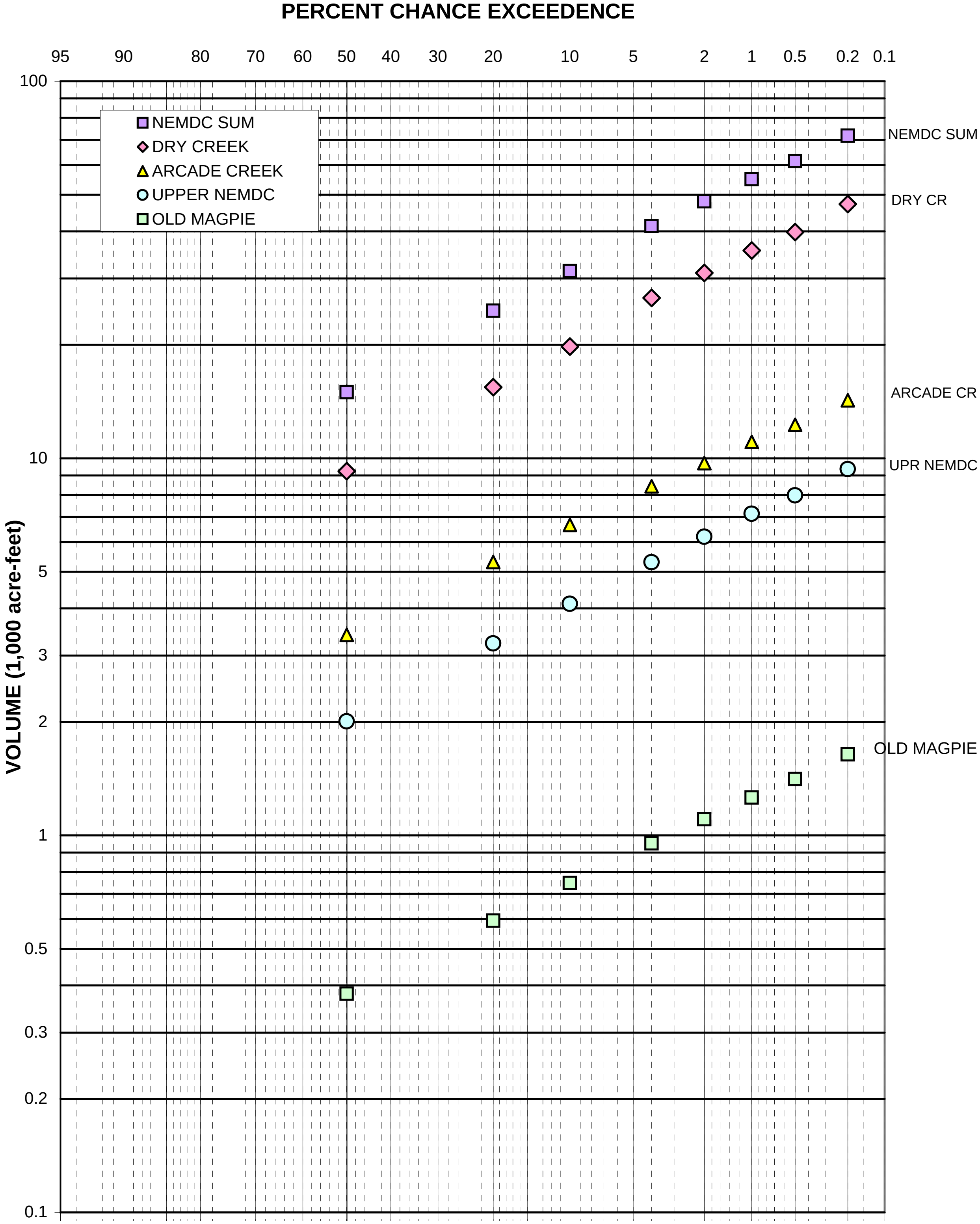


American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

ISOHYETAL MAP FOR EVENT STORM
2 FEB - 6 FEB 1998

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008 PLATE 10



NOTES:

Dry Creek 116.48 sq. mi.

Arcade Creek 40.14 sq. mi.

Upper NEMDC 27.13 sq. mi.

Old Magpie Sum 4.57 sq. mi.

NEMDC Sum 188.32 sq. mi.

Developed by LLW and prepared by JLB

AMERICAN RIVER COMMON FEATURES GRR

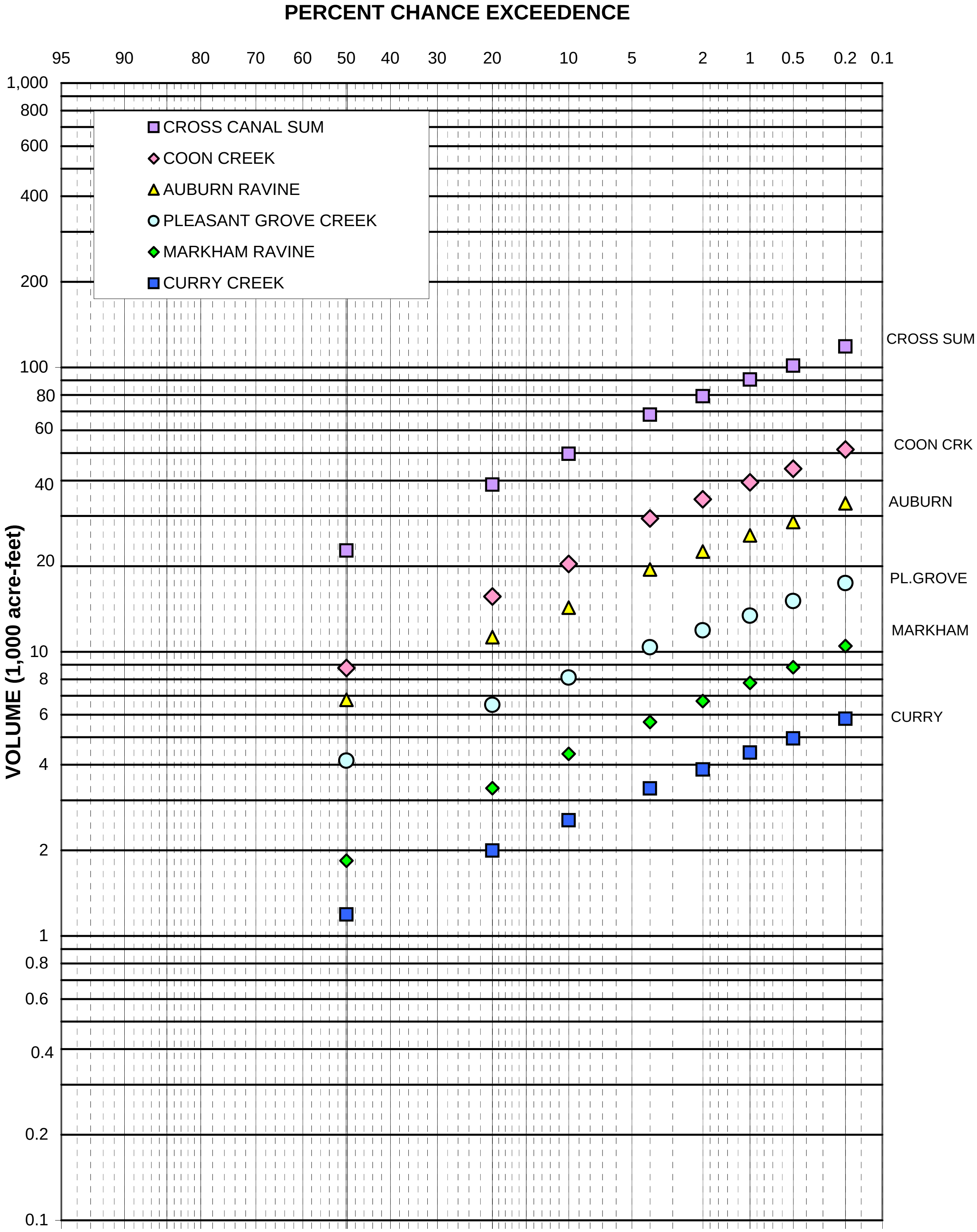
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

5-DAY VOLUME FREQUENCY CURVES
STEELHEAD CREEK DRAINAGE

U.S. ARMY CORPS OF ENGINEERS

SACRAMENTO DISTRICT

SEP 2008



NOTES:

Coon Creek 112.61 sq. mi.

Auburn Ravine 79.97 sq. mi.

Pleasant Grove Creek 46.69 sq. mi.

Markham Ravine 32.36 sq. mi.

Curry Creek 16.59 sq. mi.

Cross Canal Sum 288.22 sq. mi.

Developed by LLW and prepared by JLB

AMERICAN RIVER COMMON FEATURES GRR

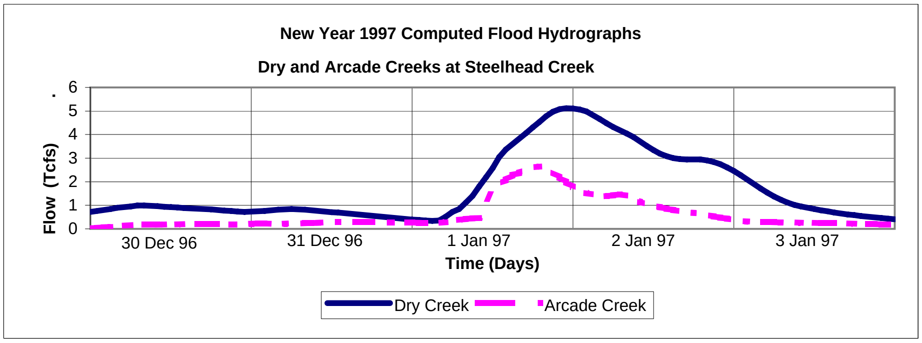
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

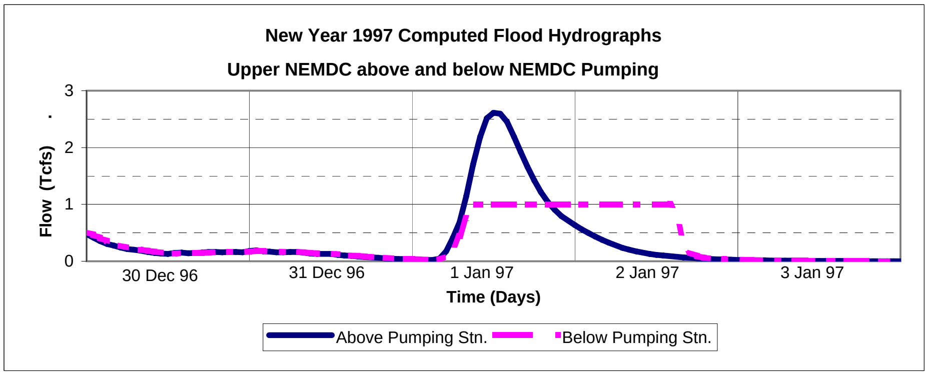
5-DAY VOLUME FREQUENCY CURVES CROSS CANAL TRIBUTARIES

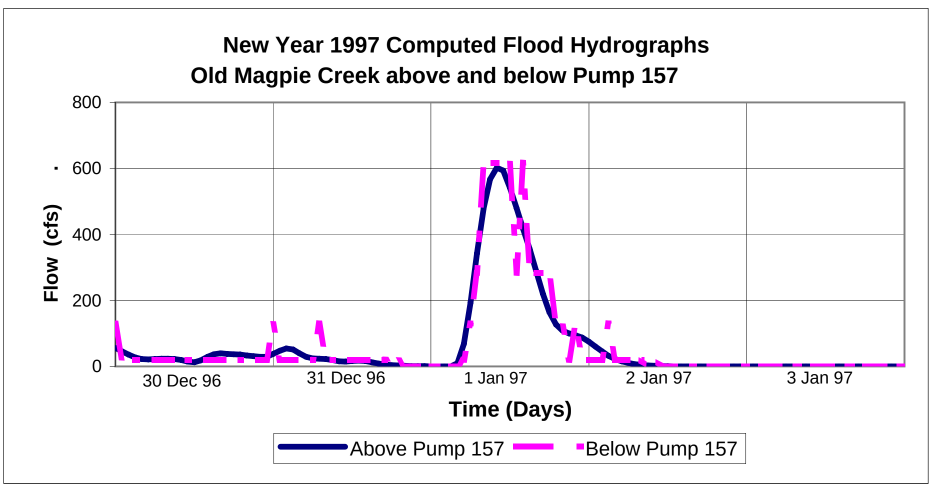
U.S. ARMY CORPS OF ENGINEERS

SACRAMENTO DISTRICT

SEP 2008



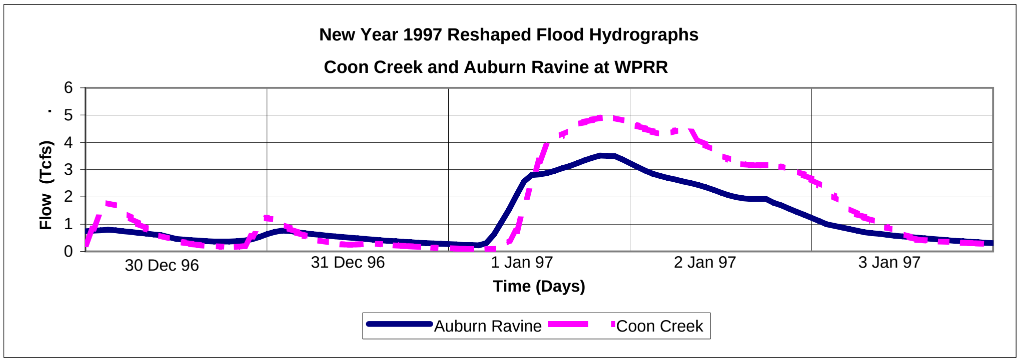


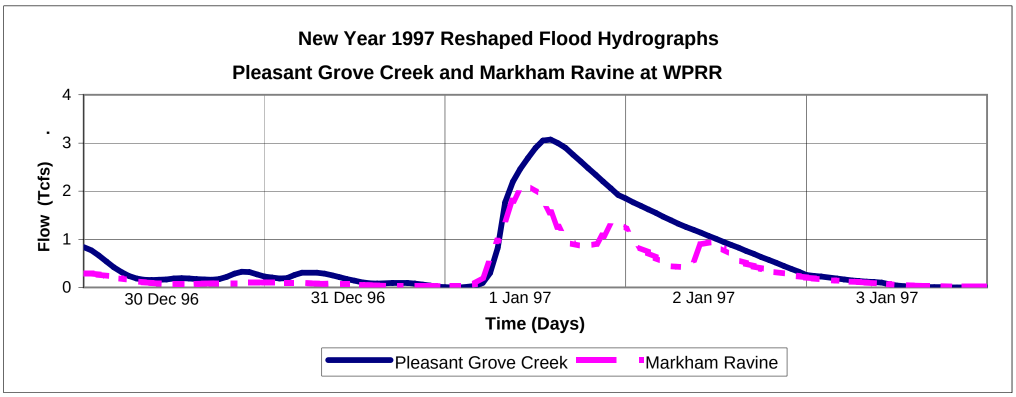


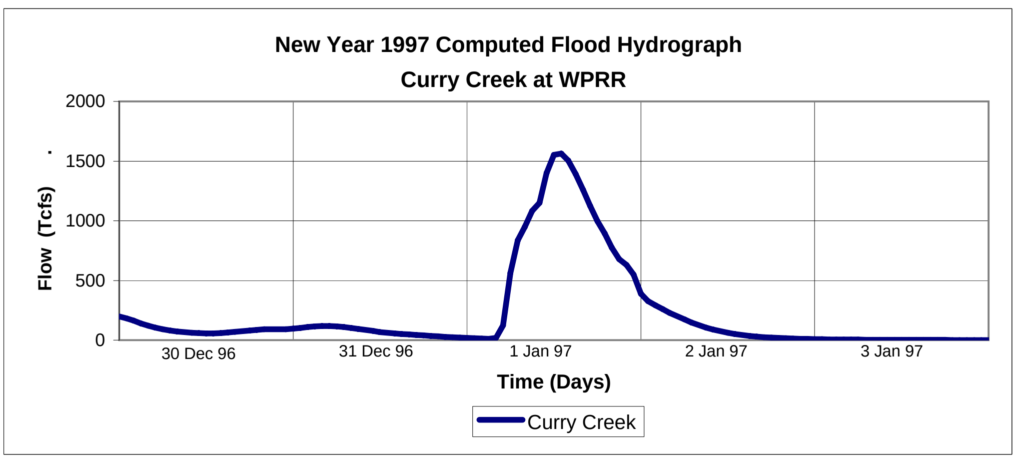
AMERICAN RIVER COMMON FEATURES GRR
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

COMPUTED AND RESHAPED HYDROGRAPHS FOR FOR NEW YEAR 1997 FLOOD EVENT STEELHEAD CREEK TRIBUTARIES

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



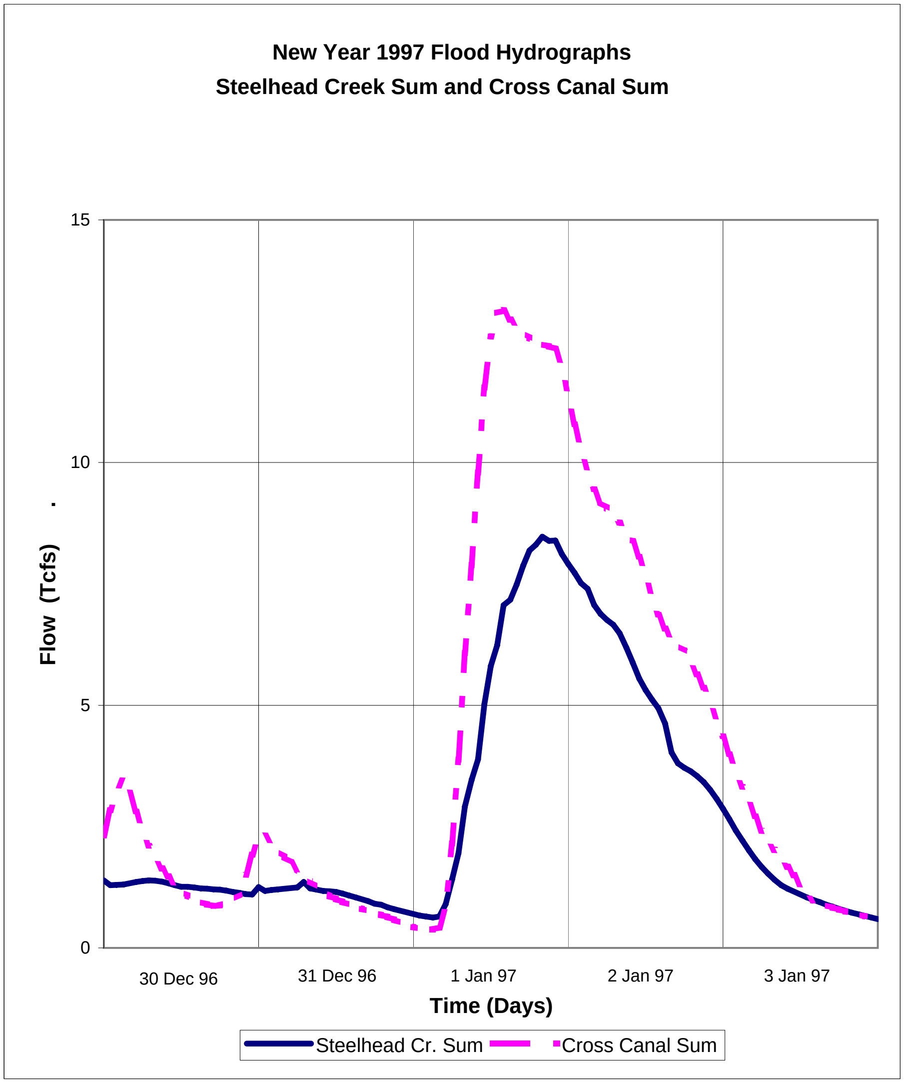




AMERICAN RIVER COMMON FEATURES GRR
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

COMPUTED AND RESHAPED HYDROGRAPHS FOR FOR NEW YEAR 1997 FLOOD EVENT NATOMAS CROSS CANAL TRIBUTARIES

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

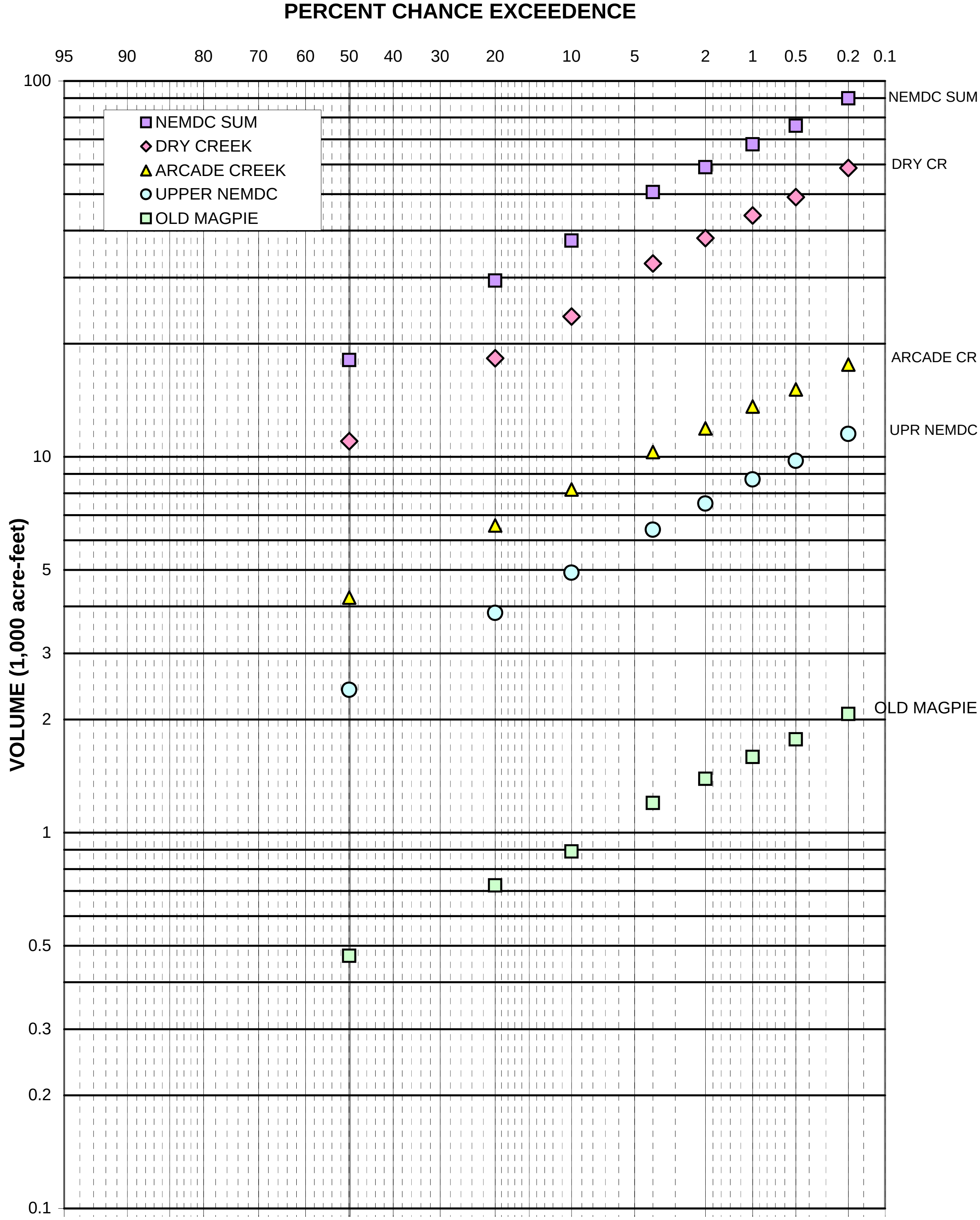


AMERICAN RIVER COMMON FEATURES GRR
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

COMPUTED AND RESHAPED HYDROGRAPHS FOR FOR NEW YEAR 1997 FLOOD EVENT STEELHEAD CREEK SUM NATOMAS CROSS CANAL SUM

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008



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Upper NEMDC 27.13 sq. mi.

Old Magpie Sum 4.57 sq. mi.

NEMDC Sum 188.32 sq. mi.

AMERICAN RIVER COMMON FEATURES GRR

PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

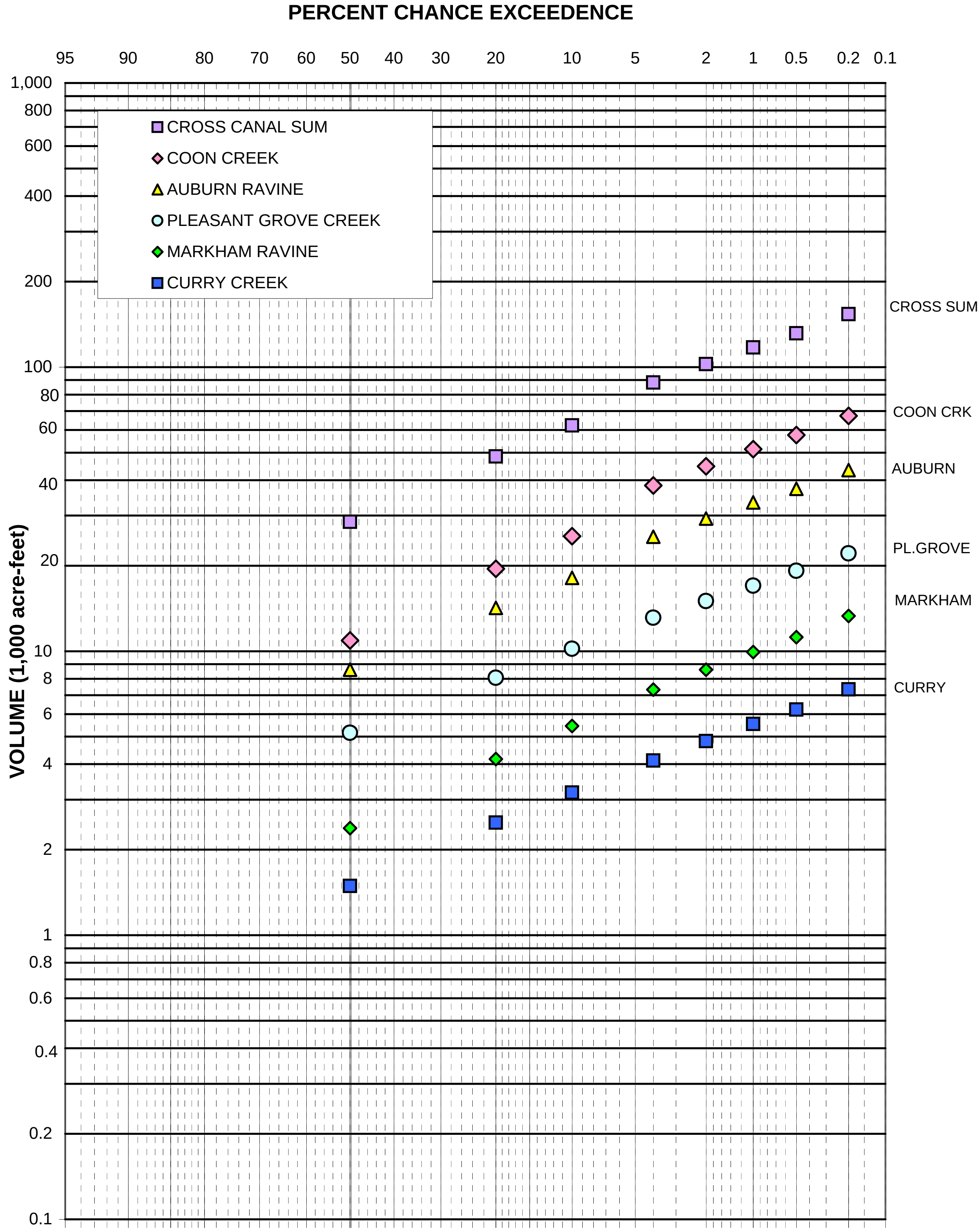
10-DAY VOLUME FREQUENCY CURVES STEELHEAD CREEK DRAINAGE

U.S. ARMY CORPS OF ENGINEERS

SACRAMENTO DISTRICT

Developed by LLW and prepared by JLB

SEP 2008



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Pleasant Grove Creek 46.69 sq. mi.
Markham Ravine 32.36 sq. mi.
Curry Creek 16.59 sq. mi.
Cross Canal Sum 288.22 sq. mi.

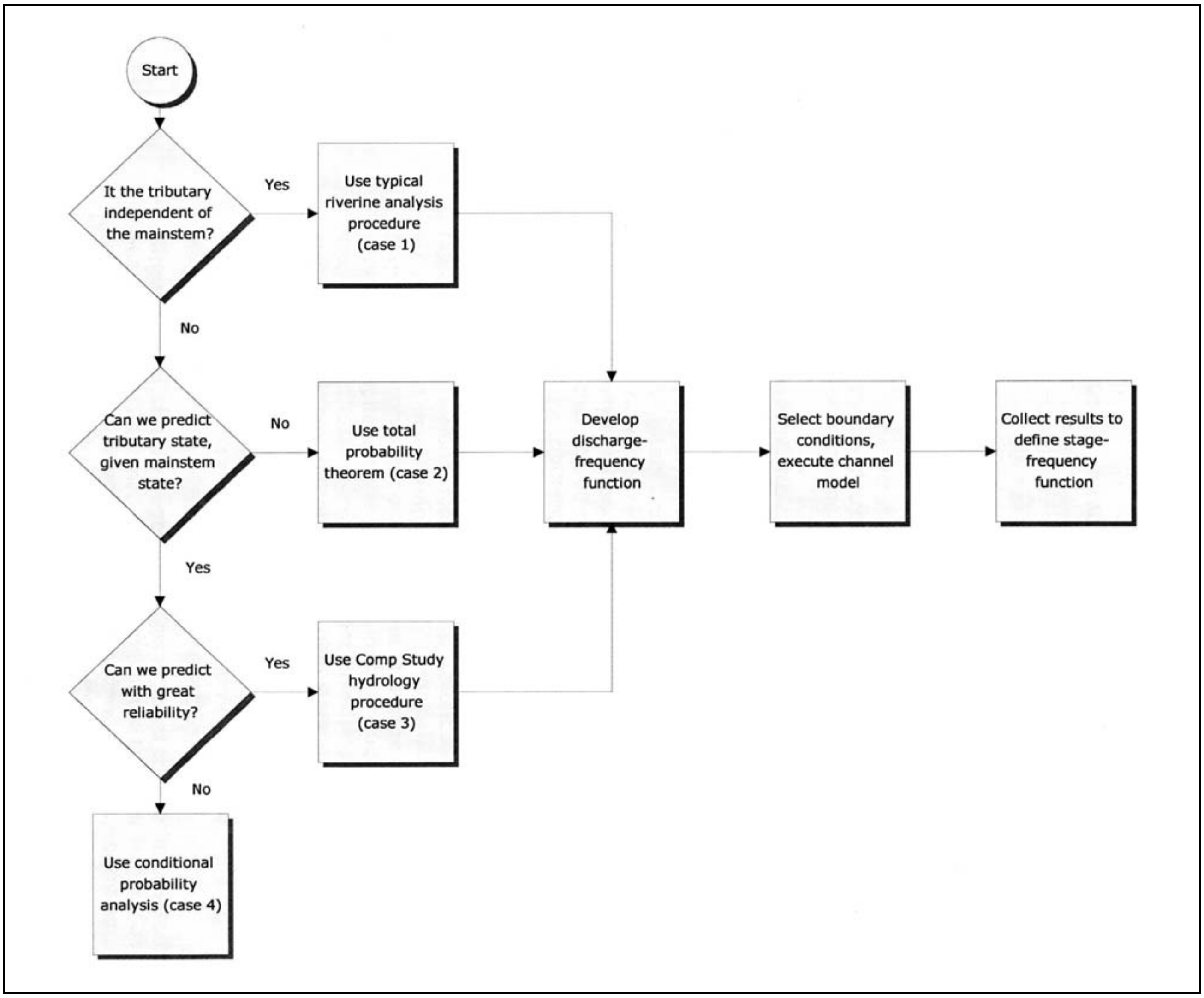
Developed by LLW and prepared by JLB

AMERICAN RIVER COMMON FEATURES GRR
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

10-DAY VOLUME FREQUENCY CURVES
CROSS CANAL TRIBUTARIES

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008



Overview Flowchart for Tributary Analysis Procedure

American River Watershed
Common Features Project
Natomas Post-Authorization Change Report

American River Hydrology
&
Folsom Dam Reservoir Operations

APPENDIX B2

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DRAFT
TABLE OF CONTENTS

FIGURES

TABLES

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AMERICAN RIVER HYDROLOGY & FOLSOM DAM RESERVOIR OPERATIONS

A-1 Purpose

The scope of this General Reevaluation Report (GRR) covers the greater Sacramento area, which includes the Lower American River and the Natomas Basin. Hydraulic and geotechnical studies of the area have been on-going and have already identified many issues (e.g. seepage, erosion, vegetation, etc) which could lead to levee failure. The latest findings indicate that the Sacramento area is still highly susceptible to flooding due to levee failure even with all the authorized repairs and improvements. The economic analyses will evaluate the flood risk and cost benefit of fixing the identified problems. This write-up covers the development of the Folsom Dam discharge hydrographs provided to Hydraulic Design for the floodplain delineation efforts and the development of the hydrologic data inputs provided to Economics for the HEC-FDA model. The economic analysis will evaluate the extent of the damage caused by levee failures within the basin. Two scenarios were evaluated for the existing condition: the without-project (WO) condition and the future without-project condition, which is labeled as the no-action (NA) condition. These scenarios provide the information needed to perform an incremental analysis of the state of the levees at various levels of improvement (objective release 115,000 cfs, 145,000 cfs, or 160,000 cfs) and of the affect of the levee state when combined with the other authorized project components. Generally, these scenarios are hypothetical and would not be built or implemented as stand-alone projects. The reservoir routings covered herein were developed for planning purposes, only. All reservoir elevations provided herein use the NGVD29 vertical datum.

A-2 Background

As an interim means of reducing flood risk, Congress authorized the American River Common Features Project under Section 101(a) (1) of the Water Resources Development Act (WRDA) 1996. The features that were common to three candidate plans identified by the Corps, SAFCA, and the State of California Reclamation Board (State Reclamation Board) in the 1996 Supplemental Information Report (SIR) were covered in the authorization. The levee repairs and improvements included:

- 24 miles of slurry wall in the levees along the lower American River
- 12 miles of levee modifications along the east bank of the Sacramento River downstream from the Natomas Cross Canal
- Installation of three telemeter streamflow gages upstream from the Folsom Reservoir
- Modification to the flood warning system along the lower American River
- Raising the left bank of the non-Federal levee upstream of Mayhew Drain for a distance of 4,500 feet by an average of 2.5 feet
- Raising the right bank of the American River levee from 1,500 feet upstream to 4,000 feet downstream of the Howe Avenue Bridge by an average of 1 foot
- Modifying the south levee of the Natomas Cross Canal for a distance of 5 miles to ensure that the south levee is consistent in level with the level of protection provided by the authorized levee along the east bank of the Sacramento River
- Modifying the north levee of the Natomas Cross Canal for a distance of 5 miles to ensure the height of the levee is equivalent to the height of the south levee as authorized (above)
- Installing gates to the existing Mayhew Drain culvert and pumps to prevent backup of floodwater on the Folsom Boulevard side of the gates
- Installing a slurry wall in the north levee of the American River from the east levee of the Natomas east Main Drain upstream for a distance of approximately 1.2 miles
- Installing a slurry wall in the north levee of the American River from 300 feet west of Jacob Lane north for a distance of approximately 1 mile to the end of the existing levee

Section 366 of WRDA 1999 authorized more improvements which included the raising and strengthening of the levees along the American River and additional work in Natomas.

The Common Features GRR was initiated because the economic basis for the original authorization has changed. The Common Features Project has been subject to significant cost increases due to major design modifications and to additional work proposals. Further investigations into additional modes of levee failure (i.e. slope stability, seepage, underground utilities and vegetative growth and long term degradation effects that include erosion) have revealed that in order to ensure the integrity of the levee system, while sustaining 160,000 cfs, much more work is required than was originally identified under WRDA 96 and WRDA 99. According to *Appendix D – Hydraulic Technical Documentation of the F3 Document*, the hydraulic modeling and geotechnical studies have identified potential seepage issues on both the Sacramento and American Rivers and erosion issues on the American River. In order to better describe the potential impact of flooding within the entire Sacramento area, the scope of the Common Features project must be expanded to consider the risk of levee failure along the Sacramento River, American River and the Natomas Basin. This system-wide approach provides a more comprehensive view of the flood risk to the Sacramento metropolitan area.

Congress also authorized the “Folsom Modifications Project” under Section 101 of WRDA 1999 and the “Folsom Dam Raise Project” in 2003. Although these projects were authorized independently, the project performances are intertwined based on when the projects are assumed completed. Due to constructability issues with the “Folsom Modifications Project”, both the “Folsom Modifications Project” and the “Folsom Dam Raise Project” required reexamination. The Corps sought to combine the objectives of these two authorized projects with Reclamation’s dam safety project. This resulted in the Joint Federal Project (JFP), which met the flood damage reduction and dam safety objectives of the USACE, Reclamation, and the local sponsor. The ability of the downstream levees to handle 160,000 cfs is a key factor in achieving the following goals: 1) control the 1-in-200 year event by holding the release at 160,000 cfs (or less) and 2) control the PMF event while maintaining at least 3 ft of freeboard.

A-3 American River Hydrology

The Comprehensive Study data provides the majority of the input to the Hydraulic Design HEC-RAS model. The one exception is the data for the American River. Both the hydrology and routing tool for American River flows differ. Although the HEC-ResSim model built for the Comprehensive Study simulates system-wide operation for multiple reservoirs on the Sacramento River along with those on its major tributaries, the Folsom Dam Excel-based reservoir routing model provides the means necessary to examine Folsom Dam project features in more detail. For consistency, the same hydrology used in other American River studies was utilized for the Common Features GRR. See *Appendix A – Synthetic Hydrology Technical Documentation* for a discussion on the differences between the Comprehensive Study and the American River studies unregulated hydrographs for the American River.

A series of hypothetical inflow hydrographs (i.e. 50%-, 10%-, 4%-, 2%-, 1%-, 0.5%-, 0.2%-annual chance flood events) were developed for the flood risk management analyses. See **Figure A-1**. Design flood hydrographs can be patterned after historical or hypothetical events. In this instance, the flood hydrographs are patterned after the synthetic 2001 PMF event. Each hydrograph consists of multiple waves -- as would occur if a series of storms moved through the region. The sequencing of waves is an important aspect to consider when developing synthetic flood hydrographs. Antecedent waves could induce encroachment into the flood pool prior to the arrival of the main wave. This situation is most likely to occur when a project has limited release capability as under the existing project condition.

The selected hydrograph pattern is proportioned to match the annual maximum 3-day volume and peak for designated exceedance probabilities. The 3-day duration is considered the most critical within the American River basin. Past analyses has shown that the 3-day duration has the greatest impact on operation of the existing flood control system (Folsom Dam and the downstream levees), as well as plan formulation for the American River Basin and most other Sacramento Basin tributaries.

The flood volumes are obtained from a family of unregulated inflow frequency curves. The statistics used to generate these curves were last updated in 2004 using the statistical procedures and methodologies outlined in *Bulletin 17B, Guidelines for Determining Flood Flow Frequency* (United States Geologic Survey [USGS], 1982). *Rain Flood Flow Frequency Analysis, American River, California* (Corps, 2004) documents this process from start to finish beginning with preparation of the data and ending with development of the Log Pearson III statistics presented in **Table A-1.** The mean daily flow at the Fair Oaks gage downstream was used to develop the unregulated inflow for Folsom Dam. The drainage area between Fair Oaks and Folsom Dam does not generate a significant amount of local flow.

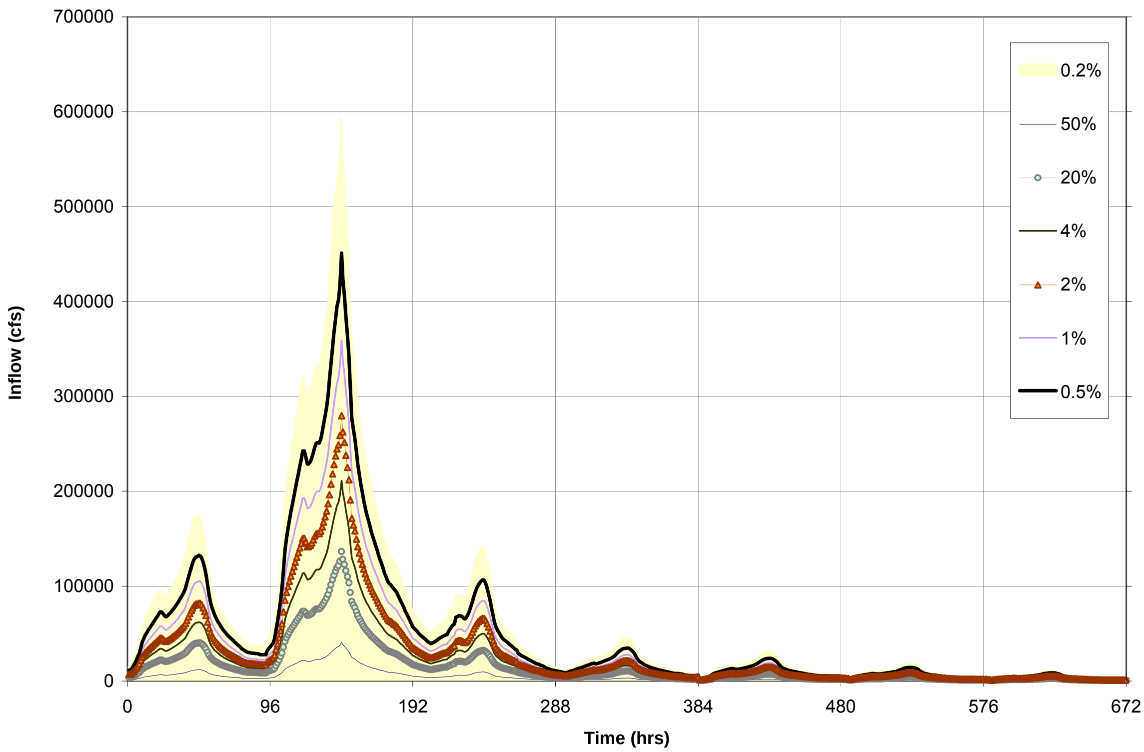

FIGURE A-1
FLOOD HYDROGRAPHS

The flood hydrographs above are based on a storm centered over the American River basin. Other storm centerings (i.e. Shanghai Bend, the mainstem of the Sacramento River) were considered to identify the conditions that would put the most stress on levee locations susceptible to failure. *Appendix A – Synthetic Hydrology Technical Documentation* contains a discussion regarding the development of the Comprehensive Study hydrographs based on the different storm centerings. The Comprehensive Study results were used to identify the coincident frequencies on the American River given a 50%-, 10%-, 4%-, 2%-, 1%-, 0.5%-, or 0.2%-annual chance flood event occurring elsewhere outside the American River basin. These coincident frequencies were used to develop two additional sets of flood hydrographs, one for the Shanghai Bend centering and another for the Sacramento River mainstem centering.

TABLE A-1: American River at Fair Oaks (1905-2004) – Unregulated Inflow Statistics			
Duration	**Log Mean (cfs)**	**Log Standard Deviation (cfs)**	**Skew**
Peak	4.581	0.430	-0.08
1 Day	4.453	0.425	-0.05
3 Day	4.326	0.414	-0.05
7 Day	4.162	0.398	-0.13
15 Day	4.015	0.373	-0.26
30 Day	3.897	0.360	-0.42

The family of unregulated rain flood frequency curves generated from these statistics is presented in **Figure A-2**. Exceedance frequencies can be read off of the mean 3-day rain flood frequency curve (**Figure A-3**). For the 0.01 probability event, the mean 3-day volume is 188,400 cfs.

A-4 Reservoir Model and Operating Assumptions

The Folsom Dam Operations and Planning Model was updated to include the latest storage capacity table developed in 2005, the auxiliary spillway rating curves derived from the Folsom Dam Auxiliary Spillway physical model study results from Nov 2007, and the dam safety assumptions coordinated with Reclamation.

a. Water Control Plan

The Water Control Diagram (WCD) provides the guidelines and limitations defining the release and storage of water within the flood control space. Around 1995, an interim WCD was implemented for Folsom Dam. This interim WCD is the product of an operational agreement between Reclamation and the Sacramento Area Flood Control Agency (SAFCA). The Folsom Dam WCD maintains a minimum allowable flood control reservation of 400,000 acre-feet. With an additional 270,000 acre-feet of variable flood space based on creditable storage available in upstream reservoirs, a maximum flood control reservation of 670,000 acre-feet is possible. This WCD will be referred to as the 400/670 WCD **(Figure A-4)**. The 400/670 diagram is more conservative than the WCD contained in the 1986 Folsom Dam Water Control Manual so there is no conflict in operation.

Under WRDA 1999, Congress directed the reduction of the variable flood control space from the current operating range of 400,000-670,000 acre-feet to 400,000-600,000 acre-feet upon the completion of improvements to Folsom Dam. The modifications to the project will include the construction of an auxiliary spillway under the JFP project, which will be followed by a 3.5 ft dam raise. The hypothetical future WCD for Folsom Dam is herein referred to as the 400/600 WCD **(Figure A-5)**.

Operation within the surcharge pool is prescribed by the applicable Emergency Spillway Release Diagram (ESRD). The diagram is constructed following procedures in EM 1110-2-3600, “Engineering and Design – Management of Water Control Systems”. The ESRD smoothes the transition from releases made under normal flood operation releases to those required for dam safety. The diagram indicates the minimum permissible release that can be made without endangering the structure and without releasing quantities in excess of natural runoff. The ESRD attenuates Folsom Dam flood outflows to a level less than the inflow to the dam. The release specified is made immediately in order to reduce the magnitude of later releases. The objective of the ESRD is to avoid creating a worse situation than already exists and to provide a set of rules to increase flows above the downstream channel capacity in order to protect the dam from overtopping. The ESRD instructs the operators on how and when to make this key operating decisions when the only information known is reservoir elevation and the current release.

b. Operational Limitations

1) Surcharge Storage (Flood Pool) Limitation

Per Code of Federal Regulations (CFR) 33.208.11, the project owner (Reclamation) has full responsibility for the safety of the dam/appurtenant facilities and for regulation of the project during surcharge utilization. In 2007, the Corps and Reclamation reached an agreement that Reclamation practices and standards should take precedence in defining dam safety operation and criteria. The maximum surcharge space requirement is greatly affected by the inflow design flood volume, the total discharge capacity of the project, and the plan of operation. Folsom Dam spillway was originally sized to handle a much smaller inflow design event (the probable maximum flood – aka PMF). The maximum surcharge pool level of 475.5 ft and the accompanying 5 feet of freeboard are no longer sufficient under current conditions. According to the report *American River Basin, California, Folsom Dam and Lake Revised PMF Study* (Corps, 2001), Folsom Dam can only pass 70 percent of the PMF -- assuming full operation of the outlets and spillway gates and no dam failure; The amount of overtopping is estimated to be 3.5 feet above all earthen structures.

Under the Joint Federal Project, the maximum surcharge storage space requirement would increase from elevation 475.5 to elevation 477.5. This increase is accompanied by a decrease in the freeboard requirement per Reclamation's freeboard analyses. Freeboard space above the maximum allowable surcharge storage is needed to prevent overtopping mainly by wind or wave action. The authorized storage space would remain constant and independent of any modifications to the project. The dam safety operation for the Folsom Dam project is constrained by downstream safety considerations which limit or delay increases above what the levees can handle until the reservoir water surface exceeds the designated Flood Pool. The release is held to the emergency objective release while the pool is less than or equal to the designated Flood Pool. Under the existing operation, the Flood Pool is set at elevation 470.0 ft. The 1986 ESRD allows usage of about 45,000 acre-feet of surcharge storage between elevation 466 ft (normal full pool) and elevation 470.0 ft. Once the Flood Pool is exceeded, any delays in meeting the dam safety release requirement may put the dam and downstream inhabitants at greater risk.

2) Discharge Rate of Increase Limitation

Corps guidance EM 1110-2-1420, "Engineering and Design - Hydrologic Engineering Requirements for Reservoirs" states that project operation plans should ensure that release rates-of-change be gradual and not exceed the historical maximum rates of increase. The current Folsom Dam rate-of-increase is 15,000 cfs per 2-hour period. This requirement was applied to all the Scenarios while the discharge remained at or below the emergency objective release. Thereafter, the rate of increase is unlimited for the WO conditions -- similar to the existing operation. For the NA conditions, the rate-of-increase changes to 100,000 cfs/hr while the discharge remains at or below 360,000 cfs. This criterion was coordinated with Reclamation as a requirement for their dam safety operation under the JFP project and the recommended plan (JFP project plus 3.5 ft Dam Raise) as described in the 2007 PAC document.

3) Downstream Channel Limitations

The objective release for normal flood control operation is specified by the WCD. Prior to the authorized Common Features levee improvements, the normal objective release was thought to be 115,000 cfs. Given the information available today, the actual "safe" target for an indefinitely sustained release is 90,000 cfs. The 90,000 cfs offers a zero percent chance of levee failure for the WO condition. The authorized levee improvements enable the levee system to handle 115,000 cfs under normal flood operations. The 115,000 cfs offers a zero percent chance of levee failure for the NA condition. The objective release changes once the emergency flood control operation begins. For the WO condition, the emergency objective release increases to 115,000 cfs. For the NA-145 Scenario, the emergency objective release is increased to 145,000 cfs. For the W-160 Scenario, the emergency objective release is increased to 160,000 cfs. The ability of the downstream channel to sustain 160,000 cfs is a critical assumption for the Joint Federal Project.

A-5 Scenario Description

The Common Features GRR study covers two different Folsom Dam flood routing scenarios for the existing condition: the without-project condition and the no-action future without-project) condition. The without-project (WO) represents the period prior to any work on the levees. The objective release is limited to 115,000 cfs. The no-action condition represents the current state of the levee system after all the authorized repairs and improvements are complete. Under the NA condition, the downstream levees can sustain 145,000 cfs Altogether, there are six routings under the existing condition: WO1, WO2, WO3, NA1-145, NA2-145, and NA3-145. There are three routings under the "with-project" condition: W1-160, W2-160, and W3-160. Refer to **Table A-2** for key information associated with the various scenarios. The following describes the assumptions for each alternative. Given study time constraints, a standard ESRD was assembled for each alternative. No effort was made to "optimize" or tailor the ESRDs beyond establishing the total spillway capacity available, the "Flood Pool" elevation, the emergency objective release limit, and placement of the minimum induced surcharge curve.

a. WO Scenarios

This represents the levee condition existing prior to WRDA 1996 & 1999. The emergency objective release is 115,000 cfs. Prior to the authorized repairs/improvements, the American River levees were thought capable of handling 115,000 cfs under normal flood operations and 160,000 cfs for a short duration to facilitate downstream evacuation. Current studies estimate that the capacity of the levee system under the "without-project condition" was actually closer to 90,000 cfs as a "safe" release for normal flood control operation and no more than 115,000 cfs for emergency releases.

1) WO1 – This represents the levee condition existing prior to WRDA 1996 & 1999. The emergency objective release is 115,000 cfs. The dam safety release is restricted to 115,000 cfs until the water surface reaches 470.0 ft to facilitate evacuation of the downstream. The water control plan consists of the 400/670 water control diagram used in conjunction with a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without maintaining adequate freeboard. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

2) WO2 – This represents the levee condition existing prior to WRDA 1996 & 1999. The emergency objective release is 115,000 cfs. The dam safety release is restricted to 115,000 cfs until the water surface reaches 470.0 ft to facilitate evacuation of the downstream. This scenario reflects improvements to Folsom Dam -- the construction of the Joint Federal Project (auxiliary spillway). The water control plan consists of the 400/600 water control diagram along with a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without overtopping the dam. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

3) WO3 – This reflects additional improvements to Folsom Dam, the construction of the Joint Federal Project (auxiliary spillway) followed by a 3.5 ft dam raise. The emergency objective downstream release is 115,000 cfs. The dam safety release is not allowed to exceed 115,000 cfs until the water surface reaches 470.0 ft in order to facilitate evacuation of the downstream. The water control plan consists of both a 400/600 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without overtopping the dam. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

b. NA Scenarios

The NA scenarios represent the levee condition following the completion of WRDA 1996 & 1999. The downstream levees are capable of sustaining 145,000 cfs. Only, NA2 and NA3 operations are designed to pass the PMF -- meaning these scenarios can contain the resultant maximum surcharge volume within the maximum surcharge pool as specified in **Table A-2**. The resultant freeboard meets the freeboard requirement set by Reclamation for dam safety purposes. This also satisfies the Corps minimum freeboard requirement per regulation *ER 1110-8-2 (FR), "Engineering and Design - Inflow Design Floods for Dams and Reservoirs"*. No other goals or performance criteria were targeted in the NA2-145 and NA3-145 routings. The operation for the NA scenarios is intended to show increased performance as modifications are made to the project. NA3-145 outperforms NA2-145 which in turn must be better than NA1. Except for the downstream emergency objective release constraint of 145,000 cfs, NA2-145 and NA3-145 have operational criteria similar to the future with-project described in the next section.

1) NA1 – This scenario reflects no improvements to Folsom Dam. The emergency objective release is 145,000 cfs. The dam safety release is restricted to 145,000 cfs until the water surface exceeds 470.0 ft to facilitate evacuation of the downstream. The water control plan is comprised of the 400/670 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without maintaining adequate freeboard. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

2) NA2 – This scenario reflects an improvement made to Folsom Dam -- the construction of the Joint Federal Project (auxiliary spillway). The dam safety release is restricted to 145,000 cfs until the water surface reaches 466.0 ft to facilitate evacuation of the downstream. Downstream considerations no longer trump the dam safety operation within the surcharge space above pool elevation 466.0 ft. The water control plan consists of the 400/600 water control diagram along with a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam can pass the PMF without overtopping the dam.

3) NA3 -- This reflects additional improvements to Folsom Dam, the construction of the Joint Federal Project (auxiliary spillway) followed by the 3.5 ft dam raise. The height of the emergency gates will be increased to enable the three emergency spillway gates to remain in the closed position for a longer period, if necessary. The emergency objective downstream release is 145,000 cfs. The dam safety release is not allowed to exceed 145,000 cfs until the water surface exceeds 471.5 ft. The water control plan consists of both a 400/600 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam can pass the PMF without overtopping the dam.

c. W Scenarios

The W scenarios are the future with-project condition. The W2 and W3 scenarios can pass the PMF while still satisfying the minimum 3 ft freeboard requirement for the top of dam. These scenarios are intended to show the increased performance gained by fixing the problems identified post WRDA 1996/1999 authorization. W2-160 and W3-160 have strong similarities to the 2007 PAC Report alternatives. W2-160 and W3-160 have the goal of passing the single 1-in-200 yr design event while maintaining a release of 160,000 cfs. Per coordination with Reclamation on the JFP, their preference is that this design event be maintained within the authorized normal full pool (elevation 466 feet). For the raise project, Reclamation prefers that the maximum water surface for the design event be confined at or below Flood Pool .5 feet.

1) W1 – This scenario reflects no improvements to Folsom Dam. The emergency objective release is 160,000 cfs. The dam safety release is restricted to 160,000 cfs until the water surface exceeds 466.0 ft. The water control plan is comprised of the 400/670 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without maintaining adequate freeboard. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

3) W2 – This scenario reflects an improvement made to Folsom Dam -- the construction of the Joint Federal Project (auxiliary spillway). The dam safety release is restricted to 160,000 cfs until the water surface exceeds 466.0 ft. Downstream considerations no longer trump the dam safety operation within the surcharge space above pool elevation 466.0 ft. The water control plan consists of the 400/600 water control diagram along with a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam can pass the PMF without overtopping the dam.

3) W3 -- This reflects additional improvements to Folsom Dam, the construction of the Joint Federal Project (auxiliary spillway) followed by the 3.5 ft dam raise. The height of the emergency gates will be increased to enable the three emergency spillway gates to remain in the closed position for a longer period, if necessary. The emergency objective downstream release is 160,000 cfs. The dam safety release is not allowed to exceed 160,000 cfs until the water surface reaches 471.5 ft. The water control plan consists of both a 400/600 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam can pass the PMF without overtopping the dam.

TABLE A-2: DESCRIPTION OF SCENARIOS

Alternative	Top of Dam	Maximum Surcharge Flood Pool[1]	Freeboard [3]	Flood Pool [4]	Emergency Objective Release	Normal Flood Control Reservation Range [5]
	El, ft	El, ft	El, ft	El, ft	Cfs	El, ft (acre-feet)
WO1 Pre-Common Features	480.5	475.5[2]	5	470.0	90,000 (< 35% encroachment) 115,000 (> 35% encroachment)	425.8 to 388.3 (400,000 – 670,000)
WO2 Pre-Common Features Auxiliary Spillway	480.5	475.5[2]	5	470.0	90,000 (< 35% encroachment) 115,000 (> 35% encroachment)	425.8 to 399.7 (400,000 – 600,000)
WO3 Pre-Common Features Auxiliary Spillway Folsom Dam Raise 3.5 ft	484.0	479.0	5	470.0	90,000 (< 35% encroachment) 115,000 (> 35% encroachment)	425.8 to 399.7 (400,000 – 600,000)
NA1-145 Common Features	480.5	475.5	5	470.0	145,000	(425.8 to 388.3 400,000 – 670,000)
NA2-145 Common Features Auxiliary Spillway	480.5	477.5	3	466.0	145,000	425.8 to 399.7 (400,000 – 600,000)
NA3-145 Common Features Auxiliary Spillway Folsom Dam Raise 3.5 ft	484.0	481.0	3	471.5	145,000	425.8 to 399.7 (400,000 – 600,000)
W1-160 Common Features	480.5	475.5	5	470.0	160,000	(425.8 to 388.3 400,000 – 670,000)
W2-160 Common Features Auxiliary Spillway	480.5	477.5	3	466.0	160,000	425.8 to 399.7 (400,000 – 600,000)
W3-160 Common Features Auxiliary Spillway Folsom Dam Raise 3.5 ft	484.0	481.0	3	471.5	160,000	425.8 to 399.7 (400,000 – 600,000)
KEY El, ft – Elevation in feet						

Notes:

1. *These values reflect the highest allowable pool elevation given both freeboard and top of dam height requirements. The maximum surcharge flood pool is established by routing a PMF through the reservoir. The PMF has been updated or revised periodically (e.g. 1946, 1980, 1991, and 2001).*
2. *The existing project requires more surcharge storage than is available under the original project design. Under existing conditions with no modifications to Folsom Dam, the 2001 PMF event would overtop Folsom Dam.*
3. *Reclamation has determined that 3 feet provides sufficient freeboard for the with-project scenarios (no action).*
4. *The FDR flood pool elevations are associated with the JFP and 3.5 Ft Dam Raise projects described in the PAC document. The release from Folsom Dam will not exceed 160,000 cfs as long as the water surface remains at or below the FDR flood pool.*
5. *The authorized storage space allocation for flood control differs with the scenarios. The flood space requirement itself varies seasonally. The maximum space would be needed only during the most critical flood period (December through February)*

A-6 Summary of Routing Output Analyses

a. WO Scenarios (pre-dates improvements authorized under WRDA 1996 & 1999)

With the addition of an auxiliary spillway in WO2, the main benefit gained is the ability to accelerate evacuation of the flood space. Although the downstream channel was originally designed to sustain an objective release of 115,000 cfs under normal flood operations, the current findings is that the potential for levee failure was greater than thought possible at that time. Under today's standards, the downstream channel was never maintained well enough to sustain safe releases of 115,000 cfs. To ensure zero percent chance of failing the downstream levees, the normal objective release requirement should have been reduced to 90,000 cfs. According to the attached **Figure A-8**, WO1 is able to limit the release to 90,000 cfs up to a 1-in-25 yr chance event. WO2 and WO3 must not utilize the extra capacity made available by the addition of the auxiliary spillway beyond this "safe" level except for events larger than a 1-in-25 yr chance event. Reservoir encroachment is the unit of measurement selected to identify event size. The encroachment volume for a 1-in-25 yr chance event never exceeded 35% in the WO1 routing. Therefore, larger events would be characterized by their larger encroachment percentages. Thus, the model was adjusted to limit the release to 90,000 cfs as long as the encroachment level remained at or below 35%. Thereafter, the release restriction would be lifted and the discharge would be allowed to ramp up to 115,000 cfs.

The operation for the WO scenarios is intended to show increased performance as modifications are made to the Common Features project and improvements are made to Folsom Dam. WO3 outperforms WO2 which in turn is better than WO1. The WO scenarios were not intended to pass the PMF. Operation for the WO scenarios was not constrained by any measurable criteria (i.e. passing a certain percentage of the PMF or limiting the magnitude of any dam overtopping to a certain amount). These scenarios cannot contain the resultant maximum surcharge volume within the confines of the maximum surcharge pool specified in **Table A-2**. The resultant freeboard is also less than the required freeboard amount. For these scenarios, the operation postpones making releases greater than 115,000 cfs due to downstream considerations by using up to 4 ft of surcharge storage space. The dam safety release is restricted to 115,000 cfs until the water surface reaches 470.0 ft to facilitate evacuation of the downstream.

b. NA Scenarios

The ESRDs created for the various scenarios may be considered much too efficient. The NA3-145 alternative is an example of this. According to the attached **Figure A-9**, the routing results indicate that Folsom Dam operations can hold the release at 145,000 cfs for a 1-in-200 yr event. Note, however, significant use of the surcharge space is required to achieve this result. The "Flood Pool" is being greatly exceeded. The release is appropriate given the circumstances in the routing with rapidly falling inflow and insignificant rate of rise in the reservoir pool elevation. The only way to make the consequences of exceeding the "Flood Pool" fully apparent in the routing is to use "simplified" ESRDs -- ones in which the pool elevation would be the only factor used to determine the discharge requirement. The "simplified" ESRD would remove any flexibility in surcharge space usage by automatically forcing the discharge to increase beyond the target flow anytime the pool elevation exceeded the designated "Flood Pool". Under this scenario, at 471.5 ft the discharge would be held to 145,000 cfs but at 471.51 the release would be greater than 145,000 cfs. The "soft" enforcement makes more sense than the "hard" enforcement approach when it comes to reservoir operations. **Table A-3** offers a comparison of maximum water surface versus "Flood Pool" specification for the various scenarios.

c. W Scenarios

TABLE A-3: FLOOD POOL ROUTING SUMMARY Ŧ

1-in-N chance per year event	WO1 (Flood Pool 470.0 ft)		WO2 (Flood Pool 470.0 ft)		WO3 (Flood Pool 470.0 ft)		NA1-145 (Flood Pool 470.0 ft)		NA2-145 (Flood Pool 466.0 ft)		NA3-145 (Flood Pool 471.5 ft)		W1-160 (Flood Pool 470.0 ft)		W2-160 (Flood Pool 466.0 ft)		W3-160 (Flood Pool 471.5 ft)	
	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)
2	403.93	30295	403.53	37708	403.53	37708	402.43	30183	403.18	25215	403.18	25215	403.08	25891	401.91	37708	403.18	25215
10	429.80	43692	408.97	90000	408.97	90000	429.13	43127	421.65	71655	421.65	71655	431.09	43519	421.65	71655	421.65	71655
25	442.53	98760	427.80	90000	427.80	90000	442.69	99738	431.43	115000	431.43	115000	444.54	104311	432.02	115000	432.02	115000
50	457.34	115000	443.02	115000	443.02	115000	457.01	115000	442.97	115000	442.97	115000	459.13	115000	444.04	115000	444.04	115000
100	476.35	123107	461.00	115000	461.00	115000	470.81	145000	460.46	115000	460.46	115000	472.32	145000	461.31	115000	461.31	115000
200	476.33	444310	476.65	169173	478.67	138359	476.40	320142	470.02	210332	474.92	145000	476.37	321017	470.02	196633	472.47	160000
250	476.65	476319	475.23	331691	477.27	232803	476.67	412114	470.65	309673	477.90	197562	476.64	408551	470.44	296022	477.15	193667
500	479.62	554268	480.97	627077	481.31	510279	479.01	512982	472.08	594159	478.32	558062	479.04	513195	471.57	594159	478.03	534386

Notes:

Ŧ *The gray shaded area depicts encroachment into the remaining surcharge storage space above the "Flood Pool" mark; Dam Safety operation takes the highest priority above the "Flood Pool" mark.*

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A-7 Risk Analysis (HEC- FDA Inputs)

Corps engineering guidance (EM 1110-2-1619, “Risk-Based Analysis for Flood Damage Reduction Studies”) and planning guidance (ER 1105-2-100, “Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures” and ER 1105-2-101, “Risk Analysis for Flood Damage Reduction Studies”) require that risk analyses be used to quantify the project performance of the various scenarios. The hydrologic data provided to Economics as input for the HEC-FDA program includes the unregulated inflow exceedance probability function and the curves defining the relationship between unregulated inflow and reservoir discharge. The uncertainty in the hydrology is defined by the confidence limits, derived via statistics. The uncertainty in reservoir discharge is derived by changing the parameters used in the reservoir routings. The risk analysis scenarios reflect the operating conditions ranging from the most likely to occur (BASE) to the most extreme operating conditions likely to produce the largest (MAXIMUM) or smallest (MINIMUM) expected release. The BASE condition assumptions and results are previously described for the W01, W02, W03, NA1, NA2, and NA3 scenarios. Generally, the operational criteria are developed based on actual flood operations, the analysis of historical data, and discussion between representatives of the Corps, SAFCA, and Reclamation. **Table A-4** presents selected assumptions used to create the different scenarios.

TABLE A-4: RISK ANALYSIS OPERATIONAL ASSUMPTIONS [1, 2]

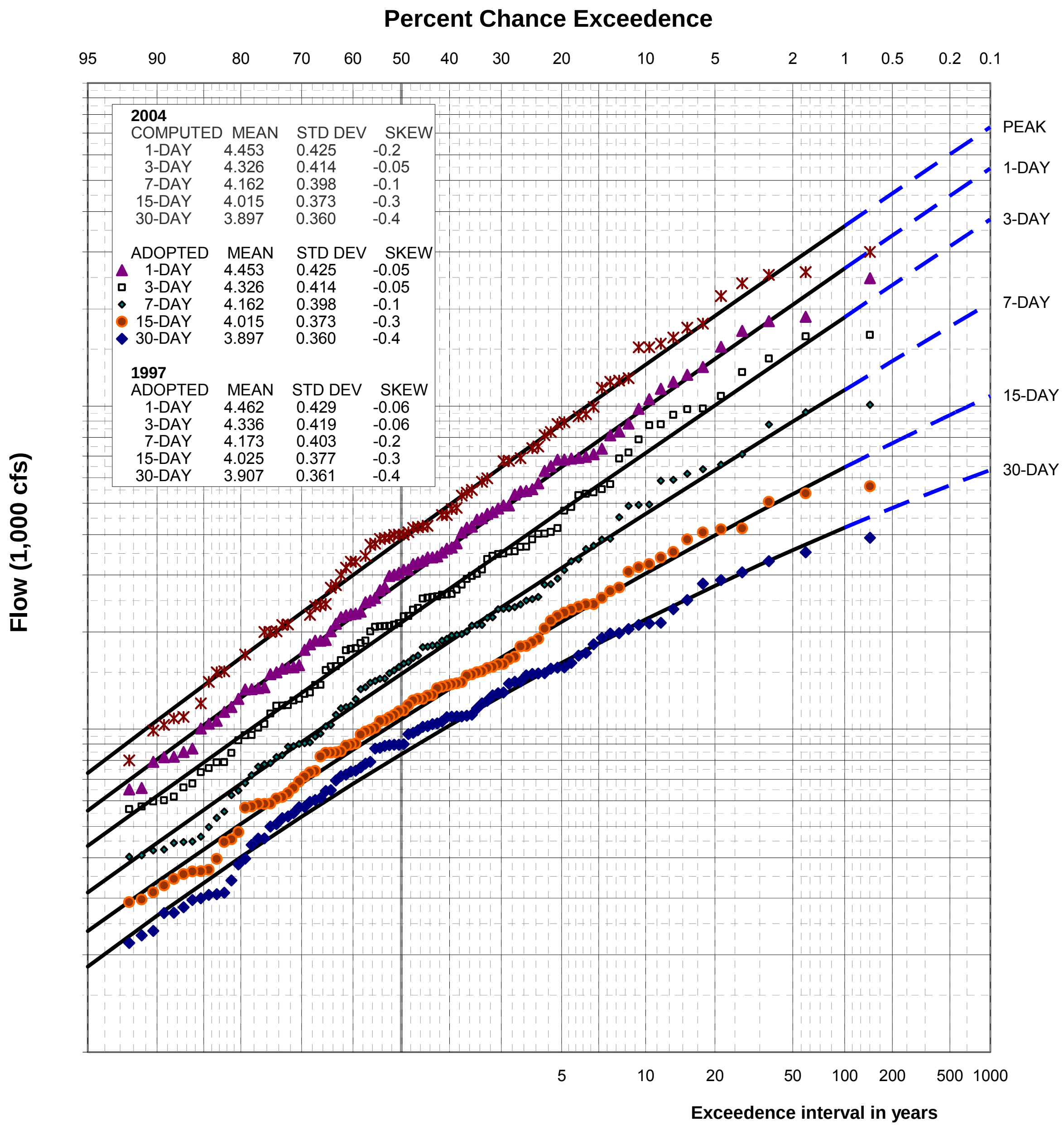
Uncertainty Parameters	Alternative	Discharge Scenario: BASE (Normal)	MAXIMUM (Upper Limit)	MINIMUM (Lower Limit)
Initial Encroachment [3] (acre-feet)	WO & NA	0	50,000	0
Extra Space in Folsom Lake (acre-feet)	WO & NA	0	0	100,000
Available Upstream Reservoir Space (acre-feet)	WO & NA	0	0	150,000
Starting Storage (acre-feet)	WO & NA	367,000	417,000	429,000
Response Time Delay [4] (hours)	WO	8	8	8
	NA	4	8	0
Main Dam River Outlets Operation During Concurrent Spillway Operation (percent gate opening)	WO & NA	60	0	60
KEY Cfs – cubic feet per second				

Notes:
1. *Discharge is presumed through only one power penstock due to maintenance work during the flood season (per Reclamation).*
2. *Application of the uncertainty parameters may sometimes result in anomalies for the smaller or more frequent events. The settings meant to induce the largest or smallest discharge may actually result in the reverse. This issue appears intermittently.*
3. *Encroachment is relative to the allowable storage as determined from the water control diagram (dependent on upstream storage space).*
4. *Lag in matching Release to previous hour Inflow – while discharge is less than the normal objective release target.*

A-8 Conclusion

Water Management produced routings for two different scenarios. The without-project (WO) condition reflects the American River levee system prior to any improvements or repair work. The no-action (NA) condition reflects the existing state of the American River levees with the improvements made as authorized by WRDA 1996 and 1999. The NA condition will result in the ability of the downstream channel to sustain 145,000 cfs (or 160,000 cfs as reported in the 2007 PAC Report). The 50%-, 20%-, 4%-, 2%-, 1%-, 0.5%, 0.2%-annual chance flood events were routed through Folsom Dam for the various WO and NA scenarios. The routing results were given to Hydraulic Design for the floodplains development and to Economics for the economic benefit analyses. The hydrographs provided to Hydraulic Design are shown in **Figures A-4 through A-6**.

Figure A-10 through A-23 provides a snapshot of the data provided to Economics in a variety of ways. **Figure A-10 through A-13** presents the set of WO, NA, and W results (BASE condition only) as regulated frequency curves. This allows one to view the increase in project performance as improvements are made to Folsom Dam. **Figure A-14** consolidates the results of all the routings (BASE condition only) as "inflow versus outflow curves" to allow comparisons across the different set of routings. **Figure A-15 through A-23** presents the uncertainty band around the discharge for any given event. Note that the uncertainty range required some adjustment around the more frequent event where the points crossed. Generally, the anomalies (MAX < BASE < MIN) where the points cross occur for events with less than 1-in-5 yr chance exceedance. In these instances, the MAX discharge is lower than BASE due to the inability to match inflow quickly (8 hour lag). This handicap is a benefit or plus for the smaller flood events. The MIN discharge is large than BASE due to the ability to match inflow quickly (1 hour lag). This advantage (rapid response) is a detriment or negative for the smaller, more frequent events. The initial starting storage also is a factor in this aspect. A full summary of the routings can be found in **Tables A-5** through **A-31**. The reservoir routings covered herein were developed for planning purposes only. These scenarios are hypothetical and would not be built or implemented as stand-alone projects. All reservoir elevations provided herein use the NGVD29 vertical datum.



NOTES:

1. Adjusted USGS gage 11446500 to account for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions.
3. Computed Probability
4. No adjustments for outliers.
4. Drainage area: 1,888 sq. mi.
5. Period of record: 1905-2004.

AMERICAN RIVER WATERSHED PROJECT
COMMON FEATURES
GENERAL REEVALUATION REPORT

RAIN FLOOD FREQUENCY CURVES
AMERICAN RIVER AT FAIR OAKS
UNREGULATED CONDITIONS

FIGURE A-2

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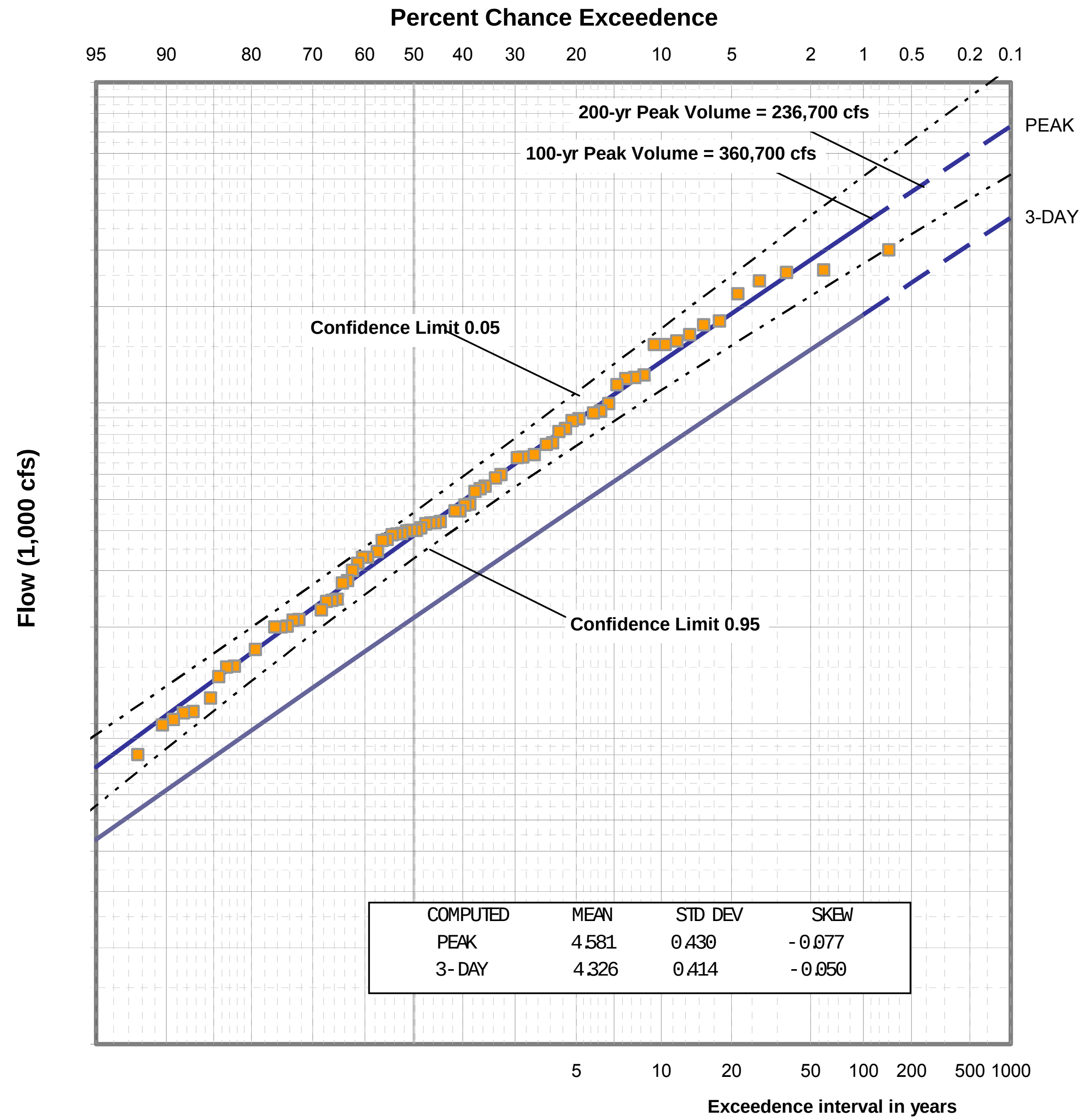

NOTES:

1. Adjusted USGS gage 11446500 to account for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions.
3. Computed Probability
4. No adjustments for outliers.
5. Confidence limits based on station statistics
6. Drainage area: 1,888 sq. mi.
7 Period of record: 1905-2004.

AMERICAN RIVER WATERSHED PROJECT

COMMON FEATURES

UNREGULATED PEAK AND MEAN 3-DAY RAIN FLOOD FREQUENCY CURVES AMERICAN RIVER AT FAIR OAKS

FIGURE A-3

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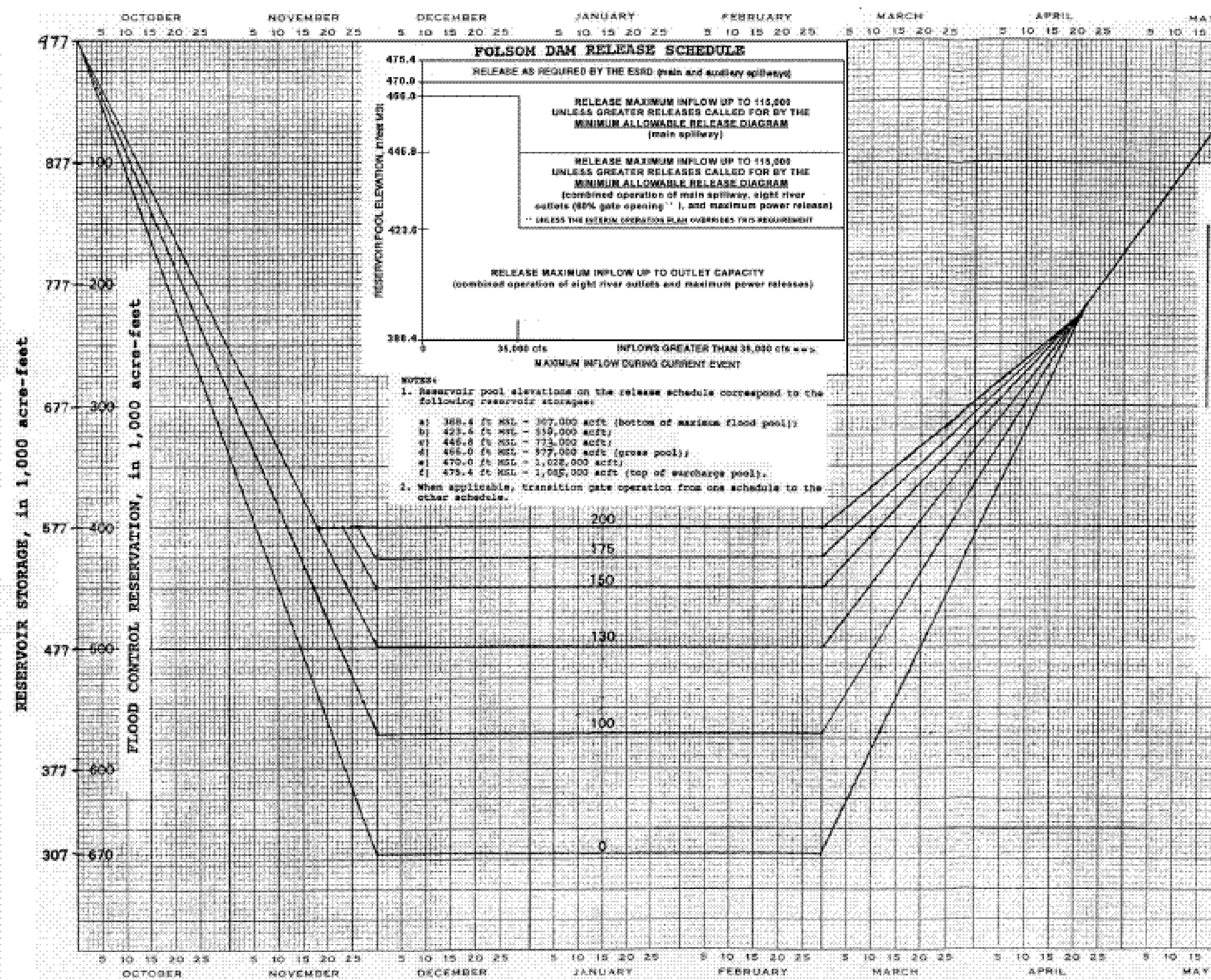

FLOOD CONTROL DIAGRAM FOR FOLSOM DAM AND LAKE

USE OF DIAGRAM

1. Folsom Dam and Lake shall be operated for flood control in accordance with the Flood Control Diagram. When water is stored within the Flood Control Reservation, reservoir releases must be in accordance with requirements of this diagram.
2. The parameters on the flood control diagram define the required Flood Control Reservation, on any given day, based on available space in the upstream reservoirs. Once the required Flood Control Reservation is computed, the Required Reservoir Storage for flood control can be determined. Water stored in excess of the Required Reservoir Storage must be evacuated. Computation of the parameter is discussed below:

FOLSOM DAM AND LAKE
COMPUTATION OF REQUIRED RESERVOIR STORAGE

1. Compute space available below spillway crest, in acre-feet, for the following reservoirs: French Meadows, Hell Hole and Union Valley.
2. The amount of creditable flood control transfer space in each reservoir is then computed by taking the smaller of the space available or the maximum creditable space for that reservoir.
   a. The maximum creditable space by reservoir is as follows:
      French Meadows - 45,000 acre-feet
      Hell Hole - 80,000 acre-feet
      Union Valley - 75,000 acre-feet
   b. Combine the creditable flood control transfer space for each reservoir to compute the total creditable space.
3. Determine the Flood Control Reservation at Folsom Lake by applying the creditable flood control transfer space (parameter on the Flood Control Diagram in 1,000 acre-feet).

SAMPLE COMPUTATION OF REQUIRED RESERVOIR STORAGE

RESERVOIR	STORAGE ON JAN 1 (TAF)	STORAGE AT SPILLWAY CREST (TAF)	SPACE AVAILABLE (TAF)	MAXIMUM CREDITABLE SPACE (TAF)	CREDITABLE FLOOD CONTROL TRANSFER SPACE (TAF)
French Meadows	75.7	110.7	35	45	35
Hell Hole	87.6	207.6	120	80	80
Union Valley	170.1	235.1	65	75	65
TOTAL CREDITABLE FLOOD CONTROL TRANSFER SPACE (TAF)					180
FLOOD CONTROL RESERVATION AT FOLSOM LAKE (TAF)					420
REQUIRED RESERVOIR STORAGE AT FOLSOM LAKE (TAF)					557

3. During a potential flood situation, water stored within the Flood Control Reservation, defined hereon, shall be released as rapidly as possible subject to the following schedule:
   a. Required Flood Control Release - Promptly release inflow up to 115,000 cfs, while inflows are increasing, as discussed in the FOLSOM DAM RELEASE SCHEDULE. Control flows in the American River below the dam to not more than 115,000 cfs, except when larger releases are required by the accompanying INTERIM OPERATION PLAN.
   b. Releases will not be increased more than 15,000 cfs or decreased more than 10,000 cfs during any 2 hour period.

4. Maintenance or repair of any one power generating unit at Folsom Dam which results in loss of the power penstock for more than one day may require additional Flood Control Reservation. The amount of additional reservation is defined by the Adjusted Required Reservoir Storage, as identified on the graph at right, depending on the following conditions:
   a. The Required Reservoir Storage in Folsom Lake, as computed in paragraph 2 above;
   b. The existing and forecasted meteorological conditions in the basin; and,
   c. The duration the generating unit is expected to be off-line.

Required Reservoir Storage greater than or equal to 525,000 acre-feet requires no additional Flood Control Reservation.

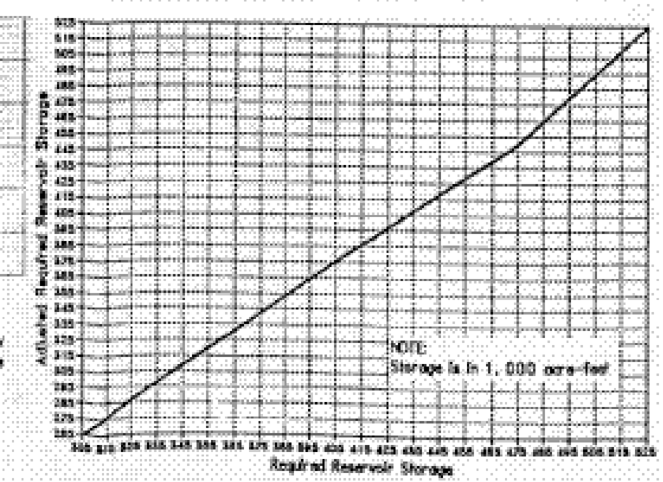

FIGURE A-4
WATER CONTROL DIAGRAM -- HISTORICAL
EXISTING CONDITION 400/670

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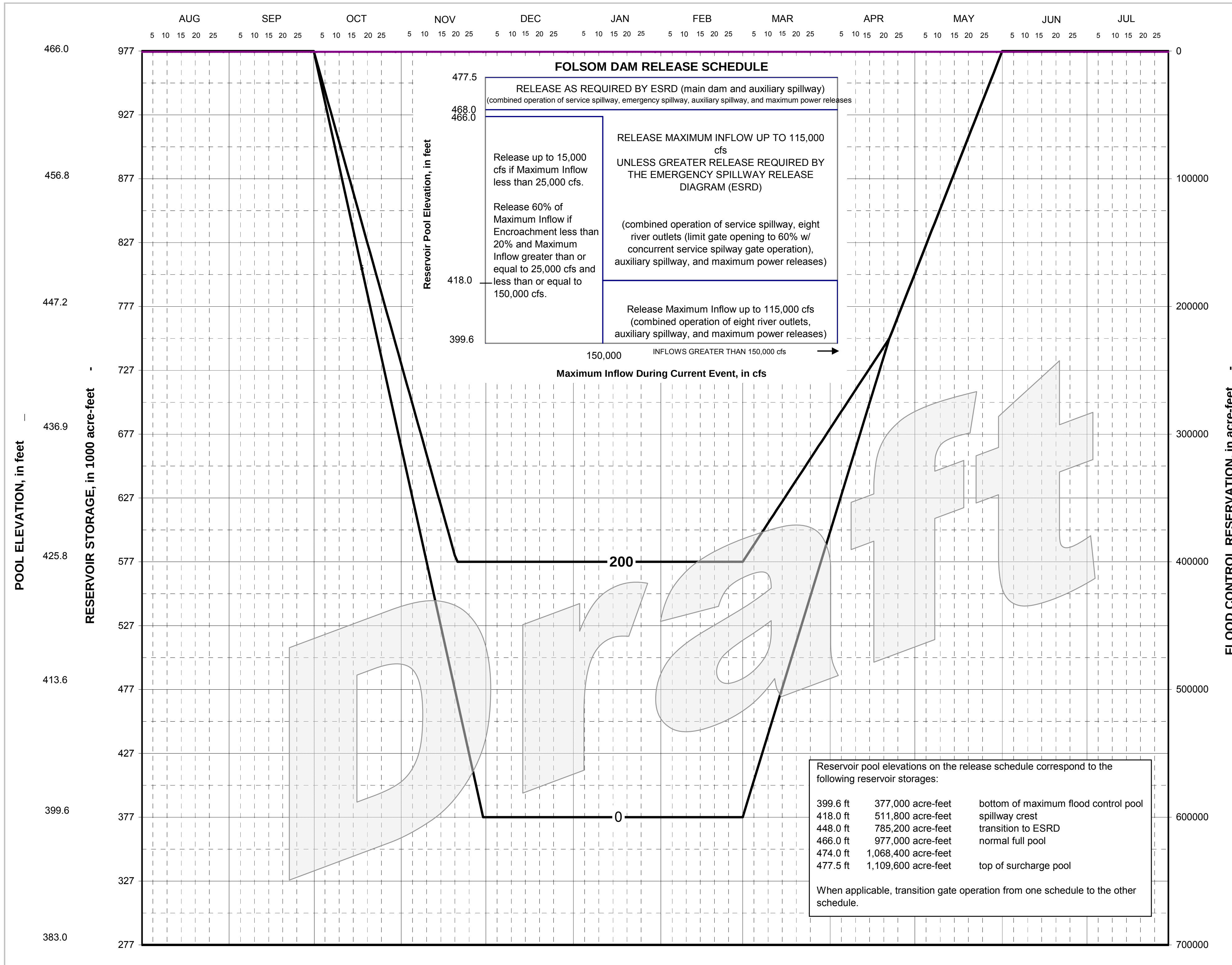

FLOOD CONTROL DIAGRAM

USE OF DIAGRAM

1. Folsom Dam and Lake shall be operated for flood control in accordance with the Flood Control Diagram. When water is stored within the Flood Control Reservation, reservoir releases must be in accordance with the requirements of this diagram.

2. The parameters on the flood control diagram define the required Flood Control Reservation, on any given day, based on available space in the upstream reservoirs. Once the required Flood Control Reservation is computed, the Required Reservoir Storage for flood control can be determined. Water stored in excess of the Required Reservoir Storage must be evacuated. Computation of the parameter is discussed below:

COMPUTATION OF REQUIRED FLOOD RESERVATION STORAGE

1. Compute space available below spillway crest, in acre-feet, for the following reservoirs: French Meadows, Hell Hole and Union Valley.

2. The amount of creditable flood control transfer space in each reservoir is then computed by taking the smaller of the space available or the maximum creditable space for that reservoir.

   a. The maximum creditable space by reservoir is as follows:
      French Meadows 45,000 acre-feet
      Hell Hole 80,000 acre-feet
      Union Valley 75,000 acre-feet

   b. Combine the creditable flood control transfer space for each reservoir to compute the total creditable space.

   c. Determine the Flood Control Reservation at Folsom Lake by applying the creditable flood control transfer space (parameter on the Flood Control Diagram in 1,000 acre-feet).

SAMPLE COMPUTATION OF REQUIRED RESERVOIR STORAGE

RESERVOIR	STORAGE ON JAN 1 (TAF)	STORAGE @ SPILLWAY CREST (TAF)	SPACE AVAILABLE (TAF)	MAXIMUM CREDITABLE SPACE (TAF)	CREDITABLE FLOOD CONTROL TRANSFER SPACE (TAF)
FRENCH MEADOWS	65.7	110.7	45	45	45
HELL HOLE	87.6	207.6	120	80	80
UNION VALLEY	160.1	235.1	75	75	75
TOTAL CREDITABLE FLOOD CONTROL TRANSFER SPACE (TAF)					200
FLOOD CONTROL RESERVATION AT FOLSOM LAKE (TAF)					577
REQUIRED RESERVOIR STORAGE AT FOLSOM LAKE (TAF)					577

RELEASE SCHEDULE

3. During a potential flood situation, water stored within the Flood Control Reservation, defined herein, shall be released as rapidly as possible subject to the following schedule:

   a. Required flood Control Release - Promptly release inflow up to 115,000 cfs while inflows are increasing, as discussed in the FOLSOM DAM RELEASE SCHEDULE. Control flows in the American River below the dam to not more than 115,000 cfs, except when larger releases are required by the accompanying EMERGENCY SPILLWAY RELEASE DIAGRAM (ESRD). Once the reservoir pool begins falling, maintain releases in excess of inflow until water stored in the Flood Control Reservation is evacuated.

   b. Releases will not be increased more than 30,000 cfs or decreased more than 10,000 cfs during any 2-hour period.

FIGURE A-5
WATER CONTROL DIAGRAM -- HYPOTHETICAL
FUTURE CONDITION 400/600

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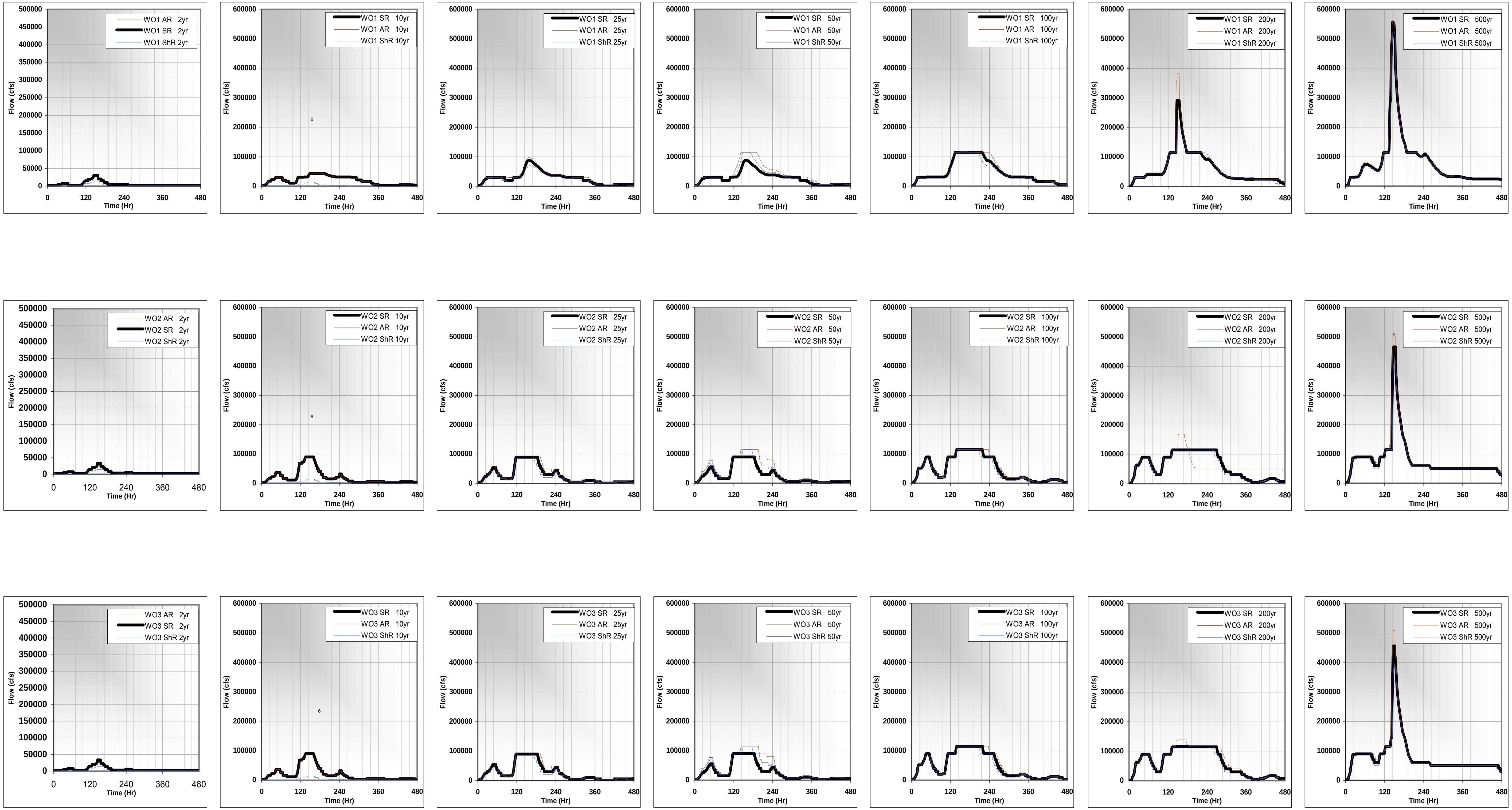

Key

SR Sacramento River
AR American River
ShR Shanghai River

FIGURE A-8: DISCHARGE HYDROGRAPHS – BASE – EXISTING CONDITION

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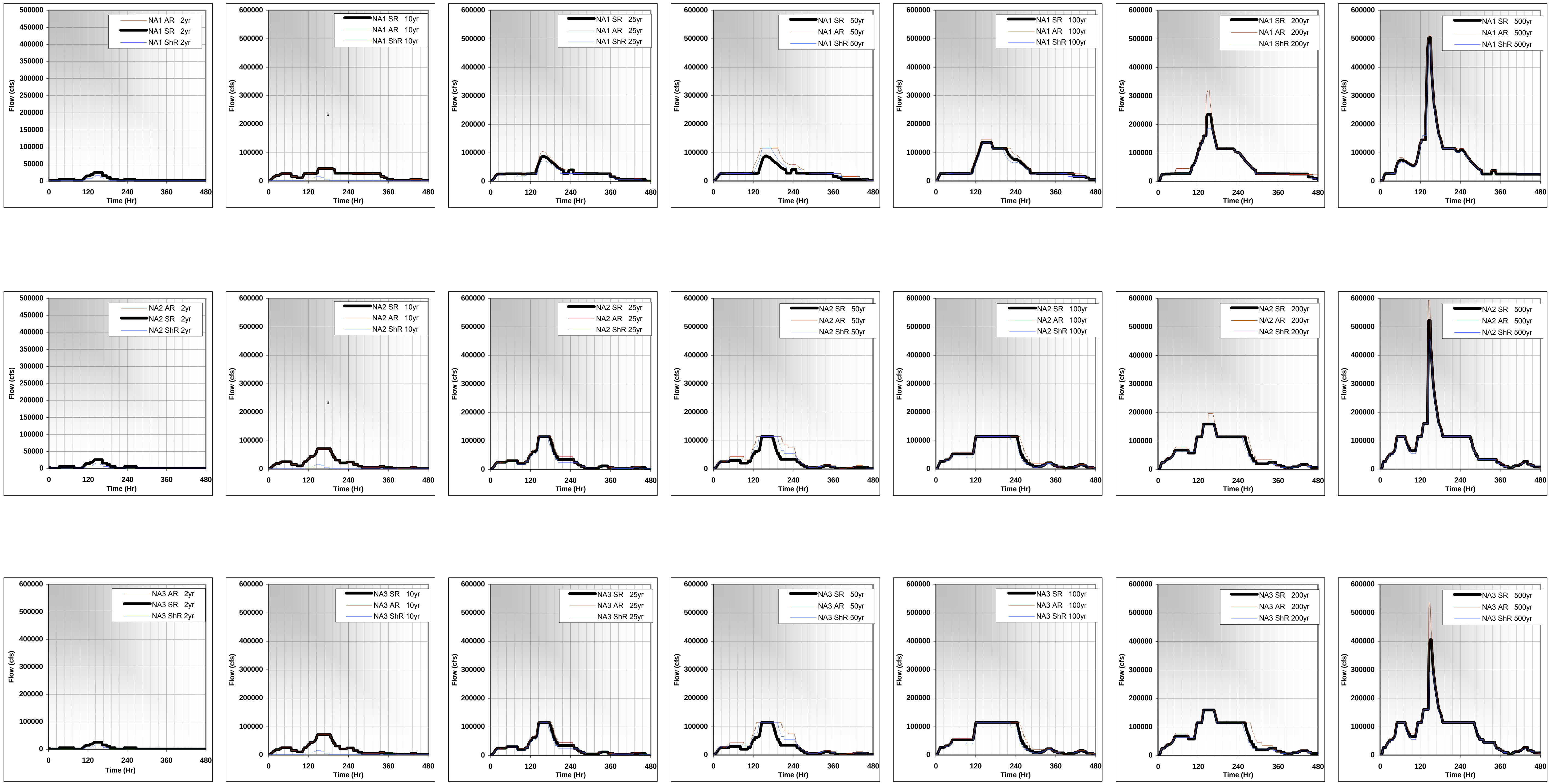

Key

SR Sacramento River
AR American River
ShR Shanghai River

FIGURE A-9: DISCHARGE HYDROGRAPHS – BASE – FUTURE WITHOUT-PROJECT (NO ACTION)

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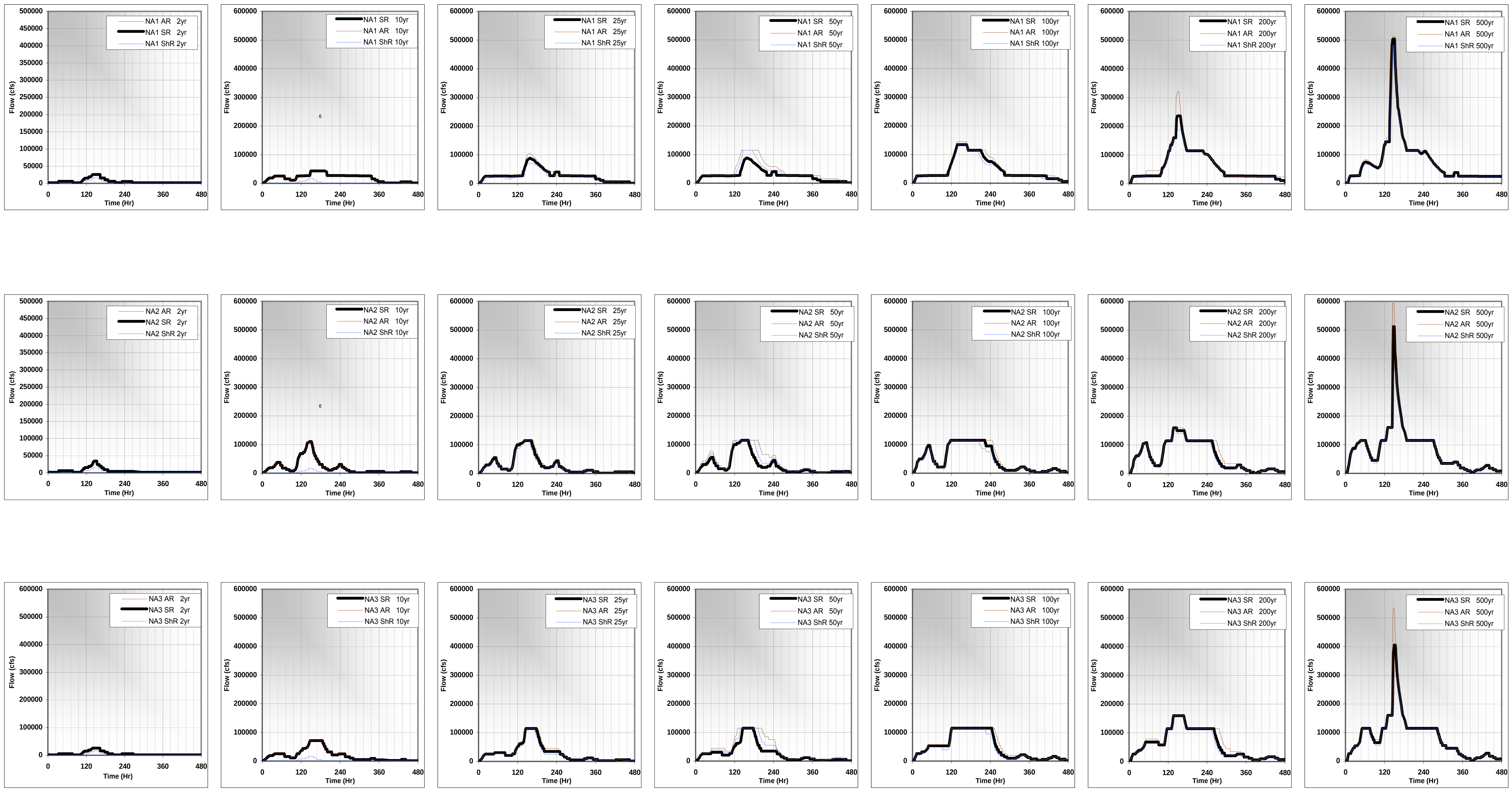

Key

SR Sacramento River
AR American River
ShR Shanghai River

FIGURE A-10: DISCHARGE HYDROGRAPHS – BASE – WITH-PROJECT

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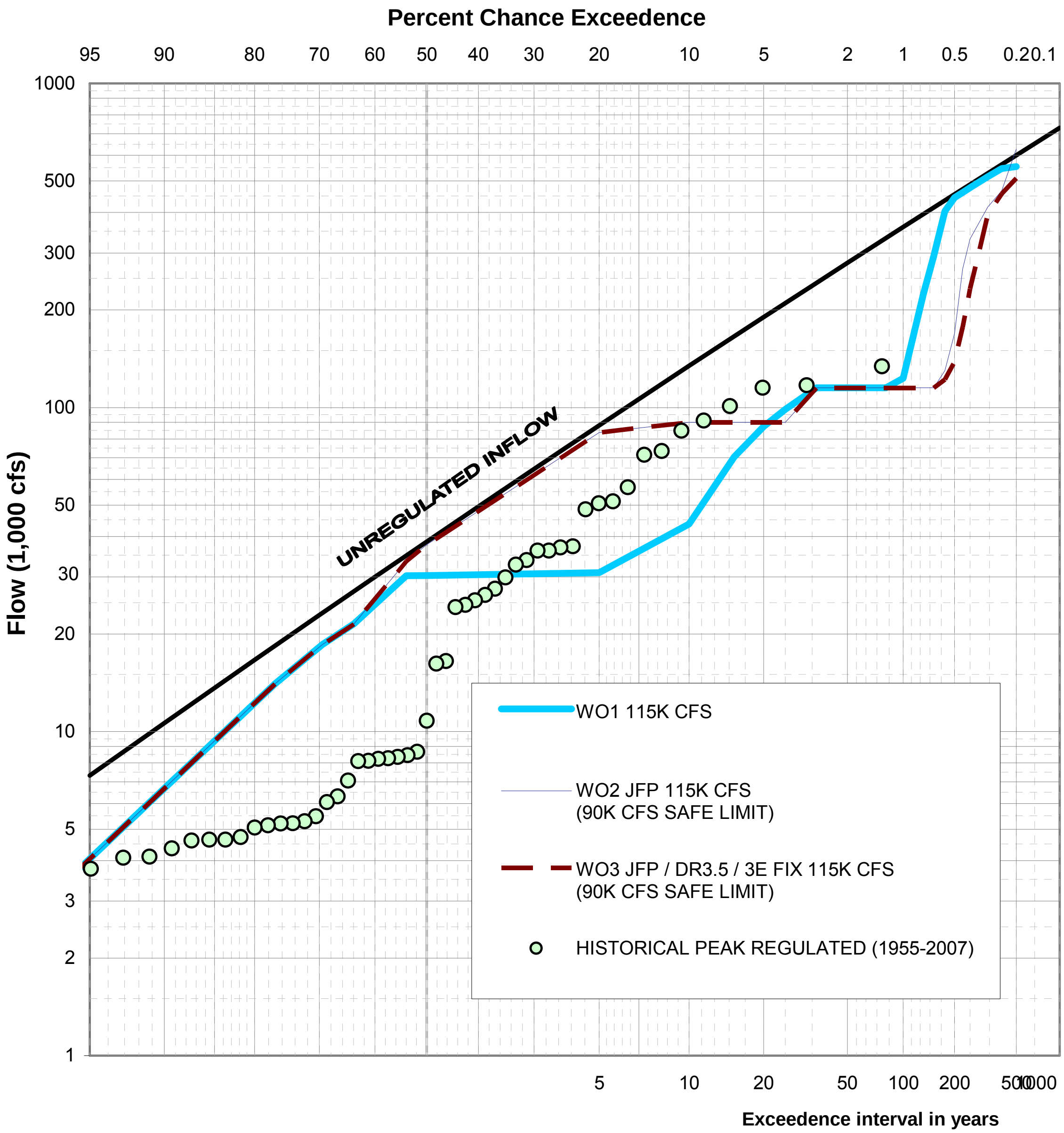

NOTES:

1. Adjusted USGS gage 11446500 to accoiunt for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions
3. No adjustments for outliers.
4. Drainage area: 1,888 sq. mi.
5. Period of record: 1905-2004 (Unregulated).

AMERICAN RIVER WATERSHED PROJECT
COMMON FEATURES GRR

REGULATED FREQUENCY CURVES
FOLSOM DAM
EXISTING CONDITION
(WITHOUT-PROJECT)

FIGURE A-11

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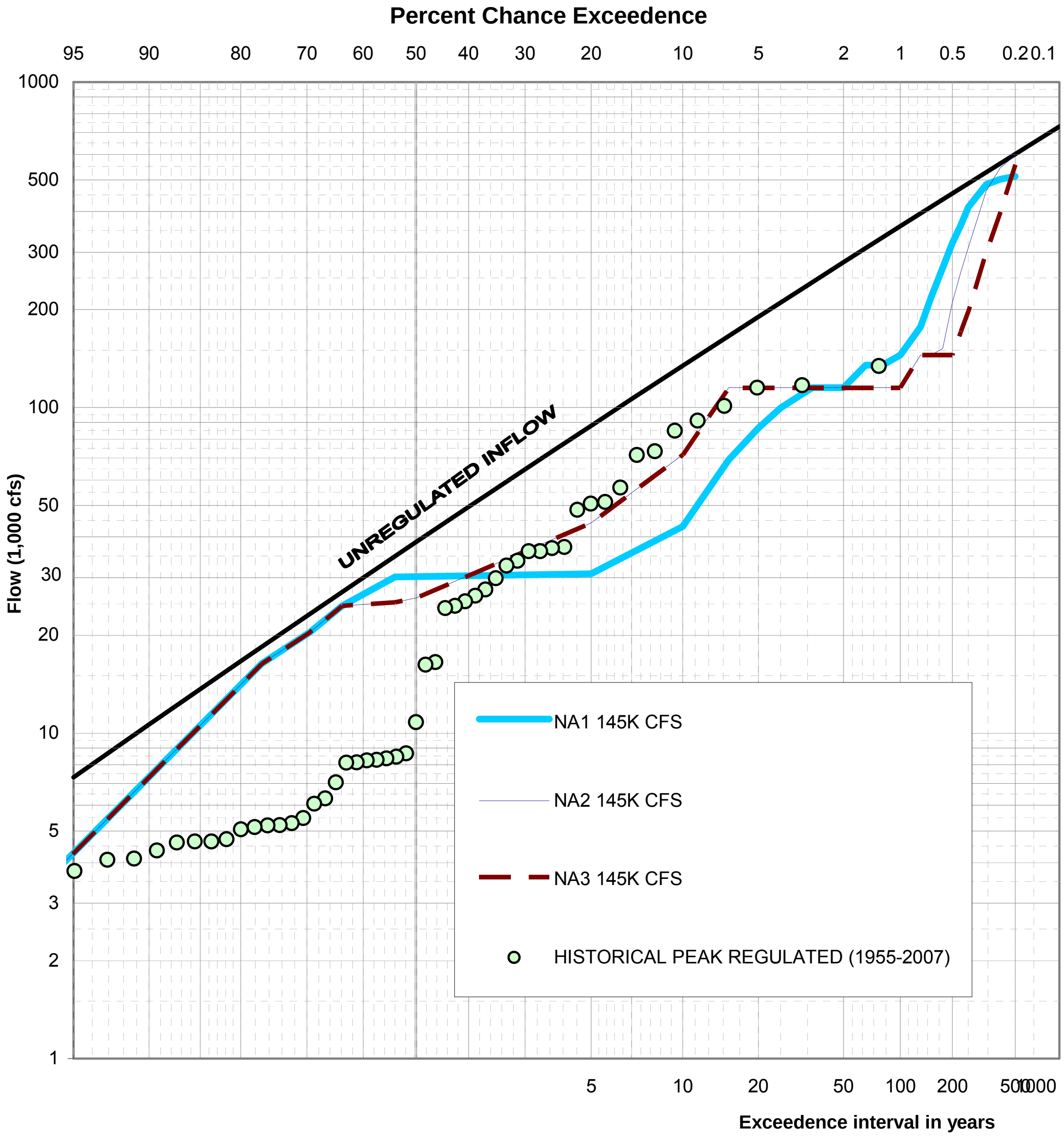

NOTES:

1. Adjusted USGS gage 11446500 to accoiunt for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions
3. No adjustments for outliers.
4. Drainage area: 1,888 sq. mi.
5. Period of record: 1905-2004 (Unregulated).

AMERICAN RIVER WATERSHED PROJECT
COMMON FEATURES GRR

REGULATED FREQUENCY CURVES
FOLSOM DAM
FUTURE WITHOUT-PROJECT
(NO ACTION)

FIGURE A-12

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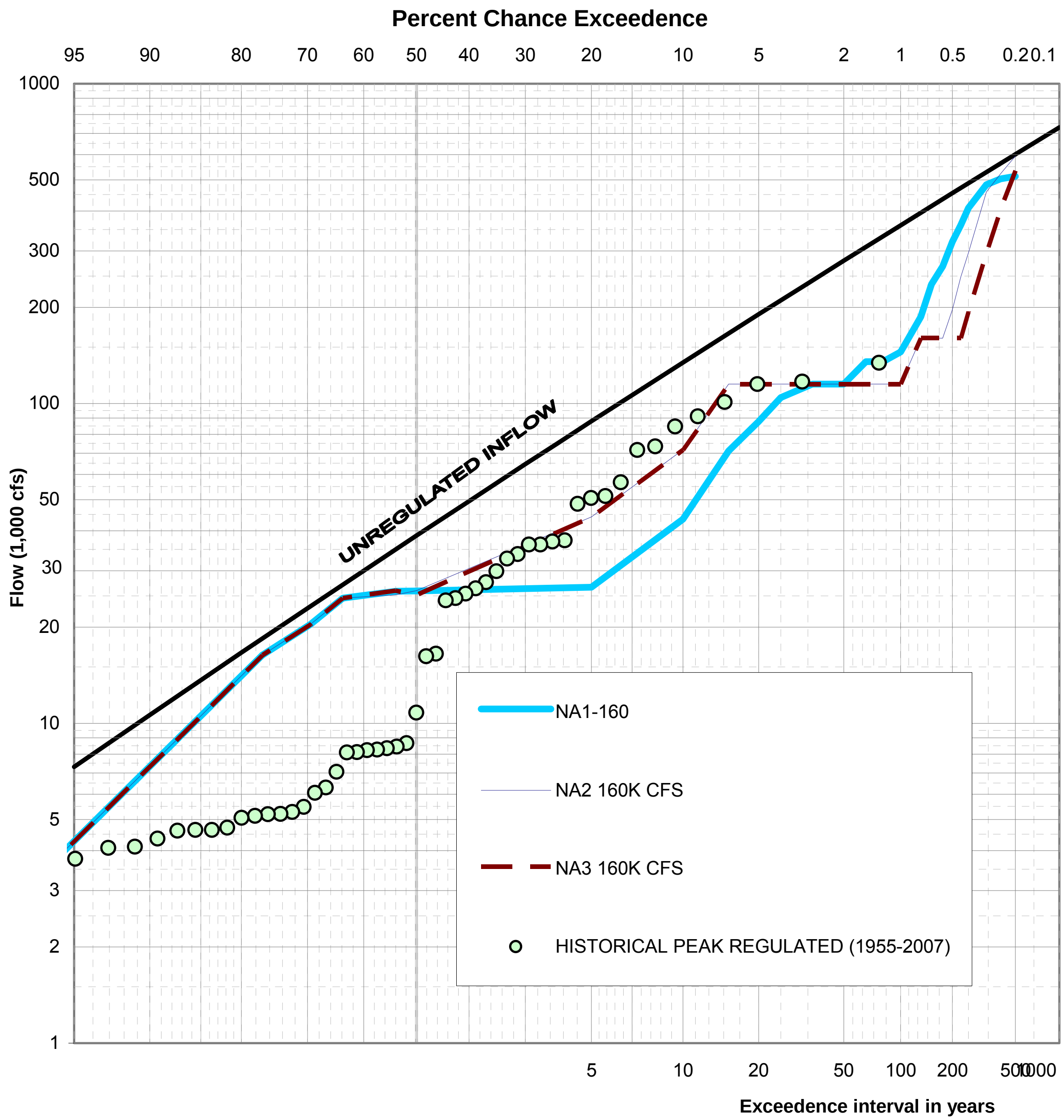

NOTES:

1. Adjusted USGS gage 11446500 to accoiunt for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions
3. No adjustments for outliers.
4. Drainage area: 1,888 sq. mi.
5. Period of record: 1905-2004 (Unregulated).

AMERICAN RIVER WATERSHED PROJECT
COMMON FEATURES GRR

REGULATED FREQUENCY CURVES
FOLSOM DAM
FUTURE PROJECT

FIGURE A-13

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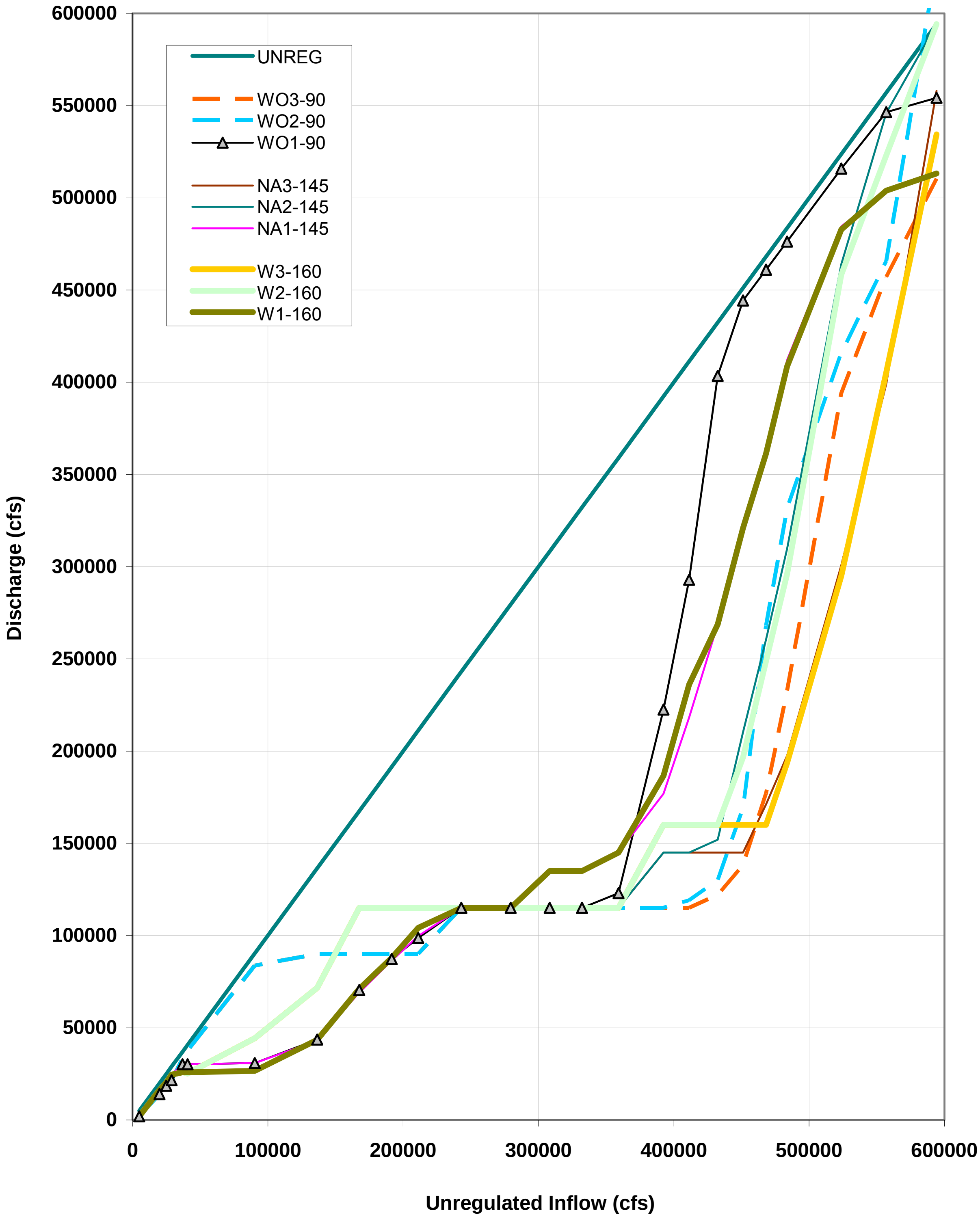

FIGURE A-14: INFLOW-OUTFLOW TRANSFORM – BASE – COMPARISON

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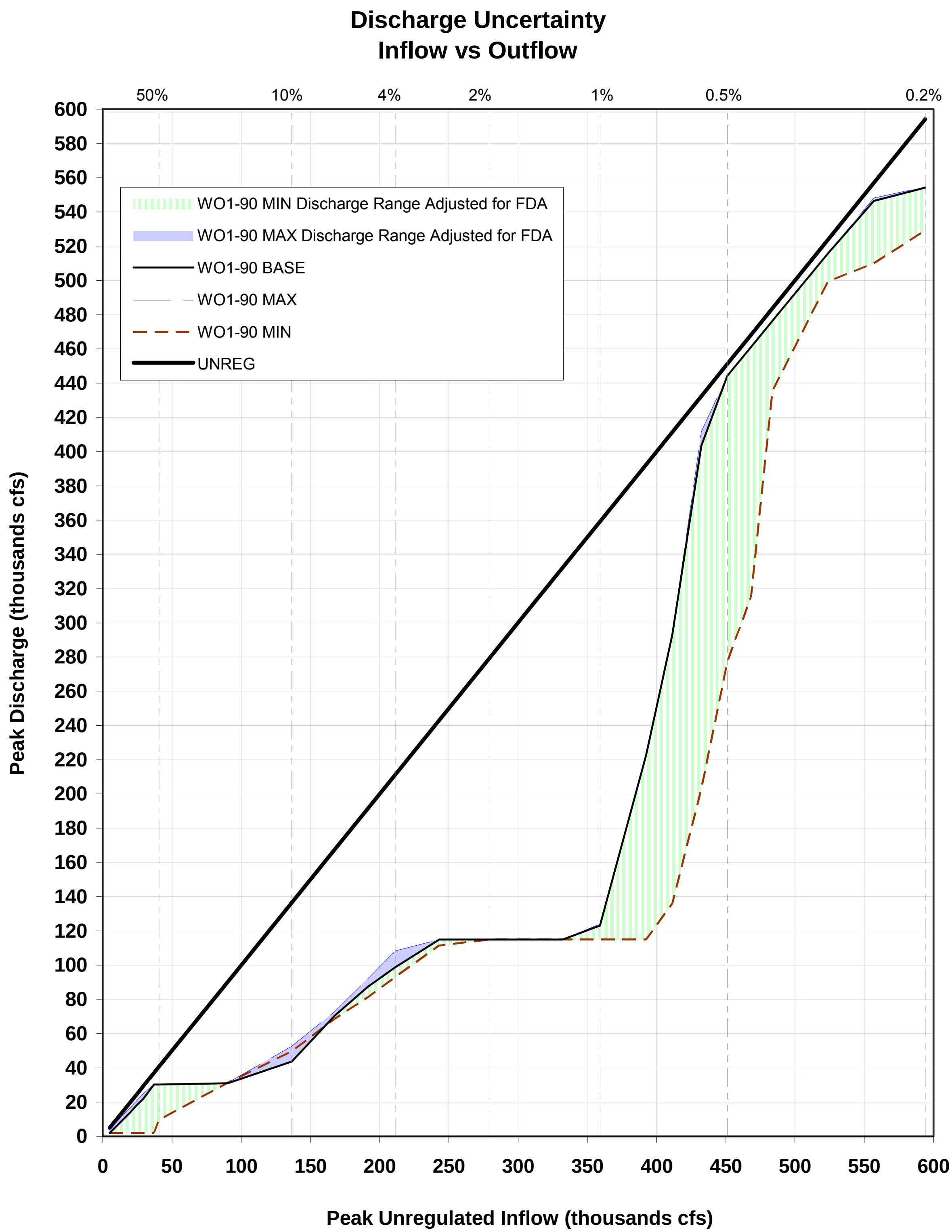

FIGURE A-15: DISCHARGE UNCERTAINTY – WO1 WITHOUT-PROJECT – 115,000 CFS

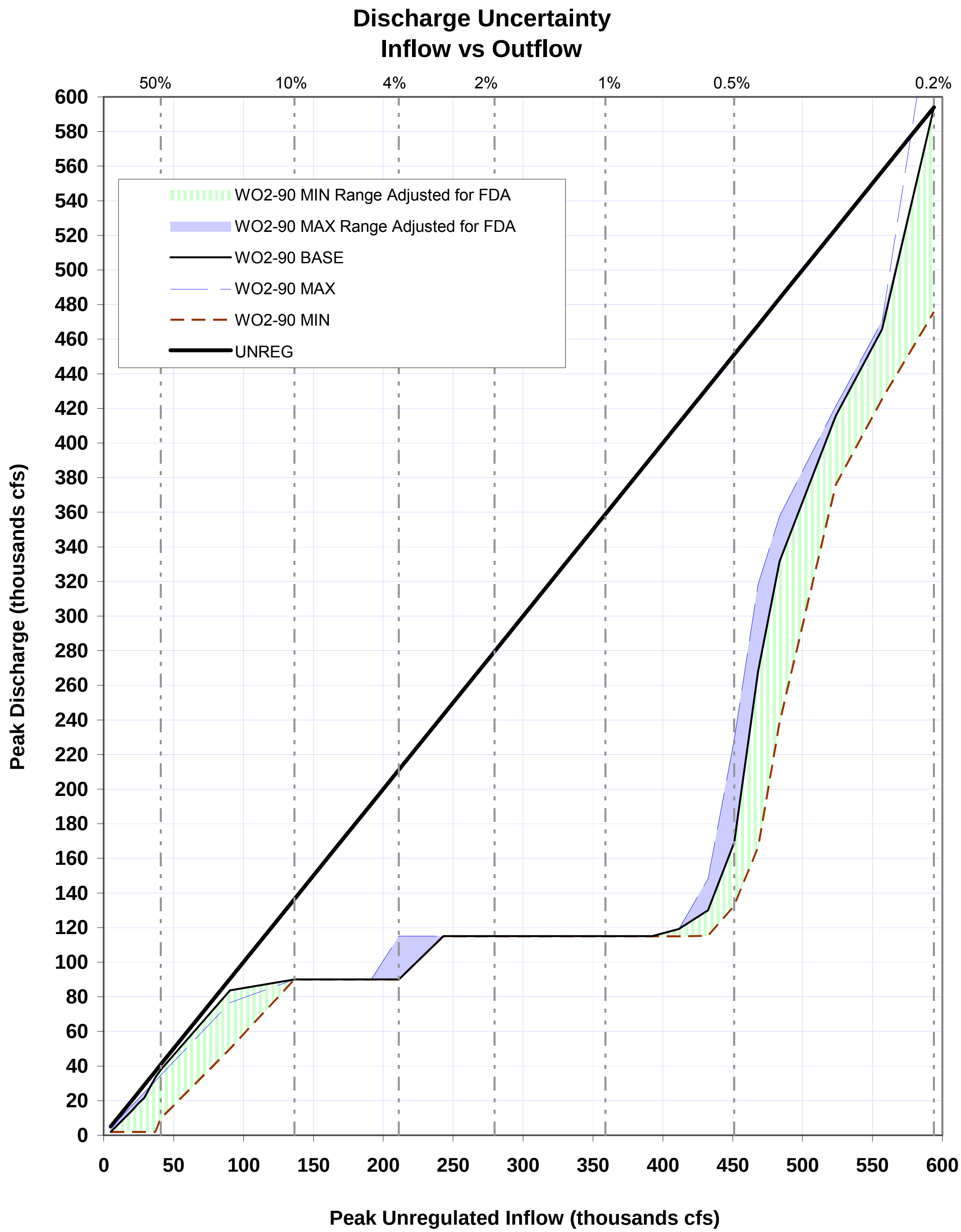

FIGURE A-16: DISCHARGE UNCERTAINTY – WO2 WITHOUT-PROJECT – 115,000 CFS

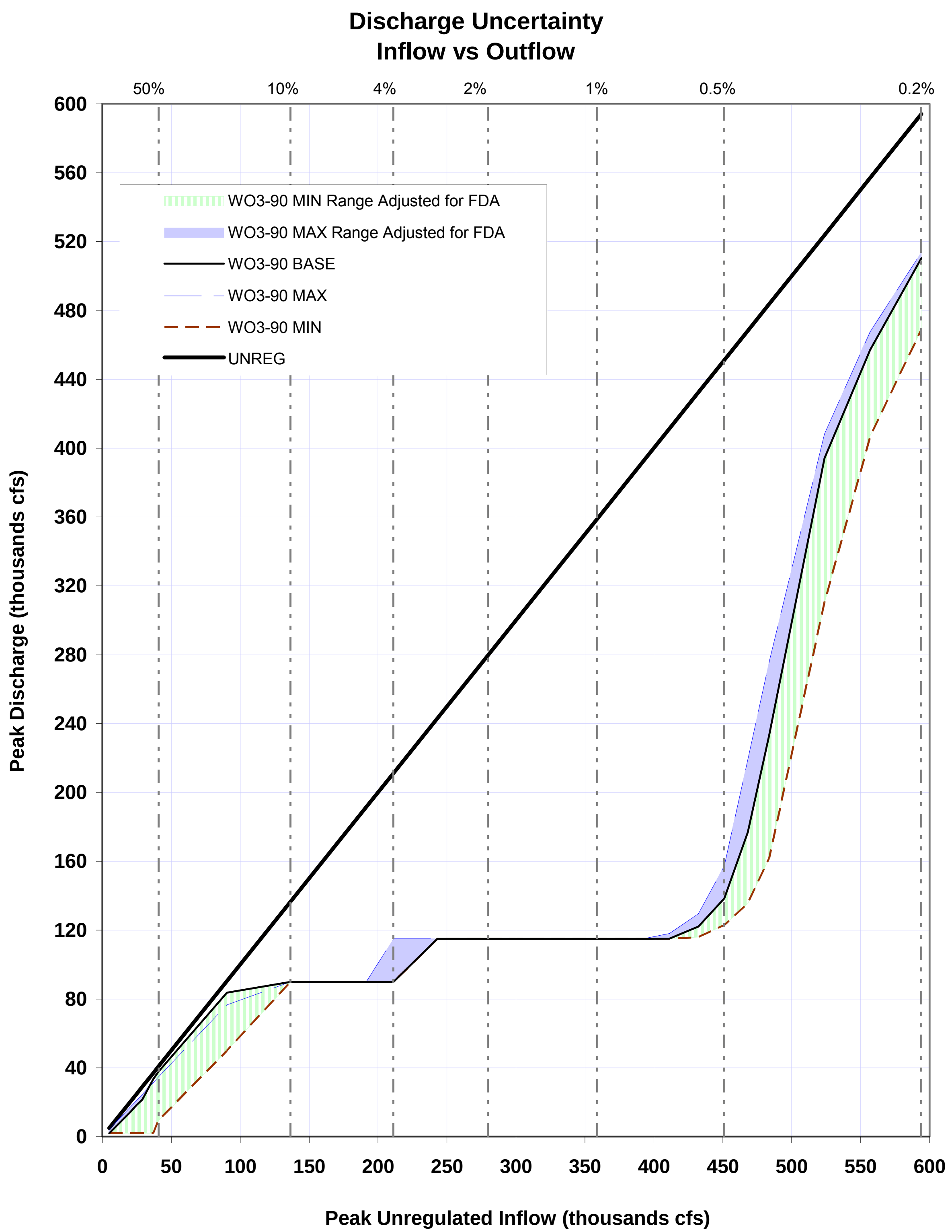

FIGURE A-17: DISCHARGE UNCERTAINTY – WO3 WITHOUT-PROJECT – 115,000 CFS

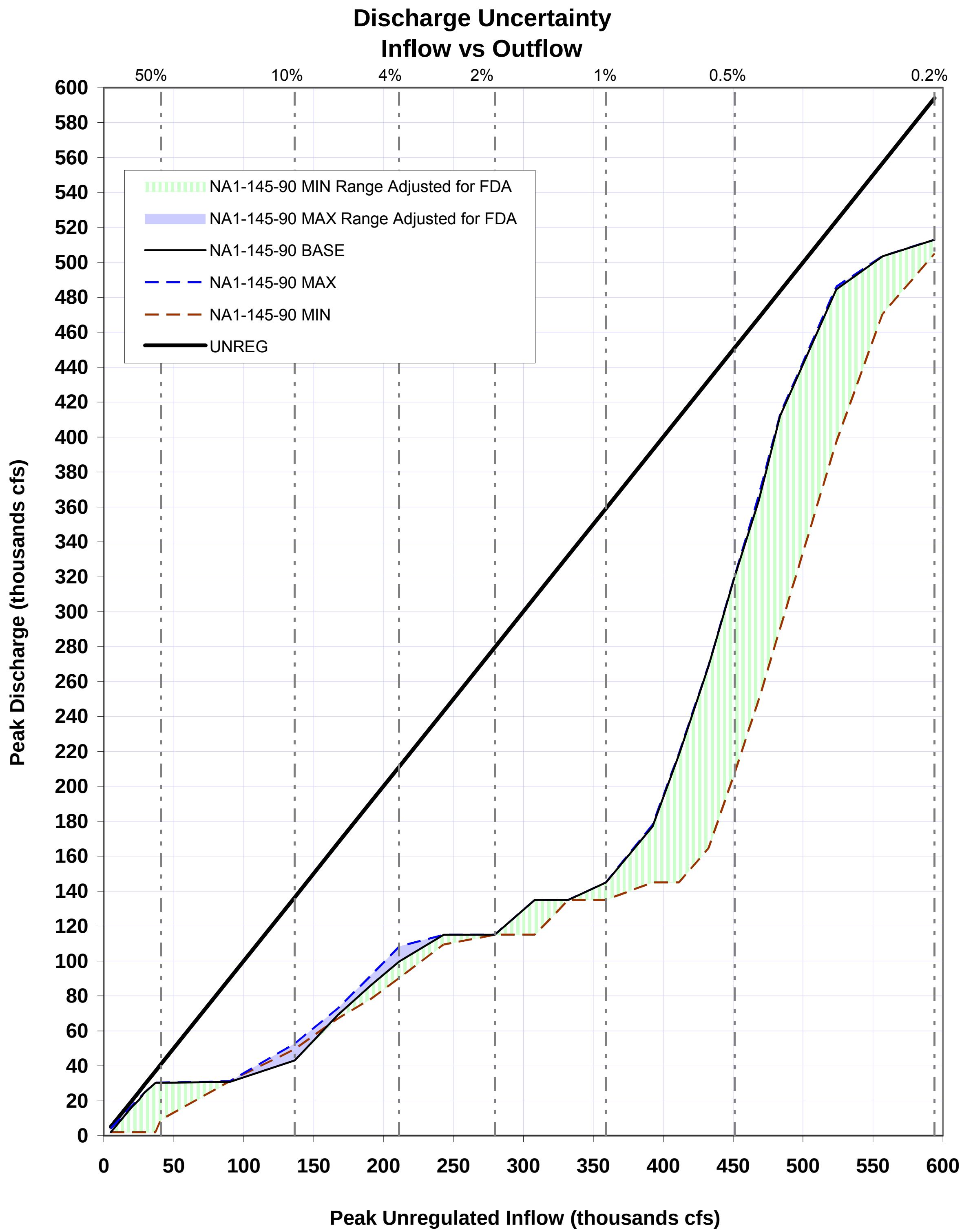

FIGURE A-18: DISCHARGE UNCERTAINTY – NA1 NO ACTION (FUTURE WITHOUT-PROJECT) – 145,000 CFS

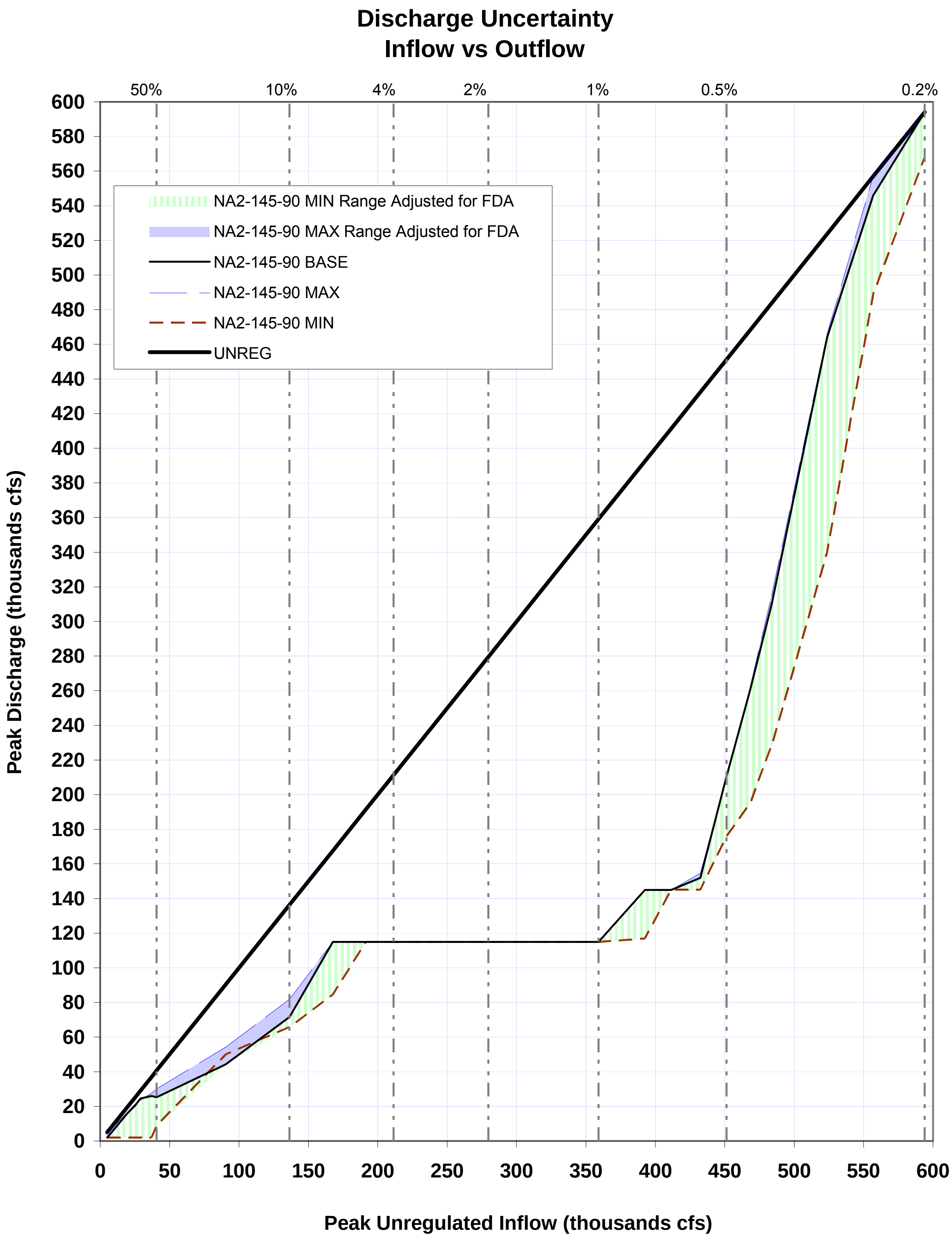

FIGURE A-19: DISCHARGE UNCERTAINTY – NA2 NO ACTION (FUTURE WITHOUT-PROJECT) – 145,000 CFS

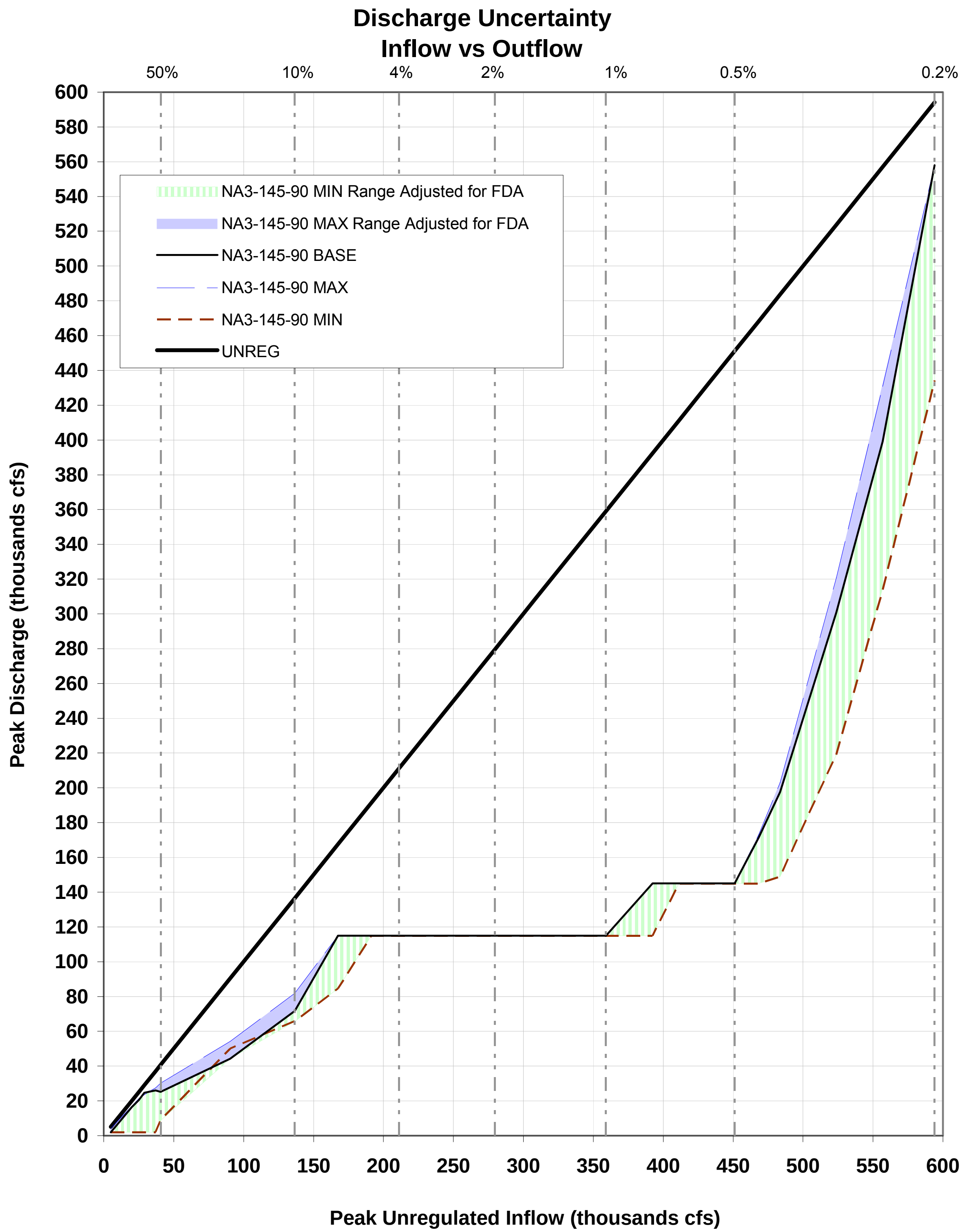

FIGURE A-20: DISCHARGE UNCERTAINTY – NA3 NO ACTION (FUTURE WITHOUT-PROJECT) – 145,000 CFS

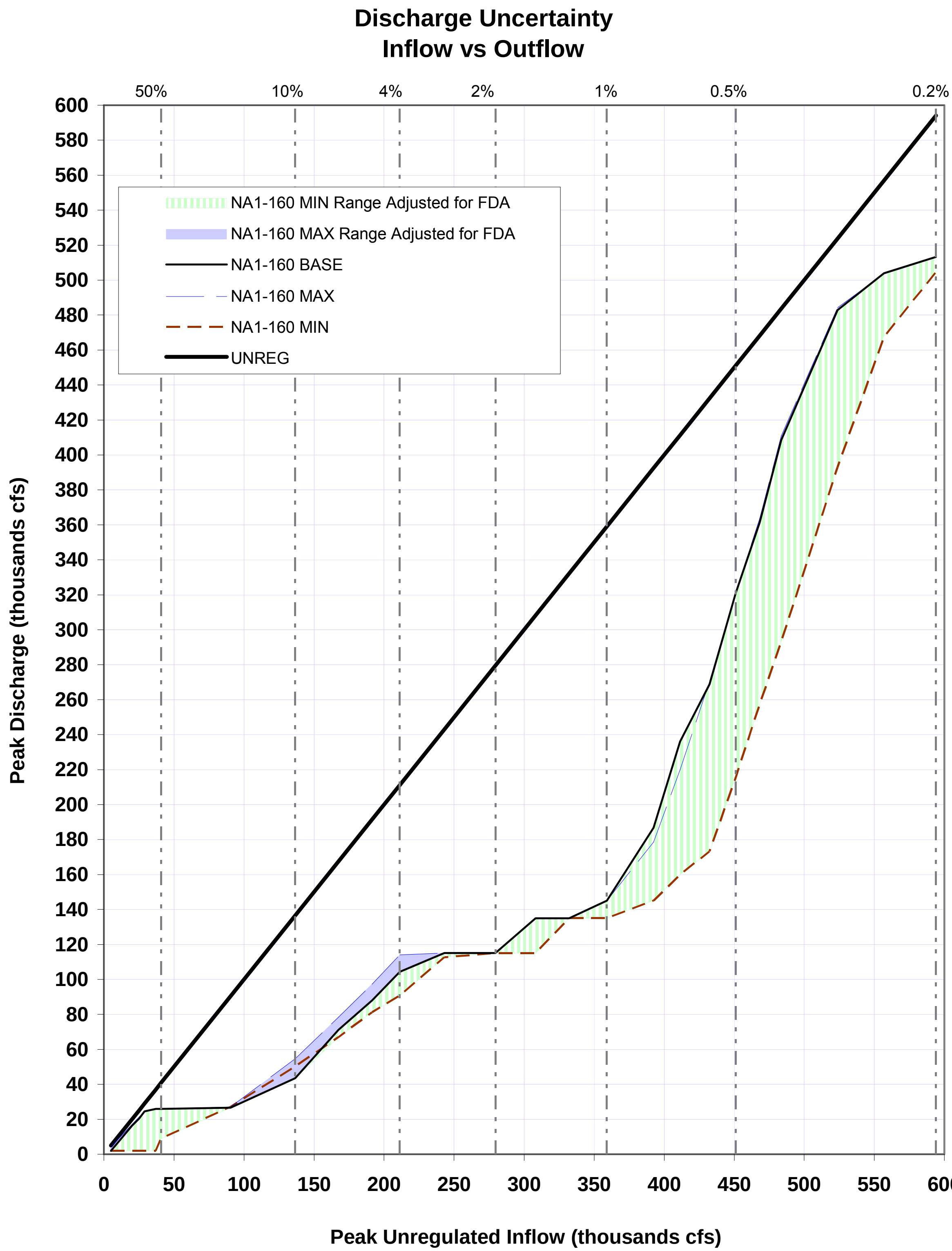

FIGURE A-21: DISCHARGE UNCERTAINTY – W1 WITH-PROJECT – 160,000 CFS

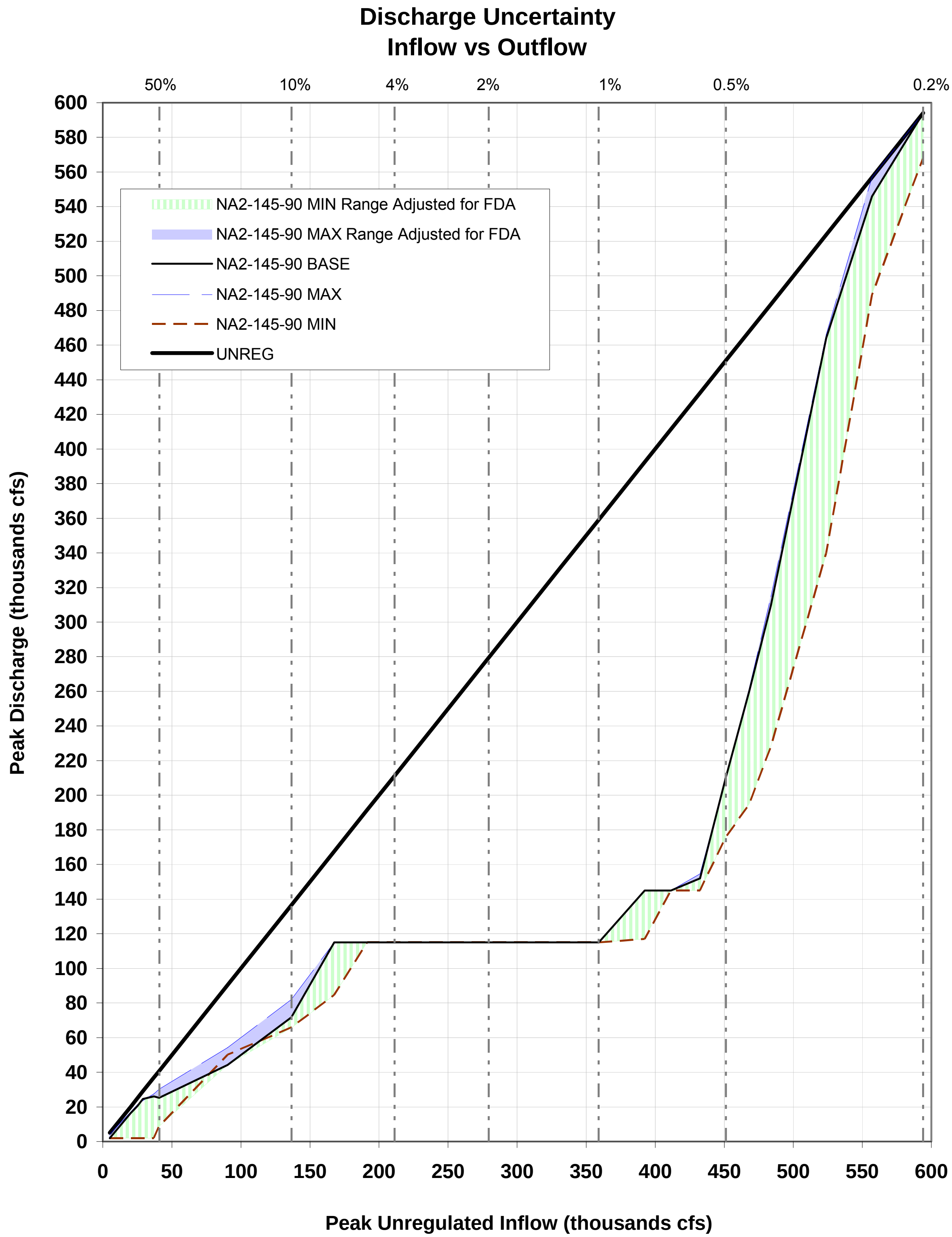

FIGURE A-22: DISCHARGE UNCERTAINTY – W2 WITH-PROJECT – 160,000 CFS

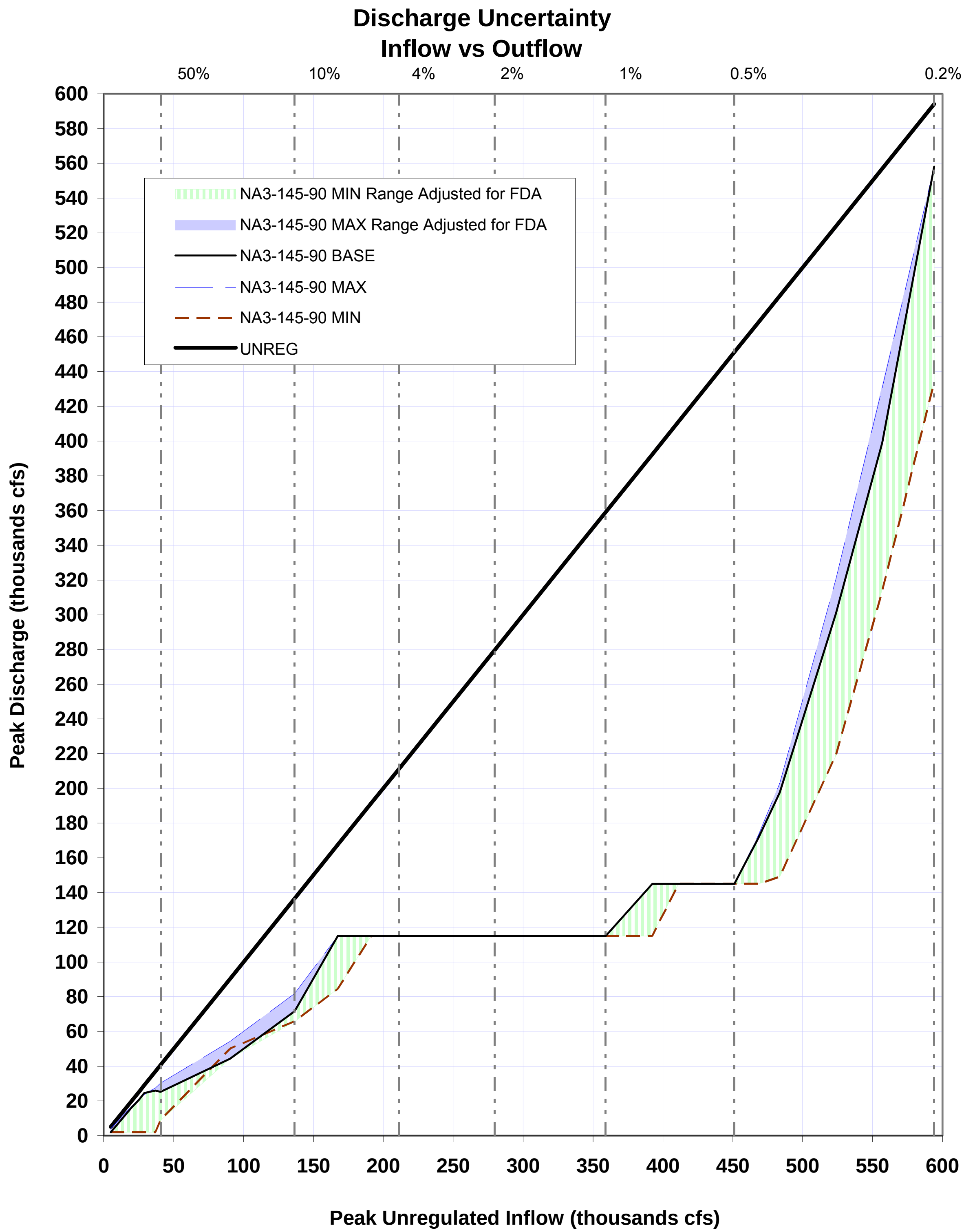

FIGURE A-23: DISCHARGE UNCERTAINTY – W3 WITH-PROJECT – 160,000 CFS

TABLE A-5: W01 BASE (R000_800CF_No Fix_115_FP470_P1_20080914)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.83	391.87	20002	20002	14057	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.49	389.60	25004	25004	18558	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	403.38	395.66	29000	29000	21525	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.75	398.21	37002	37002	30284	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.93	399.44	40722	40722	30295	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.82	494.78	90369	90369	30928	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	429.80	602.86	136522	136522	43692	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	435.17	651.53	167533	167533	70490	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	439.45	691.72	191482	191482	87307	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	442.53	721.38	211227	211227	98760	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	448.80	783.74	243016	243016	115000	0	0.00	0	0	0	23	23	0	0	0	0	0	0
50	457.34	872.50	279485	279485	115000	0	0.00	0	0	0	50	50	0	0	0	0	0	0
65	464.38	948.85	308218	308218	115000	0	0.00	0	0	0	68	68	0	0	0	0	0	0
80	470.37	1016.12	332148	332148	115000	0	0.00	0	0	33	84	84	0	0	0	0	0	0
100	**476.35**	**1085.39**	**359078**	**359078**	**123107**	**0**	**0.00**	**0**	**0**	**52**	**105**	**91**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.79	1078.77	392399	392399	222593	0	0.00	0	0	46	100	85	22	0	14	0	20	0
150	476.38	1085.73	411351	411351	292965	0	0.00	0	0	42	96	81	24	0	18	0	-6	28
175	474.78	1066.96	432395	432395	403445	0	0.00	0	0	27	90	74	27	0	21	10	56	146
200	**476.33**	**1085.18**	**451163**	**451163**	**444310**	**0**	**0.00**	**0**	**0**	**27**	**92**	**76**	**29**	**0**	**23**	**12**	**-6**	**88**
225	476.68	1089.26	468139	468139	461029	0	0.00	0	0	28	94	78	33	0	25	14	74	70
250	476.65	1088.92	483665	483665	476319	0	0.00	0	0	28	93	78	34	0	27	16	68	100
325	477.59	1099.94	523757	523757	515802	0	0.00	0	0	31	97	83	38	0	31	20	55	101
400	478.55	1111.42	556967	556967	546433	0	0.00	0	0	34	100	88	42	0	35	22	38	96
500	479.62	1124.16	594159	594159	554268	0	0.00	0	0	39	109	93	48	0	40	26	31	85

TABLE A-6: W01 MAX (R000_800CF_No Fix_115_FP470_P1_20080914)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.76	419.28	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.90	420.25	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	407.01	421.03	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.22	422.55	37002	37002	30425	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.31	423.19	40722	40722	30425	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	417.33	498.81	90369	90369	31248	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	429.70	601.95	136522	136522	52675	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	435.68	656.20	167533	167533	72904	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	440.67	703.42	191482	191482	92040	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	444.39	739.61	211227	211227	108290	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	450.77	803.74	243016	243016	115000	0	0.00	0	0	0	30	30	0	0	0	0	0	0
50	458.92	889.35	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	465.45	960.66	308218	308218	115000	0	0.00	0	0	0	71	71	0	0	0	0	0	0
80	470.97	1022.96	332148	332148	115000	0	0.00	0	0	36	86	86	0	0	0	0	0	0
100	**476.32**	**1085.03**	**359078**	**359078**	**124034**	**0**	**0.00**	**0**	**0**	**52**	**105**	**91**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.79	1078.80	392399	392399	222320	0	0.00	0	0	46	100	85	22	0	14	0	20	0
150	476.39	1085.81	411351	411351	293316	0	0.00	0	0	42	96	81	24	0	18	0	-6	28
175	474.87	1068.07	432395	432395	411752	0	0.00	0	0	26	90	74	27	0	21	10	57	150
200	**476.37**	**1085.67**	**451163**	**451163**	**444310**	**0**	**0.00**	**0**	**0**	**28**	**93**	**77**	**29**	**0**	**23**	**12**	**-6**	**89**
225	476.67	1089.18	468139	468139	461029	0	0.00	0	0	28	94	78	33	0	25	14	68	70
250	476.66	1089.00	483665	483665	476319	0	0.00	0	0	28	94	78	34	0	27	16	49	99
325	477.74	1101.76	523757	523757	515802	0	0.00	0	0	31	97	83	38	0	32	20	53	95
400	478.68	1112.95	556967	556967	548181	0	0.00	0	0	36	101	88	42	0	36	22	32	98
500	479.76	1125.81	594159	594159	554678	0	0.00	0	0	39	111	93	49	0	40	26	53	81

TABLE A-7: W01 MIN (R000_800CF_No Fix_115_FP470_P1_20080914)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.84	464.05	40722	38674	9546	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	417.31	498.68	90369	84059	31233	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	428.58	592.07	136522	126249	49521	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	434.39	644.26	167533	154598	68674	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	437.99	677.88	191482	176491	81089	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	440.84	705.03	211227	194541	92975	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	445.30	748.65	243016	223601	111456	0	0.00	0	0	0	0	0	0	0	0	0	0	0
50	451.35	809.74	279485	256938	115000	0	0.00	0	0	0	32	32	0	0	0	0	0	0
65	456.85	867.28	308218	283204	115000	0	0.00	0	0	0	50	50	0	0	0	0	0	0
80	461.62	918.59	332148	305080	115000	0	0.00	0	0	0	62	62	0	0	0	0	0	0
100	**466.62**	**973.78**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**14**	**75**	**75**	**0**	**0**	**0**	**0**	**0**	**0**
130	474.76	1066.72	392399	360850	115000	0	0.00	0	0	49	105	94	0	0	0	0	0	0
150	476.90	1091.89	411351	381289	135890	0	0.00	0	0	54	116	104	0	0	0	0	0	0
175	475.90	1080.08	432395	407507	203141	0	0.00	0	0	48	105	92	22	0	13	0	29	0
200	**475.92**	**1080.38**	**451163**	**429875**	**277213**	**0**	**0.00**	**0**	**0**	**41**	**97**	**84**	**25**	**0**	**18**	**0**	**73**	**27**
225	475.85	1079.48	468139	448786	314895	0	0.00	0	0	36	97	83	27	0	21	10	97	0
250	475.00	1069.51	483665	465352	435147	0	0.00	0	0	23	89	75	29	0	23	11	162	58
325	475.98	1080.98	523757	506439	499294	0	0.00	0	0	23	92	77	33	0	27	15	-6	147
400	476.25	1084.24	556967	540033	509928	0	0.00	0	0	27	96	81	39	0	31	19	106	156
500	477.41	1097.86	594159	577133	529188	0	0.00	0	0	31	104	88	45	0	36	22	70	132

TABLE A-8: W02 BASE (R060_800FM_No Fix_115_FP470_P1_20080908)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.83	391.87	20002	20002	14057	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.49	389.60	25004	25004	18558	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	403.38	395.66	29000	29000	21525	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.67	397.61	37002	37002	33505	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.53	396.67	40722	40722	37708	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	403.99	399.85	90369	90369	83680	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	408.97	435.18	136522	136522	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	415.21	482.19	167533	167533	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	421.58	532.97	191482	191482	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	427.80	585.26	211227	211227	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	433.71	638.02	243016	243016	115000	0	0.00	0	0	0	29	29	0	0	0	0	0	0
50	443.02	726.21	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	449.41	789.86	308218	308218	115000	0	0.00	0	0	0	68	68	0	0	0	0	0	0
80	454.76	845.21	332148	332148	115000	0	0.00	0	0	0	81	81	0	0	0	0	0	0
100	**461.00**	**911.80**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**102**	**102**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.81	987.06	392399	392399	115000	0	0.00	0	0	23	127	127	0	0	0	0	0	0
150	472.83	1044.29	411351	411351	115000	0	0.00	0	0	43	138	126	0	0	0	0	0	0
175	476.38	1085.74	432395	432395	129972	0	0.00	0	0	54	147	133	0	0	0	0	0	0
200	**476.65**	**1088.93**	**451163**	**451163**	**169173**	**0**	**0.00**	**0**	**0**	**152**	**53**	**14**	**20**	**0**	**0**	**0**	**3**	**0**
225	474.79	1067.11	468139	468139	268061	0	0.00	0	0	80	56	15	24	0	17	0	55	0
250	475.23	1072.29	483665	483665	331691	0	0.00	0	0	62	58	14	25	0	19	8	-5	45
325	478.02	1105.13	523757	523757	415711	0	0.00	0	0	62	62	15	30	0	24	13	-6	80
400	479.76	1125.81	556967	556967	465830	0	0.00	0	0	63	65	15	34	0	28	16	-5	80
500	480.97	1140.34	594159	594159	627077	25	0.47	0	0	47	68	13	40	0	32	20	-2	77

TABLE A-9: W02 MAX (R060_800FM_No Fix_115_FP470_P1_20080908)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.76	419.28	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.90	420.25	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	407.01	421.03	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.22	422.55	37002	37002	31387	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.31	423.19	40722	40722	34542	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	408.59	432.46	90369	90369	76656	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	412.41	460.77	136522	136522	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	418.11	505.04	167533	167533	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	423.57	549.39	191482	191482	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	428.52	591.54	211227	211227	115000	0	0.00	0	0	0	10	10	0	0	0	0	0	0
35	434.66	646.79	243016	243016	115000	0	0.00	0	0	0	32	32	0	0	0	0	0	0
50	443.71	732.89	279485	279485	115000	0	0.00	0	0	0	55	55	0	0	0	0	0	0
65	450.81	804.21	308218	308218	115000	0	0.00	0	0	0	72	72	0	0	0	0	0	0
80	456.24	860.80	332148	332148	115000	0	0.00	0	0	0	86	86	0	0	0	0	0	0
100	**463.05**	**934.22**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**113**	**113**	**0**	**0**	**0**	**0**	**0**	**0**
130	470.95	1022.74	392399	392399	115000	0	0.00	0	0	37	135	135	0	0	0	0	0	0
150	475.46	1074.96	411351	411351	118897	0	0.00	0	0	51	146	130	0	0	0	0	0	0
175	476.56	1087.80	432395	432395	148180	0	0.00	0	0	53	149	132	0	0	0	0	0	0
200	**474.94**	**1068.80**	**451163**	**451163**	**228405**	**0**	**0.00**	**0**	**0**	**99**	**57**	**17**	**22**	**0**	**15**	**0**	**53**	**0**
225	474.96	1069.02	468139	468139	318472	0	0.00	0	0	63	59	17	26	0	18	6	64	38
250	476.00	1081.29	483665	483665	357755	0	0.00	0	0	59	60	16	26	0	20	9	-5	57
325	478.22	1107.42	523757	523757	421382	0	0.00	0	0	63	64	16	31	0	25	14	-6	80
400	479.93	1127.86	556967	556967	470310	0	0.00	0	0	62	66	15	35	0	29	17	-5	77
500	481.08	1141.63	594159	594159	663803	33	0.58	0	0	47	70	14	41	0	33	21	-2	78

TABLE A-10: W02 MIN (R060_800FM_No Fix_115_FP470_P1_20080908)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.84	464.05	40722	38674	9546	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	415.33	483.15	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	418.32	506.69	136522	126249	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	421.73	534.18	167533	154598	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	425.96	569.46	191482	176491	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	429.85	603.33	211227	194541	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	434.59	646.18	243016	223601	115000	0	0.00	0	0	0	22	22	0	0	0	0	0	0
50	441.93	715.57	279485	256938	115000	0	0.00	0	0	0	46	46	0	0	0	0	0	0
65	448.57	781.34	308218	283204	115000	0	0.00	0	0	0	63	63	0	0	0	0	0	0
80	453.72	834.34	332148	305080	115000	0	0.00	0	0	0	79	79	0	0	0	0	0	0
100	**460.26**	**903.83**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**100**	**100**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.01	978.09	392399	360850	115000	0	0.00	0	0	16	127	127	0	0	0	0	0	0
150	469.43	1005.36	411351	381289	115000	0	0.00	0	0	32	134	134	0	0	0	0	0	0
175	474.36	1062.07	432395	407507	115124	0	0.00	0	0	50	147	130	0	0	0	0	0	0
200	**476.58**	**1088.06**	**451163**	**429875**	**132562**	**0**	**0.00**	**0**	**0**	**55**	**152**	**136**	**0**	**0**	**0**	**0**	**0**	**0**
225	476.63	1088.71	468139	448786	166144	0	0.00	0	0	155	55	15	20	0	0	0	3	0
250	474.88	1068.11	483665	465352	238763	0	0.00	0	0	95	57	16	24	0	17	0	50	15
325	476.43	1086.35	523757	506439	375796	0	0.00	0	0	55	61	15	28	0	22	10	-6	68
400	478.35	1108.97	556967	540033	425258	0	0.00	0	0	57	64	15	33	0	26	14	-1	84
500	480.11	1129.97	594159	577133	475823	0	0.00	0	0	47	68	15	39	0	30	17	-2	83

TABLE A-11: W03 BASE (R060_800DR3.5e_115_FP470_P1_20080907)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.83	391.87	20002	20002	14057	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.49	389.60	25004	25004	18558	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	403.38	395.66	29000	29000	21525	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.67	397.61	37002	37002	33505	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.53	396.67	40722	40722	37708	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	403.99	399.85	90369	90369	83680	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	408.97	435.18	136522	136522	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	415.21	482.19	167533	167533	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	421.58	532.97	191482	191482	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	427.80	585.26	211227	211227	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	433.71	638.02	243016	243016	115000	0	0.00	0	0	0	29	29	0	0	0	0	0	0
50	443.02	726.21	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	449.41	789.86	308218	308218	115000	0	0.00	0	0	0	68	68	0	0	0	0	0	0
80	454.76	845.21	332148	332148	115000	0	0.00	0	0	0	81	81	0	0	0	0	0	0
100	**461.00**	**911.80**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**102**	**102**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.81	987.06	392399	392399	115000	0	0.00	0	0	23	127	127	0	0	0	0	0	0
150	472.81	1044.11	411351	411351	115088	0	0.00	0	0	43	138	126	0	0	0	0	0	0
175	476.78	1090.42	432395	432395	122131	0	0.00	0	0	55	149	135	0	0	0	0	0	0
200	**478.67**	**1112.80**	**451163**	**451163**	**138359**	**0**	**0.00**	**0**	**0**	**60**	**154**	**140**	**0**	**0**	**0**	**0**	**0**	**0**
225	478.38	1109.31	468139	468139	176952	0	0.00	0	0	162	56	15	24	0	0	0	15	0
250	477.27	1096.17	483665	483665	232803	0	0.00	0	0	150	58	14	25	0	18	0	34	0
325	477.72	1101.50	523757	523757	394043	0	0.00	0	0	72	62	15	30	0	24	13	-6	75
400	479.49	1122.65	556967	556967	457102	0	0.00	0	0	76	65	15	34	0	28	16	-5	74
500	481.31	1144.50	594159	594159	510279	0	0.00	0	0	77	68	14	40	0	32	20	-2	75
PMF	486.00	1201.47	905770	905770	1105372	214	2.00	0	0	131	134	18	93	0	61	49	247	146

TABLE A-12: W03 MAX (R060_800DR3.5e_115_FP470_P1_20080907)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.76	419.28	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.90	420.25	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	407.01	421.03	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.22	422.55	37002	37002	31387	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.31	423.19	40722	40722	34542	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	408.59	432.46	90369	90369	76656	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	412.41	460.77	136522	136522	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	418.11	505.04	167533	167533	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	423.57	549.39	191482	191482	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	428.52	591.54	211227	211227	115000	0	0.00	0	0	0	10	10	0	0	0	0	0	0
35	434.66	646.79	243016	243016	115000	0	0.00	0	0	0	32	32	0	0	0	0	0	0
50	443.71	732.89	279485	279485	115000	0	0.00	0	0	0	55	55	0	0	0	0	0	0
65	450.81	804.21	308218	308218	115000	0	0.00	0	0	0	72	72	0	0	0	0	0	0
80	456.24	860.80	332148	332148	115000	0	0.00	0	0	0	86	86	0	0	0	0	0	0
100	**463.05**	**934.22**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**113**	**113**	**0**	**0**	**0**	**0**	**0**	**0**
130	470.95	1022.74	392399	392399	115000	0	0.00	0	0	37	135	135	0	0	0	0	0	0
150	475.37	1073.85	411351	411351	118010	0	0.00	0	0	51	146	130	0	0	0	0	0	0
175	477.72	1101.61	432395	432395	129653	0	0.00	0	0	57	152	135	0	0	0	0	0	0
200	**478.72**	**1113.42**	**451163**	**451163**	**157288**	**0**	**0.00**	**0**	**0**	**60**	**156**	**138**	**0**	**0**	**0**	**0**	**0**	**0**
225	477.50	1098.98	468139	468139	219082	0	0.00	0	0	151	59	18	25	0	17	0	24	0
250	476.79	1090.52	483665	483665	275466	0	0.00	0	0	105	60	16	26	0	20	0	53	7
325	478.10	1106.01	523757	523757	408380	0	0.00	0	0	73	64	16	31	0	25	13	74	61
400	479.87	1127.11	556967	556967	467639	0	0.00	0	0	77	66	15	35	0	29	16	71	59
500	481.43	1147.07	594159	594159	513811	0	0.00	0	0	78	70	15	41	0	33	21	-2	72

TABLE A-13: W03 MIN (R060_800DR3.5e_115_FP470_P1_20080907)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.84	464.05	40722	38674	9546	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	415.33	483.15	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	418.32	506.69	136522	126249	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	421.73	534.18	167533	154598	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	425.96	569.46	191482	176491	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	429.85	603.33	211227	194541	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	434.59	646.18	243016	223601	115000	0	0.00	0	0	0	22	22	0	0	0	0	0	0
50	441.93	715.57	279485	256938	115000	0	0.00	0	0	0	46	46	0	0	0	0	0	0
65	448.57	781.34	308218	283204	115000	0	0.00	0	0	0	63	63	0	0	0	0	0	0
80	453.72	834.34	332148	305080	115000	0	0.00	0	0	0	79	79	0	0	0	0	0	0
100	**460.26**	**903.83**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**100**	**100**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.01	978.09	392399	360850	115000	0	0.00	0	0	16	127	127	0	0	0	0	0	0
150	469.43	1005.36	411351	381289	115000	0	0.00	0	0	32	134	134	0	0	0	0	0	0
175	474.21	1060.36	432395	407507	115938	0	0.00	0	0	49	146	128	0	0	0	0	0	0
200	**477.18**	**1095.22**	**451163**	**429875**	**122877**	**0**	**0.00**	**0**	**0**	**57**	**154**	**138**	**0**	**0**	**0**	**0**	**0**	**0**
225	478.80	1114.32	468139	448786	135427	0	0.00	0	0	61	158	143	0	0	0	0	0	0
250	479.05	1117.37	483665	465352	161583	0	0.00	0	0	171	57	16	23	0	0	0	0	0
325	476.54	1087.58	523757	506439	310409	0	0.00	0	0	83	61	15	28	0	22	9	55	38
400	478.05	1105.51	556967	540033	406670	0	0.00	0	0	64	64	15	33	0	26	14	-1	78
500	479.89	1127.38	594159	577133	468932	0	0.00	0	0	68	68	16	39	0	30	16	77	70

TABLE A-14: NA1-145 BASE (R000_800CF_No Fix_145_FP470_P1_20080919)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.17	394.19	37002	37002	30237	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	402.43	389.16	40722	40722	30183	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	415.74	486.34	90369	90369	30848	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	429.13	596.95	136522	136522	43127	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	434.65	646.67	167533	167533	69092	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	439.01	687.48	191482	191482	86500	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	442.69	722.96	211227	211227	99738	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	448.44	780.03	243016	243016	115000	0	0.00	0	0	0	22	22	0	0	0	0	0	0
50	457.01	868.92	279485	279485	115000	0	0.00	0	0	0	49	49	0	0	0	0	0	0
65	461.09	912.80	308218	308218	135000	0	0.00	0	0	0	61	61	0	0	0	0	0	0
80	466.44	971.77	332148	332148	135000	0	0.00	0	0	9	74	74	0	0	0	0	0	0
100	**470.81**	**1021.10**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**39**	**92**	**92**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.31	1073.15	392399	392399	177012	0	0.00	0	0	49	105	105	20	1	0	0	14	0
150	475.74	1078.26	411351	411351	218286	0	0.00	0	0	49	106	106	23	0	16	0	18	0
175	476.07	1082.12	432395	432395	268700	0	0.00	0	0	47	104	103	25	0	19	0	-6	16
200	**476.40**	**1086.00**	**451163**	**451163**	**320142**	**0**	**0.00**	**0**	**0**	**45**	**104**	**102**	**28**	**0**	**21**	**11**	**65**	**42**
225	476.58	1088.08	468139	468139	363164	0	0.00	0	0	41	103	102	31	0	24	12	48	42
250	476.67	1089.18	483665	483665	412114	0	0.00	0	0	36	101	99	32	0	26	14	-5	47
325	477.17	1095.00	523757	523757	484550	0	0.00	0	0	35	102	101	37	0	30	18	56	49
400	477.83	1102.80	556967	556967	503557	0	0.00	0	0	38	107	107	41	0	35	21	37	46
500	479.01	1116.85	594159	594159	512982	0	0.00	0	0	42	129	115	47	0	39	25	30	52

TABLE A-15: NA1-145 MAX (R000_800CF_No Fix_145_FP470_P1_20080919)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	30423	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	30424	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	417.33	498.81	90369	90369	31248	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	429.70	601.95	136522	136522	52675	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	435.68	656.20	167533	167533	72904	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	440.67	703.42	191482	191482	92040	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	444.39	739.61	211227	211227	108290	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	450.77	803.74	243016	243016	115000	0	0.00	0	0	0	30	30	0	0	0	0	0	0
50	458.92	889.35	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	462.29	925.84	308218	308218	135000	0	0.00	0	0	0	65	65	0	0	0	0	0	0
80	467.20	980.23	332148	332148	135000	0	0.00	0	0	16	78	78	0	0	0	0	0	0
100	**471.07**	**1024.11**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**40**	**92**	**92**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.32	1073.28	392399	392399	178071	0	0.00	0	0	48	105	105	20	0	0	0	13	0
150	475.75	1078.35	411351	411351	218943	0	0.00	0	0	49	106	106	23	0	17	0	14	0
175	476.08	1082.17	432395	432395	269028	0	0.00	0	0	47	104	103	25	0	19	0	-6	15
200	**476.41**	**1086.03**	**451163**	**451163**	**320618**	**0**	**0.00**	**0**	**0**	**45**	**104**	**102**	**28**	**0**	**22**	**11**	**56**	**42**
225	476.59	1088.23	468139	468139	366078	0	0.00	0	0	40	104	102	31	0	24	12	50	42
250	476.67	1089.19	483665	483665	413033	0	0.00	0	0	36	102	99	32	0	26	14	-5	46
325	477.19	1095.24	523757	523757	485904	0	0.00	0	0	35	102	101	37	0	31	18	45	47
400	477.84	1102.98	556967	556967	503676	0	0.00	0	0	39	108	108	41	0	35	21	36	47
500	479.02	1116.98	594159	594159	513077	0	0.00	0	0	42	126	115	48	0	39	25	44	52

TABLE A-16: NA1-145 MIN (R000_800CF_No Fix_145_FP470_P1_20080919)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	417.13	497.21	90369	84059	31233	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	428.58	592.07	136522	126249	49521	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	433.77	638.60	167533	154598	67040	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	437.47	672.98	191482	176491	78462	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	440.35	700.34	211227	194541	90191	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	444.94	745.01	243016	223601	109419	0	0.00	0	0	0	0	0	0	0	0	0	0	0
50	451.06	806.80	279485	256938	115000	0	0.00	0	0	0	31	31	0	0	0	0	0	0
65	456.61	864.68	308218	283204	115000	0	0.00	0	0	0	48	48	0	0	0	0	0	0
80	458.74	887.48	332148	305080	135000	0	0.00	0	0	0	56	56	0	0	0	0	0	0
100	**463.26**	**936.53**	**359078**	**329825**	**135000**	**0**	**0.00**	**0**	**0**	**0**	**68**	**68**	**0**	**0**	**0**	**0**	**0**	**0**
130	468.87	998.98	392399	360850	145000	0	0.00	0	0	26	84	84	0	0	0	0	0	0
150	472.83	1044.26	411351	381289	145000	0	0.00	0	0	47	103	103	0	0	0	0	0	0
175	475.66	1077.28	432395	407507	164525	0	0.00	0	0	52	118	118	14	1	0	0	2	0
200	**475.58**	**1076.33**	**451163**	**429875**	**207706**	**0**	**0.00**	**0**	**0**	**50**	**117**	**117**	**23**	**0**	**17**	**0**	**4**	**0**
225	475.81	1079.03	468139	448786	249188	0	0.00	0	0	48	118	118	25	0	19	0	-5	5
250	476.10	1082.45	483665	465352	290568	0	0.00	0	0	45	106	106	27	0	21	0	-5	21
325	476.63	1088.66	523757	506439	397225	0	0.00	0	0	36	105	104	32	0	26	13	-6	92
400	476.97	1092.73	556967	540033	470504	0	0.00	0	0	32	104	103	37	0	30	18	-1	63
500	478.01	1105.02	594159	577133	505120	0	0.00	0	0	36	121	110	43	0	34	21	-2	50

TABLE A-17: NA2-145 BASE (R060_800FM_No Fix_145_FP466_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.59	397.11	37002	37002	26005	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.18	394.30	40722	40722	25215	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	413.74	470.92	90369	90369	44261	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	421.65	533.58	136522	136522	71655	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	424.92	560.66	167533	167533	115000	0	0.00	0	0	0	26	26	0	0	0	0	0	0
20	428.02	587.24	191482	191482	115000	0	0.00	0	0	0	36	36	0	0	0	0	0	0
25	431.43	617.37	211227	211227	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
35	437.15	669.98	243016	243016	115000	0	0.00	0	0	0	57	57	0	0	0	0	0	0
50	442.97	725.68	279485	279485	115000	0	0.00	0	0	0	75	75	0	0	0	0	0	0
65	449.11	786.85	308218	308218	115000	0	0.00	0	0	0	103	103	0	0	0	0	0	0
80	453.74	834.47	332148	332148	115000	0	0.00	0	0	0	127	127	0	0	0	0	0	0
100	**460.46**	**906.02**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**137**	**137**	**0**	**0**	**0**	**0**	**0**	**0**
130	461.49	917.22	392399	392399	145000	0	0.00	0	0	0	144	125	0	0	0	0	0	0
150	466.34	970.57	411351	411351	145000	0	0.00	0	0	11	153	134	0	0	0	0	0	0
175	469.94	1011.13	432395	432395	151924	0	0.00	0	0	33	160	142	0	0	0	0	0	0
200	**470.02**	**1012.12**	**451163**	**451163**	**210332**	**0**	**0.00**	**0**	**0**	**25**	**160**	**142**	**21**	**0**	**15**	**0**	**0**	**0**
225	470.31	1015.33	468139	468139	260498	0	0.00	0	0	21	160	143	25	0	18	0	-5	19
250	470.65	1019.31	483665	483665	309673	0	0.00	0	0	18	160	144	26	0	20	9	-5	35
325	471.23	1025.87	523757	523757	464074	0	0.00	0	0	13	183	144	30	0	24	13	-6	111
400	471.61	1030.22	556967	556967	545951	0	0.00	0	0	12	189	146	34	0	28	16	-5	176
500	472.08	1035.62	594159	594159	594159	0	0.00	0	0	14	196	150	40	0	32	20	-2	153
PMF	477.51	1099.03	905770	905770	812199	0	0.00	0	0	57	255	129	85	0	55	43	0	146

TABLE A-18: NA2-145 MAX (R060_800FM_No Fix_145_FP466_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	23588	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	27464	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	30225	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	414.02	473.00	90369	90369	54221	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	423.56	549.32	136522	136522	81913	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	427.18	579.96	167533	167533	115000	0	0.00	0	0	0	28	28	0	0	0	0	0	0
20	430.25	606.84	191482	191482	115000	0	0.00	0	0	0	38	38	0	0	0	0	0	0
25	433.17	633.13	211227	211227	115000	0	0.00	0	0	0	47	47	0	0	0	0	0	0
35	438.57	683.35	243016	243016	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
50	444.11	736.88	279485	279485	115000	0	0.00	0	0	0	77	77	0	0	0	0	0	0
65	450.07	796.61	308218	308218	115000	0	0.00	0	0	0	105	105	0	0	0	0	0	0
80	456.02	858.42	332148	332148	115000	0	0.00	0	0	0	128	128	0	0	0	0	0	0
100	**461.90**	**921.63**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**139**	**139**	**0**	**0**	**0**	**0**	**0**	**0**
130	460.99	911.73	392399	392399	145000	0	0.00	0	0	0	142	124	0	0	0	0	0	0
150	466.51	972.53	411351	411351	145000	0	0.00	0	0	14	153	135	0	0	0	0	0	0
175	469.98	1011.59	432395	432395	154629	0	0.00	0	0	32	178	142	0	0	0	0	0	0
200	**470.03**	**1012.15**	**451163**	**451163**	**209579**	**0**	**0.00**	**0**	**0**	**25**	**180**	**142**	**21**	**0**	**15**	**0**	**0**	**0**
225	470.31	1015.44	468139	468139	262069	0	0.00	0	0	21	182	143	25	0	18	0	-5	10
250	470.71	1019.97	483665	483665	314605	0	0.00	0	0	18	183	143	26	0	20	9	-5	25
325	471.20	1025.58	523757	523757	466105	0	0.00	0	0	13	187	144	31	0	25	14	-6	100
400	471.75	1031.94	556967	556967	556967	0	0.00	0	0	13	193	145	35	0	29	17	-5	143
500	471.90	1033.55	594159	594159	594159	0	0.00	0	0	13	201	150	40	0	32	21	-2	140

TABLE A-19: NA2-145 MIN (R060_800FM_No Fix_145_FP466_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.16	489.65	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	424.05	553.36	136522	126249	65753	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	428.10	587.92	167533	154598	84559	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	428.64	592.60	191482	176491	115000	0	0.00	0	0	0	25	25	0	0	0	0	0	0
25	431.48	617.85	211227	194541	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
35	436.67	665.48	243016	223601	115000	0	0.00	0	0	0	48	48	0	0	0	0	0	0
50	442.99	725.90	279485	256938	115000	0	0.00	0	0	0	62	62	0	0	0	0	0	0
65	448.29	778.56	308218	283204	115000	0	0.00	0	0	0	81	81	0	0	0	0	0	0
80	451.74	813.77	332148	305080	115000	0	0.00	0	0	0	106	106	0	0	0	0	0	0
100	**458.45**	**884.30**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**131**	**131**	**0**	**0**	**0**	**0**	**0**	**0**
130	465.67	963.13	392399	360850	116941	0	0.00	0	0	0	146	118	0	0	0	0	0	0
150	463.97	944.31	411351	381289	145000	0	0.00	0	0	0	146	127	0	0	0	0	0	0
175	468.50	994.84	432395	407507	145000	0	0.00	0	0	31	156	138	0	0	0	0	0	0
200	**470.21**	**1014.21**	**451163**	**429875**	**175825**	**0**	**0.00**	**0**	**0**	**32**	**159**	**141**	**19**	**1**	**0**	**0**	**5**	**0**
225	470.28	1015.10	468139	448786	194960	0	0.00	0	0	29	160	144	21	0	0	0	0	0
250	470.16	1013.63	483665	465352	228088	0	0.00	0	0	23	161	146	23	0	17	0	-5	0
325	470.84	1021.39	523757	506439	339880	0	0.00	0	0	17	162	147	28	0	22	10	-6	67
400	471.55	1029.61	556967	540033	488931	0	0.00	0	0	13	162	148	33	0	26	14	-1	119
500	471.80	1032.40	594159	577133	568037	0	0.00	0	0	12	180	150	39	0	30	17	-2	229

TABLE A-20: NA3-145 BASE (R060_800DR3.5e_145_FP471.5_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.59	397.11	37002	37002	26005	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.18	394.30	40722	40722	25215	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	413.74	470.92	90369	90369	44261	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	421.65	533.58	136522	136522	71655	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	424.92	560.66	167533	167533	115000	0	0.00	0	0	0	26	26	0	0	0	0	0	0
20	428.02	587.24	191482	191482	115000	0	0.00	0	0	0	36	36	0	0	0	0	0	0
25	431.43	617.37	211227	211227	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
35	437.15	669.98	243016	243016	115000	0	0.00	0	0	0	57	57	0	0	0	0	0	0
50	442.97	725.68	279485	279485	115000	0	0.00	0	0	0	75	75	0	0	0	0	0	0
65	449.11	786.85	308218	308218	115000	0	0.00	0	0	0	103	103	0	0	0	0	0	0
80	453.74	834.47	332148	332148	115000	0	0.00	0	0	0	127	127	0	0	0	0	0	0
100	**460.46**	**906.02**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**137**	**137**	**0**	**0**	**0**	**0**	**0**	**0**
130	461.49	917.22	392399	392399	145000	0	0.00	0	0	0	144	125	0	0	0	0	0	0
150	466.26	969.69	411351	411351	145000	0	0.00	0	0	7	152	133	0	0	0	0	0	0
175	469.90	1010.67	432395	432395	145000	0	0.00	0	0	34	160	142	0	0	0	0	0	0
200	**474.92**	**1068.57**	**451163**	**451163**	**145000**	**0**	**0.00**	**0**	**0**	**53**	**171**	**153**	**0**	**0**	**0**	**0**	**0**	**0**
225	477.03	1093.42	468139	468139	171154	0	0.00	0	0	56	173	156	19	1	0	0	4	0
250	477.36	1097.31	483665	483665	197562	0	0.00	0	0	56	174	158	23	0	0	0	7	0
325	477.22	1095.62	523757	523757	300796	0	0.00	0	0	48	196	157	28	0	22	10	-6	78
400	477.90	1103.69	556967	556967	399130	0	0.00	0	0	39	201	158	32	0	26	14	-5	100
500	478.32	1108.60	594159	594159	558062	0	0.00	0	0	28	205	159	38	0	30	18	-2	151

TABLE A-21: NA3-145 MAX (R060_800DR3.5e_145_FP471.5_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	23588	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	27464	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	30225	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	414.02	473.00	90369	90369	54221	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	423.56	549.32	136522	136522	81913	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	427.18	579.96	167533	167533	115000	0	0.00	0	0	0	28	28	0	0	0	0	0	0
20	430.25	606.84	191482	191482	115000	0	0.00	0	0	0	38	38	0	0	0	0	0	0
25	433.17	633.13	211227	211227	115000	0	0.00	0	0	0	47	47	0	0	0	0	0	0
35	438.57	683.35	243016	243016	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
50	444.11	736.88	279485	279485	115000	0	0.00	0	0	0	77	77	0	0	0	0	0	0
65	450.07	796.61	308218	308218	115000	0	0.00	0	0	0	105	105	0	0	0	0	0	0
80	456.02	858.42	332148	332148	115000	0	0.00	0	0	0	128	128	0	0	0	0	0	0
100	**461.90**	**921.63**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**139**	**139**	**0**	**0**	**0**	**0**	**0**	**0**
130	460.99	911.73	392399	392399	145000	0	0.00	0	0	0	142	124	0	0	0	0	0	0
150	466.41	971.34	411351	411351	145000	0	0.00	0	0	9	151	133	0	0	0	0	0	0
175	470.14	1013.42	432395	432395	145000	0	0.00	0	0	34	179	143	0	0	0	0	0	0
200	**#N/A**	**#N/A**	**451163**	**451163**	**#N/A**	**#N/A**	**#N/A**	**0**	**0**	**15**	**74**	**#N/A**	**5**	**#N/A**	**0**	**0**	**#N/A**	**#N/A**
225	477.06	1093.72	468139	468139	172840	0	0.00	0	0	56	195	156	19	0	0	0	4	0
250	477.19	1095.30	483665	483665	202925	0	0.00	0	0	55	197	157	23	0	17	0	1	0
325	477.39	1097.70	523757	523757	320734	0	0.00	0	0	47	199	156	29	0	22	11	100	35
400	478.01	1104.97	556967	556967	430723	0	0.00	0	0	37	205	157	32	0	26	14	-5	100
500	478.08	1105.79	594159	594159	558062	0	0.00	0	0	27	210	159	38	0	30	18	-2	167

TABLE A-22: NA3-145 MIN (R060_800DR3.5e_145_FP471.5_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.16	489.65	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	424.05	553.36	136522	126249	65753	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	428.10	587.92	167533	154598	84559	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	428.64	592.60	191482	176491	115000	0	0.00	0	0	0	25	25	0	0	0	0	0	0
25	431.48	617.85	211227	194541	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
35	436.67	665.48	243016	223601	115000	0	0.00	0	0	0	48	48	0	0	0	0	0	0
50	442.99	725.90	279485	256938	115000	0	0.00	0	0	0	62	62	0	0	0	0	0	0
65	448.29	778.56	308218	283204	115000	0	0.00	0	0	0	81	81	0	0	0	0	0	0
80	451.74	813.77	332148	305080	115000	0	0.00	0	0	0	106	106	0	0	0	0	0	0
100	**458.45**	**884.30**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**131**	**131**	**0**	**0**	**0**	**0**	**0**	**0**
130	465.98	966.59	392399	360850	115000	0	0.00	0	0	0	147	147	0	0	0	0	0	0
150	463.97	944.31	411351	381289	145000	0	0.00	0	0	0	146	127	0	0	0	0	0	0
175	468.21	991.54	432395	407507	145000	0	0.00	0	0	23	155	137	0	0	0	0	0	0
200	**471.93**	**1033.95**	**451163**	**429875**	**145000**	**0**	**0.00**	**0**	**0**	**44**	**164**	**146**	**0**	**0**	**0**	**0**	**0**	**0**
225	474.23	1060.55	468139	448786	145000	0	0.00	0	0	52	169	153	0	0	0	0	0	0
250	476.87	1091.48	483665	465352	145000	0	0.00	0	0	60	176	161	0	0	0	0	0	0
325	481.26	1143.79	523757	506439	182469	0	0.00	0	0	69	184	169	22	0	0	0	10	0
400	485.04	1189.63	556967	540033	406641	80	1.04	0	0	61	182	168	25	0	17	6	167	63
500	486.04	1201.93	594159	577133	670948	220	2.04	0	0	49	197	167	32	0	23	10	-2	440

TABLE A-23: NA1-160 BASE (R000_800CF_No Fix_160_FP470_P1_20081214)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.60	397.18	37002	37002	25945	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.08	393.58	40722	40722	25891	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	418.26	506.23	90369	90369	26643	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	431.09	614.37	136522	136522	43519	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	436.58	664.64	167533	167533	71079	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	440.71	703.82	191482	191482	87949	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	444.54	741.14	211227	211227	104311	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	450.77	803.76	243016	243016	115000	0	0.00	0	0	0	28	28	0	0	0	0	0	0
50	459.13	891.67	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	462.93	932.91	308218	308218	135000	0	0.00	0	0	0	65	65	0	0	0	0	0	0
80	468.15	990.91	332148	332148	135000	0	0.00	0	0	23	78	78	0	0	0	0	0	0
100	**472.32**	**1038.47**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**43**	**96**	**96**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.29	1072.93	392399	392399	186741	0	0.00	0	0	48	104	102	30	10	0	0	22	0
150	475.86	1079.58	411351	411351	236150	0	0.00	0	0	48	105	102	32	8	17	0	36	5
175	476.03	1081.62	432395	432395	268705	0	0.00	0	0	48	105	103	34	9	19	0	82	17
200	**476.37**	**1085.66**	**451163**	**451163**	**321017**	**0**	**0.00**	**0**	**0**	**45**	**104**	**102**	**37**	**9**	**22**	**11**	**53**	**42**
225	476.56	1087.85	468139	468139	361431	0	0.00	0	0	41	104	102	40	9	24	12	64	42
250	476.64	1088.84	483665	483665	408551	0	0.00	0	0	38	103	100	41	9	26	14	71	46
325	477.14	1094.64	523757	523757	482854	0	0.00	0	0	35	102	101	48	12	30	18	86	49
400	477.86	1103.24	556967	556967	503865	0	0.00	0	0	38	107	107	41	0	35	21	36	48
500	479.04	1117.15	594159	594159	513195	0	0.00	0	0	43	129	115	48	0	39	25	32	51

TABLE A-24: NA1-160 MAX (R000_800CF_No Fix_160_FP470_P1_20081214)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	26113	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	26125	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	419.62	517.12	90369	90369	27084	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	431.28	616.04	136522	136522	54716	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	438.30	680.77	167533	167533	78433	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	443.02	726.15	191482	191482	97176	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	446.43	759.84	211227	211227	114092	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	452.77	824.45	243016	243016	115000	0	0.00	0	0	0	34	34	0	0	0	0	0	0
50	460.96	911.37	279485	279485	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
65	464.23	947.20	308218	308218	135000	0	0.00	0	0	0	69	68	0	0	0	0	0	0
80	468.90	999.41	332148	332148	135000	0	0.00	0	0	27	82	80	0	0	0	0	0	0
100	**472.51**	**1040.60**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**44**	**96**	**96**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.13	1071.01	392399	392399	178470	0	0.00	0	0	48	102	102	30	10	0	0	9	0
150	475.70	1077.77	411351	411351	219943	0	0.00	0	0	48	104	103	32	9	17	0	45	0
175	476.05	1081.83	432395	432395	270444	0	0.00	0	0	48	105	103	34	9	19	0	86	15
200	**476.38**	**1085.69**	**451163**	**451163**	**321521**	**0**	**0.00**	**0**	**0**	**45**	**104**	**102**	**37**	**9**	**22**	**11**	**51**	**42**
225	476.57	1087.95	468139	468139	363644	0	0.00	0	0	41	104	102	40	9	24	12	69	42
250	476.65	1088.93	483665	483665	410822	0	0.00	0	0	37	102	99	41	9	26	14	77	46
325	477.16	1094.95	523757	523757	484299	0	0.00	0	0	35	103	101	48	11	31	18	48	48
400	477.82	1102.73	556967	556967	503510	0	0.00	0	0	38	107	107	51	10	35	21	49	46
500	479.01	1116.84	594159	594159	512982	0	0.00	0	0	43	127	115	57	9	39	25	36	52

TABLE A-25: NA1-160 MIN (R000_800CF_No Fix_160_FP470_P1_20081214)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	418.60	508.91	90369	84059	27017	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	430.16	606.08	136522	126249	50159	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	435.32	652.92	167533	154598	66881	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	438.84	685.88	191482	176491	81222	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	441.67	713.08	211227	194541	90784	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	446.19	757.45	243016	223601	112689	0	0.00	0	0	0	0	0	0	0	0	0	0	0
50	452.28	819.34	279485	256938	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
65	457.86	877.97	308218	283204	115000	0	0.00	0	0	0	50	50	0	0	0	0	0	0
80	459.50	895.65	332148	305080	135000	0	0.00	0	0	0	56	56	0	0	0	0	0	0
100	**464.68**	**952.16**	**359078**	**329825**	**135000**	**0**	**0.00**	**0**	**0**	**0**	**70**	**70**	**0**	**0**	**0**	**0**	**0**	**0**
130	470.22	1014.37	392399	360850	145000	0	0.00	0	0	34	87	87	0	0	0	0	0	0
150	471.89	1033.48	411351	381289	160000	0	0.00	0	0	45	99	98	22	24	0	0	0	0
175	475.26	1072.52	432395	407507	173176	0	0.00	0	0	50	111	111	31	10	0	0	8	0
200	**475.59**	**1076.50**	**451163**	**429875**	**215563**	**0**	**0.00**	**0**	**0**	**50**	**110**	**110**	**33**	**9**	**17**	**0**	**27**	**0**
225	475.83	1079.24	468139	448786	257292	0	0.00	0	0	48	107	106	34	8	20	0	65	28
250	476.10	1082.41	483665	465352	293291	0	0.00	0	0	46	107	106	36	8	22	0	65	49
325	476.60	1088.34	523757	506439	392913	0	0.00	0	0	36	106	105	40	8	26	14	90	61
400	476.93	1092.23	556967	540033	467440	0	0.00	0	0	32	105	104	49	12	30	17	74	65
500	477.96	1104.34	594159	577133	504670	0	0.00	0	0	36	124	111	55	12	34	20	87	52

TABLE A-26: NA2-160 BASE (R060_800FM_No Fix_160_FP466_P1_20090106)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.59	397.11	37002	37002	26005	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.18	394.30	40722	40722	25215	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	413.74	470.92	90369	90369	44261	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	421.65	533.58	136522	136522	71655	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	425.56	566.08	167533	167533	115000	0	0.00	0	0	0	24	24	0	0	0	0	0	0
20	428.70	593.15	191482	191482	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
25	432.02	622.68	211227	211227	115000	0	0.00	0	0	0	42	42	0	0	0	0	0	0
35	437.51	673.33	243016	243016	115000	0	0.00	0	0	0	56	56	0	0	0	0	0	0
50	444.04	736.19	279485	279485	115000	0	0.00	0	0	0	75	75	0	0	0	0	0	0
65	449.69	792.72	308218	308218	115000	0	0.00	0	0	0	101	101	0	0	0	0	0	0
80	454.35	840.92	332148	332148	115000	0	0.00	0	0	0	125	125	0	0	0	0	0	0
100	**461.31**	**915.15**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**134**	**134**	**0**	**0**	**0**	**0**	**0**	**0**
130	459.65	897.26	392399	392399	160000	0	0.00	0	0	0	137	121	30	31	0	0	0	0
150	464.33	948.31	411351	411351	160000	0	0.00	0	0	0	146	131	32	33	0	0	0	0
175	467.74	986.26	432395	432395	160000	0	0.00	0	0	26	156	141	30	31	0	0	0	0
200	**470.09**	**1012.88**	**451163**	**451163**	**196633**	**0**	**0.00**	**0**	**0**	**26**	**157**	**143**	**37**	**17**	**0**	**0**	**37**	**0**
225	470.16	1013.72	468139	468139	248894	0	0.00	0	0	21	156	143	40	16	17	0	85	3
250	470.44	1016.88	483665	483665	296022	0	0.00	0	0	18	157	144	41	16	19	0	128	7
325	471.17	1025.16	523757	523757	458379	0	0.00	0	0	13	176	144	44	14	24	13	127	110
400	471.32	1026.88	556967	556967	523129	0	0.00	0	0	13	183	146	47	13	28	16	120	127
500	471.57	1029.74	594159	594159	594159	0	0.00	0	0	12	191	149	53	14	31	20	175	146
PMF	477.46	1098.49	905770	905770	811646	0	0.00	0	0	58	246	124	109	18	56	44	100	139

TABLE A-27: NA2-160 MAX (R060_800FM_No Fix_160_FP466_P1_20090106)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	23588	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	27464	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	30225	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	415.59	485.14	90369	90369	54221	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	423.58	549.49	136522	136522	81913	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	427.91	586.22	167533	167533	115000	0	0.00	0	0	0	26	26	0	0	0	0	0	0
20	430.94	612.96	191482	191482	115000	0	0.00	0	0	0	36	36	0	0	0	0	0	0
25	434.32	643.67	211227	211227	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
35	439.26	689.88	243016	243016	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
50	446.18	757.39	279485	279485	115000	0	0.00	0	0	0	78	78	0	0	0	0	0	0
65	451.66	812.98	308218	308218	115000	0	0.00	0	0	0	104	104	0	0	0	0	0	0
80	457.54	874.57	332148	332148	115000	0	0.00	0	0	0	126	126	0	0	0	0	0	0
100	**462.60**	**929.22**	**359078**	**359078**	**115155**	**0**	**0.00**	**0**	**0**	**0**	**136**	**106**	**0**	**0**	**0**	**0**	**0**	**0**
130	461.22	914.25	392399	392399	160000	0	0.00	0	0	0	136	123	30	31	0	0	0	0
150	464.46	949.80	411351	411351	160000	0	0.00	0	0	0	145	131	32	33	0	0	0	0
175	467.90	988.09	432395	432395	160000	0	0.00	0	0	27	154	141	30	31	0	0	0	0
200	**470.10**	**1012.99**	**451163**	**451163**	**195966**	**0**	**0.00**	**0**	**0**	**26**	**176**	**143**	**37**	**17**	**0**	**0**	**33**	**0**
225	470.20	1014.16	468139	468139	254566	0	0.00	0	0	20	177	143	40	16	17	0	92	2
250	470.35	1015.82	483665	483665	288029	0	0.00	0	0	18	179	144	41	16	19	0	118	7
325	471.14	1024.90	523757	523757	452926	0	0.00	0	0	13	185	145	44	14	24	14	175	94
400	471.33	1027.02	556967	556967	540048	0	0.00	0	0	12	189	146	47	13	28	17	175	120
500	471.55	1029.62	594159	594159	594159	0	0.00	0	0	13	197	149	53	13	32	21	175	107

TABLE A-28: NA2-160 MIN (R060_800FM_No Fix_160_FP466_P1_20090106)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.16	489.65	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	424.05	553.36	136522	126249	65753	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	428.30	589.62	167533	154598	84559	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	429.46	599.84	191482	176491	115000	0	0.00	0	0	0	23	23	0	0	0	0	0	0
25	432.18	624.13	211227	194541	115000	0	0.00	0	0	0	31	31	0	0	0	0	0	0
35	437.35	671.86	243016	223601	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
50	443.50	730.90	279485	256938	115000	0	0.00	0	0	0	61	61	0	0	0	0	0	0
65	448.76	783.26	308218	283204	115000	0	0.00	0	0	0	80	80	0	0	0	0	0	0
80	452.90	825.76	332148	305080	115000	0	0.00	0	0	0	116	116	0	0	0	0	0	0
100	**459.22**	**892.62**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**129**	**129**	**0**	**0**	**0**	**0**	**0**	**0**
130	466.33	970.48	392399	360850	121233	0	0.00	0	0	9	143	119	0	0	0	0	0	0
150	463.02	933.87	411351	381289	160000	0	0.00	0	0	0	140	125	27	28	0	0	0	0
175	467.23	980.56	432395	407507	160000	0	0.00	0	0	23	150	137	25	26	0	0	0	0
200	**470.09**	**1012.92**	**451163**	**429875**	**176230**	**0**	**0.00**	**0**	**0**	**30**	**153**	**140**	**33**	**14**	**0**	**0**	**6**	**0**
225	470.13	1013.35	468139	448786	198409	0	0.00	0	0	26	154	143	36	15	0	0	35	0
250	470.11	1013.14	483665	465352	227979	0	0.00	0	0	23	156	146	38	15	17	0	68	0
325	470.67	1019.48	523757	506439	327556	0	0.00	0	0	17	158	147	42	14	22	10	125	38
400	471.18	1025.30	556967	540033	463776	0	0.00	0	0	13	159	149	47	14	26	14	125	114
500	471.42	1028.03	594159	577133	544670	0	0.00	0	0	12	175	151	52	14	29	17	175	177

TABLE A-29: NA3-160 BASE (R060_800DR3.5e_160_FP471.5_P1_20081215)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.59	397.11	37002	37002	26005	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.18	394.30	40722	40722	25215	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	413.74	470.92	90369	90369	44261	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	421.65	533.58	136522	136522	71655	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	425.56	566.08	167533	167533	115000	0	0.00	0	0	0	24	24	0	0	0	0	0	0
20	428.70	593.15	191482	191482	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
25	432.02	622.68	211227	211227	115000	0	0.00	0	0	0	42	42	0	0	0	0	0	0
35	437.51	673.33	243016	243016	115000	0	0.00	0	0	0	56	56	0	0	0	0	0	0
50	444.04	736.19	279485	279485	115000	0	0.00	0	0	0	75	75	0	0	0	0	0	0
65	449.69	792.72	308218	308218	115000	0	0.00	0	0	0	101	101	0	0	0	0	0	0
80	454.35	840.92	332148	332148	115000	0	0.00	0	0	0	125	125	0	0	0	0	0	0
100	**461.31**	**915.15**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**134**	**134**	**0**	**0**	**0**	**0**	**0**	**0**
130	459.65	897.26	392399	392399	160000	0	0.00	0	0	0	137	121	30	31	0	0	0	0
150	464.33	948.31	411351	411351	160000	0	0.00	0	0	0	146	131	32	33	0	0	0	0
175	467.47	983.26	432395	432395	160000	0	0.00	0	0	17	153	138	35	36	0	0	0	0
200	**472.47**	**1040.14**	**451163**	**451163**	**160000**	**0**	**0.00**	**0**	**0**	**46**	**164**	**150**	**32**	**33**	**0**	**0**	**0**	**0**
225	476.20	1083.60	468139	468139	160000	0	0.00	0	0	57	170	157	34	35	0	0	0	0
250	477.15	1094.82	483665	483665	193667	0	0.00	0	0	54	170	157	41	19	0	0	18	0
325	477.08	1093.97	523757	523757	294943	0	0.00	0	0	48	189	157	44	16	22	0	78	4
400	477.78	1102.31	556967	556967	405477	0	0.00	0	0	38	195	158	47	15	26	14	100	81
500	478.03	1105.25	594159	594159	534386	0	0.00	0	0	28	202	160	53	16	29	17	107	150

TABLE A-30: NA3-160 MAX (R060_800DR3.5e_160_FP471.5_P1_20081215)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	26113	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	26125	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	419.62	517.12	90369	90369	27084	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	431.28	616.04	136522	136522	54716	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	438.30	680.77	167533	167533	78433	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	443.02	726.15	191482	191482	97176	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	446.43	759.84	211227	211227	114092	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	452.77	824.45	243016	243016	115000	0	0.00	0	0	0	34	34	0	0	0	0	0	0
50	460.96	911.37	279485	279485	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
65	464.23	947.20	308218	308218	135000	0	0.00	0	0	0	69	68	0	0	0	0	0	0
80	468.90	999.41	332148	332148	135000	0	0.00	0	0	27	82	80	0	0	0	0	0	0
100	**472.51**	**1040.59**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**44**	**96**	**96**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.46	1074.96	392399	392399	187473	0	0.00	0	0	49	103	103	20	0	0	0	5	0
150	475.86	1079.56	411351	411351	229570	0	0.00	0	0	49	104	103	23	0	17	0	-6	6
175	476.20	1083.56	432395	432395	279376	0	0.00	0	0	47	104	102	26	0	20	0	47	8
200	**476.47**	**1086.84**	**451163**	**451163**	**329343**	**0**	**0.00**	**0**	**0**	**44**	**104**	**102**	**28**	**0**	**22**	**11**	**-6**	**41**
225	476.60	1088.35	468139	468139	376509	0	0.00	0	0	40	103	101	31	0	24	12	-5	42
250	476.70	1089.55	483665	483665	421762	0	0.00	0	0	37	102	99	33	0	26	14	56	48
325	477.24	1095.85	523757	523757	488365	0	0.00	0	0	35	103	101	37	0	31	19	-6	42
400	477.87	1103.32	556967	556967	503936	0	0.00	0	0	38	107	107	42	0	35	21	45	48
500	479.04	1117.22	594159	594159	513243	0	0.00	0	0	43	127	115	48	0	39	25	30	51

TABLE A-31: NA3-160 MIN (R060_800DR3.5e_160_FP471.5_P1_20081215)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.16	489.65	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	424.05	553.36	136522	126249	65753	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	428.30	589.62	167533	154598	84559	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	429.46	599.84	191482	176491	115000	0	0.00	0	0	0	23	23	0	0	0	0	0	0
25	432.18	624.13	211227	194541	115000	0	0.00	0	0	0	31	31	0	0	0	0	0	0
35	437.35	671.86	243016	223601	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
50	443.50	730.90	279485	256938	115000	0	0.00	0	0	0	61	61	0	0	0	0	0	0
65	448.76	783.26	308218	283204	115000	0	0.00	0	0	0	80	80	0	0	0	0	0	0
80	452.90	825.76	332148	305080	115000	0	0.00	0	0	0	116	116	0	0	0	0	0	0
100	**459.22**	**892.62**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**129**	**129**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.26	980.92	392399	360850	115000	0	0.00	0	0	19	144	144	0	0	0	0	0	0
150	463.02	933.87	411351	381289	160000	0	0.00	0	0	0	140	125	27	28	0	0	0	0
175	467.04	978.39	432395	407507	160000	0	0.00	0	0	14	147	134	31	32	0	0	0	0
200	**470.56**	**1018.21**	**451163**	**429875**	**160000**	**0**	**0.00**	**0**	**0**	**36**	**155**	**142**	**33**	**34**	**0**	**0**	**0**	**0**
225	472.71	1042.91	468139	448786	160000	0	0.00	0	0	49	162	151	31	32	0	0	0	0
250	474.90	1068.38	483665	465352	160000	0	0.00	0	0	55	168	158	33	35	0	0	0	0
325	477.32	1096.76	523757	506439	214967	0	0.00	0	0	57	173	162	42	17	18	0	40	0
400	477.15	1094.84	556967	540033	310772	0	0.00	0	0	47	172	162	47	16	24	11	89	15
500	477.79	1102.40	594159	577133	420080	0	0.00	0	0	34	187	163	52	16	27	14	125	96

US Army Corps of Engineers
Sacramento District

American River Watershed
Common Features Project
Natomas Post-Authorization Change Report

Appendix B3

Dry and Arcade Creeks Flow Frequency Curves and Synthetic 8-Flood Series Hydrographs Upstream of Steelhead Creek

January 2010

AMERICAN RIVER WATERSHED COMMON FEATURES PROJECT
NATOMAS POST-AUTHORIZATION CHANGE REPORT

APPENDIX B3
DRY AND ARCADE CREEKS FLOW FREQUENCY CURVES AND SYNTHETIC 8-FLOOD SERIES HYDROGRAPHS UPSTREAM OF STEELHEAD CREEK

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Peer Review Appendix 3: Peak Flow Frequency Relationships for Dry Creek at Vernon Street and Arcade Creek at American River, 1996

Cover photo: Dry Creek upstream of the confluence of Magpie and Robla creeks

AMERICAN RIVER COMMON FEATURES, CALIFORNIA
GRR FEASIBILITY STUDY

APPENDIX B3
DRY AND ARCADE CREEKS FLOW FREQUENCY CURVES AND SYNTHETIC 8-FLOOD SERIES HYDROGRAPHS UPSTREAM OF STEELHEAD CREEK

1. STUDY BACKGROUND AND SCOPE

This report presents the hydrologic peak flow frequency analysis of flows on Dry and Arcade creeks for the synthetic 8-flood series hydrographs. The synthetic 8-flood series consists of the 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2% chance floods on Dry and Arcade creeks. The flow frequency analysis includes updating the peak flow record to 2009 as well as developing or revising flow frequency curves for the 1-, 3-, 5-, and 10-day durations.

This analysis is being conducted in response to questions raised about the influence high peak flows upstream on the Steelhead Creek tributaries would have on Steelhead Creek flood stages. (Steelhead Creek is also known as Natomas East Main Drainage Canal (NEMDC)). Included in the analysis is a revision of the synthetic 8-flood series hydrographs presented in the American River Common Features General Reevaluation Report (AR CF GRR) Appendix A, Synthetic Hydrology Technical Documentation (**Reference 1**). Future modeling for the AR CF GRR will include hydraulic modeling up the NEMDC tributaries, for Dry Creek upstream to the Placer-Sacramento County line, and for Arcade Creek upstream to the Sacramento County gage on Arcade Creek at Winding Way.

The revised synthetic 8-flood series hydrographs include balanced hydrographs with higher peaks for Dry Creek at Vernon Street (Roseville) and Arcade Creek at the "near Del Paso Heights" gage, based on the updated flow frequency curves for those locations. The total 8-flood series hydrographs for downstream locations on Dry and Arcade creeks also have higher peak flows, only because the Vernon Street and Del Paso Heights hydrographs have been revised. The 8-flood series hydrographs for downstream local flows on Dry and Arcade creeks, as well as the other NEMDC tributaries, were not revised: there were no stream gages to calibrate to for the higher flood flows. Also, the higher flood peaks on Dry and Arcade creeks are produced by greater and more intense rainfall on the higher eastside elevations of these watersheds, and not by more intense rainfall on the flat valley floor. **Plate 1**, the General Map, shows the locations of Steelhead Creek (NEMDC) and its tributaries, Dry Creek, Arcade Creek, Upper NEMDC, and Old Magpie Creek. **Plate 2** shows locations of the index points for Dry Creek and **Plate 3** those index points for Arcade Creek for which hydrographs were developed.

2. DRY CREEK HYDROLOGY/HYDRAULICS PEER REVIEW AND CONCENSUS EFFORT

2.1 Peer Review Background. An intense storm hit Sacramento and western Placer counties on the evening of January 9 through the early morning hours of January 10, 1995. Overflow from the streams in the area caused severe flooding in both counties. Peak flows on Dry and Arcade creeks for the January 1995 storm are the largest of record for those streams. Sacramento area government agencies initiated a post storm analysis, the Dry Creek Hydrology/Hydraulics Peer Review and Consensus Effort (Peer Review). Agencies and consulting engineering firms involved in the Peer Review included the U.S. Army Corps of Engineers; Sacramento County Water Resources Division; Sacramento Area Flood Control Agency (SAFCA); Placer County Flood Control & Water Conservation District; City of Sacramento Utilities Department; and the engineering firms of Ensign & Buckley Consulting Engineers; DC Consulting; Montgomery Watson Consulting Engineers; Borcalli & Associates; HYDMET, Incorporated; CH2M Hill; and Murray, Burns, & Keinlen. Appendix 3 (**Reference 2**) of the draft hydrology report presented an analysis of the peak flow frequency relationships for Dry Creek at Vernon Street, and Arcade Creek at American River. The Peer Review Statement of Findings, dated 6 November 1996 (**Reference 3**), includes a peak flow frequency curve for Dry Creek at Vernon Street in Roseville, California.

2.2 Dry Creek at Vernon Street Peak Flow Frequency Curve. The California State Department of Water Resources (DWR) operated a stream gage (gage A00040) on Dry Creek in Roseville upstream of the SPRR culverts for water years 1950 to 1966. The drainage area at this location is 78.2 square miles. In 1966 the gage was discontinued and relocated (as gage A00047) to upstream of Douglas Boulevard, with a drainage area of 57.9 square miles. Gage A00047 is referred to in the record as both "Dry Creek at Roseville above Douglas Boulevard" and "Dry Creek at Royer Park." This gage was discontinued in 1984 and moved to Vernon Street, about 1,500 feet upstream of the SPRR bridge. This gage, A00041, "Dry Creek below Roseville," with a drainage area of about 78 square miles, was damaged by the February 1986 flood and discontinued. Records for the three stream gages are incomplete. The City of Roseville established a gage at the Vernon Street location in 1987 (Sensor ID #1603) as part of the ALERT (Automated Local Evaluation in Real Time) system (**Reference 4**) to provide local stream and weather information during storm events. The City of Roseville also operates an ALERT gage at the Royer Park location (Sensor ID #1630).

As part of the Peer Review, a peak flow frequency curve was developed for the Dry Creek at Vernon Street location for water years 1950 to 1995 using peak flow records for the DWR gages A00040, A00041, and A00047. Peak flows for 1968 to 1975 and 1978 to 1981 were developed for the Vernon Street location based on a drainage area relationship between the Vernon Street gage and the upstream Royer Park gage. With so much missing data for the Vernon Street, SPRR culvert, and Royer Park locations, peak flows between 1950 and 1995 were also estimated from the mean daily flow record, observed flow on Arcade Creek, storm precipitation, HEC-2 and HEC-RAS modeling, and high water marks in Roseville and downstream near Elverta Road in Sacramento County. See Peer Review Appendix 3, included as an appendix to this report, for additional information.

The 46-year record, using recorded and estimated peak flows, was used with the Corps of Engineers Flood Frequency Analysis program, HEC-FFA (**Reference 5**), to compute statistics for the peak flow frequency curve for Dry Creek at Vernon Street. The FFA program identified 1977 as a low outlier. The FFA final results statistics were almost the same as those for the final Dry Creek at Vernon Street peak flow frequency curve included in the Dry Creek Peer Review Statement of Findings, dated 6 November 1996 (**Reference 3**).

The final Peer Review peak flow frequency curve for Vernon Street includes tabulations for two sets of n-flood series peak flows. One set is for the flow frequency curve, with flows based on the adjusted gage measurements. These flows are very similar to the peak flows computed in the HEC-FFA run. The other n-flood peak flow tabulation is for flood flows from an HEC-1 calibration to the January 1995 flood. **Table 1** lists the n-flood peaks for the HEC-FFA run, the adjusted gage measurements, and the HEC-1 calibration. Part of the process in developing the balanced flood hydrographs was a decision as to which set of n-flood peaks to use for the balanced hydrographs for Dry Creek at Vernon Street.

Files associated with the Peer Review analysis include hydrographs from the HEC-1 calibration for Dry Creek, with n-flood series hydrographs (10-, 2-, 1-, 0.5-, and 0.2%) for various locations on the NEMDC tributaries. These are compiled in a single spreadsheet file referred to elsewhere in this report as "Excel spreadsheet" with n-flood series hydrographs (10-, 2-, 1-, 0.5-, and 0.2% floods) for various locations on the NEMDC tributaries. These hydrographs are from the Peer Review HEC-1 Calibration for Dry Creek. **Table 1** also lists the peak flows for Dry Creek routed to NEMDC from the HEC-1 Calibration.

Table 1
Dry Creek Peak Flow Comparison for Synthetic 8-Flood Series Hydrographs
Peer Review HEC-1 Model and FFA Program

	Flood Event and Peak Flows (cfs)				
Dry Cr. at Vernon St. (78.12 sq mi)	10% 2%		1%	0.50%	0.20%
HEC-1 Calibration	7,300	13,000	15,900	18,700	23,600
Adjusted Gage Measurement	5,640	11,200	14,400	18,300	24,500
HEC-FFA Run	5,620	11,100	14,300	18,200	24,400
	Flood Event and Peak Flows (cfs)				
Dry Cr. at NEMDC (115.8 sq mi)	10%	2% 1%		0.50%	0.20%
HEC-1 Calibration	6,860	12,300	13,900	16,440	21,500

2.3 Arcade Creek Peak Flow Frequency Curve. The USGS operated a stream gage (ID 11447360), Arcade Creek near Del Paso Heights, for water years 1964 to 1978, when the gage was discontinued. This gage was located just upstream of Watt Avenue, with a drainage area of 31.8 square miles. The County of Sacramento has operated a gage, Arcade Creek at Winding Way (Sensor ID 298), from 1961 to present, with some missing years. This gage, also known as the American River College gage, has a drainage area of 28.4 square miles. It is currently part of the ALERT (Automated Local Evaluation in Real Time) system.

As part of the Peer Review, a peak flow frequency curve for water years 1962 to 1995 was computed for Arcade Creek using flow records for the USGS gage combined with the Sacramento County gage. The difference in drainage area between the USGS gage and the upstream Sacramento County gage is only 3.4 square miles. Data for the missing years (1979 to 1981 and 1985) were estimated using peak flows for Dry Creek at Vernon Street.

The 34-year record for the combined gages, including estimated flows, was used with the Corps of Engineers' Flood Frequency Analysis program, HEC-FFA, to compute statistics for the peak flow frequency curve for Arcade Creek at Winding Way/Del Paso Heights. See Peer Review Appendix 3, included as an appendix to this report, for additional information.

Additional files associated with the Peer Review analysis include "HEC-1 flood runs" for the NEMDC tributaries only for the 2- and 1% event storms for the HEC-1 Calibration. The modeling includes hydrographs for Arcade Creek at Winding Way, at the "near Del Paso Heights gage," and at NEMDC. **Table 2** lists the peak flows for these three locations for the 2- and 1% floods, as well as the n-flood series peak flows from the HEC-FFA program. The difference between peak flows at Winding Way and at the downstream Del Paso Heights gage is less than 1%.

Table 2

Arcade Creek Peak Flow Comparison

Peer Review HEC-1 Model and FFA Program

Peer Review HEC-1 Model Results					
	Flood Event and Flows (cfs)				
10%		2%	1%	0.50%	0.20%
Arcade Cr. - Winding Way (28.4 sq mi) Peak (cfs)	N/A	3,960	4,500	N/A	N/A
Arcade Cr. - Del Paso gage (31.8 sq mi) Peak (cfs)	N/A	3,950	4,470	N/A	N/A
Arcade Cr. - NEMDC (40.1 sq mi) Peak (cfs)	N/A	3,860	4,440	N/A	N/A
Peer Review FFA Program Results					
Arcade Creek for Winding Way/ Del Paso Heights gage	3,010	4,260	4,770	5,260	5,900

Note: N/A = flows not available

3. UPDATED PEAK FLOW RECORDS FOR DRY AND ARCADE CREEKS

3.1 Dry Creek at Vernon Street Gage. In 1996, the USGS established a gage (USGS ID 11447293, Dry Creek at Vernon Street Bridge at Roseville, CA) at the Vernon Street location. Only a few days of data were recorded for each of the water years 1997 through 1999. The USGS gage has annual peak flows for 1997 and for 2000 to 2009. The City of Roseville provided peak and mean day flow data for the Vernon Street ALERT gage for 1996, 1998 and 1999. With this information, the peak flow record for the Vernon Street gage was updated from 1995 to 2009.

The annual peak flow record for 60 years, for 1950 to 2009, for Dry Creek at Vernon Street gage, was created using observed and estimated flows based on stage records and high water marks at three DWR gages, a USGS gage, and an ALERT gage. The drainage areas for the DWR gages, A00040 and A00041, the USGS gage, and the ALERT gage are all around 78 square miles. Peak flows observed or estimated for the DWR Royer Park gage and stages downstream at Elverta Road were areally adjusted to the Vernon Street drainage area. The annual peak flows for 1950 to 2009 were used with the HEC-FFA program to compute statistics for the updated record for Dry Creek at Vernon Street. 1977 was identified as a low outlier year. **Table 3** compares the peak flow statistics for Dry Creek at Vernon Street.

Table 3
Dry Creek at Vernon Street
Comparison of Peak Flow Frequency Statistics

	Mean	Standard Deviation	Skew	Years of Record
Peer Review FFA	3.3184	0.3294	0.3	46 (1950 - 1995)
Peer Review Findings Adjusted Gage Measurement	3.3189	0.3301	0.3	46 (1950 - 1995)
Updated Record FFA	3.3367	0.3213	0.4	60 (1950 - 2009)

3.2 Arcade Creek: Winding Way and Del Paso Heights Gages. The peak flow records for Arcade Creek near Del Paso Heights were updated using records from the USGS stream gage, which was reestablished in water year 1996. The annual peak flow record for Arcade Creek includes peak flows from the "near Del Paso Heights" gage for 1964 to 1978 and 1996 to 2008; peak flows for the Sacramento County gage at Winding Way for 1962 and 1963, 1982 to 1984, and 1986 to 1995; and recorded or estimated flows on Dry Creek for 1979 to 1981 and 1985. The 47 years of annual peak flows (1962 to 2008) for Arcade Creek were used with the HEC-FFA program to compute statistics for the updated record. 1976 was identified as a low outlier year. **Table 4** presents a comparison of the peak flow statistics for Arcade Creek.

Table 4
Arcade Creek at Winding Way/Del Paso Heights Gage
Comparison of Peak Flow Frequency Statistics

	Mean	Std. Dev.	Skew	Years of Record
Peer Review FFA	3.1699	0.2504	-0.4	34 (1962 - 1995)
Updated Record FFA	3.1777	0.2326	-0.4	47 (1962 - 2008)

4. MEAN DAILY FLOWS FOR DRY AND ARCADE CREEKS

Flow frequency curves for longer durations for Dry and Arcade creeks are needed in order to develop balanced synthetic 8-flood series hydrographs on those watersheds. The 1996 Peer Review was concerned with computation of the peak flow frequency curves, not the longer duration curves. **Table 5** lists the one-day flows associated with the n-flood peak flows for Dry Creek at Vernon Street and at NEMDC. These one-day flows were computed from the n-flood hydrographs in the Peer Review "Excel spreadsheet" file. **Table 5** also lists the one-day flows associated with the Arcade Creek peak flows in the HEC-1 model for the 2- and 1% storm events.

Table 5
Dry and Arcade Creeks
One-Day Flows Associated with Synthetic 8-Flood Peak Flows

Stream and Index Location	Flood Event and One-Day Flows (avg. cfs)				
	10%	2%	1%	0.50%	0.20%
Dry Cr. at Vernon St. (78.12 sq.mi.)	3,050	5,520	6,770	8,110	10,720
Dry Cr. at NEMDC (115.8 sq.mi.)	3,920	7,120	8,630	10,560	14,790
Arcade Cr. at Winding Way (28.4 sq.mi.)	N/A	1,690	1,960	N/A	N/A
Arcade Cr. at Del Paso Heights gage (31.8 sq.mi.)	N/A	1,700	1,970	N/A	N/A
Arcade Cr. at NEMDC (40.1 sq.mi.)	N/A	1,520	1,850	N/A	N/A

Note: N/A = data not available.

4.1 <u>Dry Creek Flow Duration Data</u>. Much of the Dry Creek daily flow record is missing for periods when flows were very high. For the Corps of Engineers' Dry Creek Hydrology Office Report, revised July 1987 (**Reference 6**), the annual Dry Creek peak and associated one-day flows were either observed or estimated for the DWR stream gage A00040, upstream of the SPRR culvert, near Vernon Street in Roseville. The peak and one-day flows for 1951 to 1966 are based on the gage at this location. Peak and one-day flows for gage A00040 for 1967 to 1982 were based on a drainage area relationship with DWR gage A00047 upstream of Douglas Boulevard in Roseville. The estimated one-day flows for A00040 were not used for every year, but were used for 23 years between 1952 and 1981. **Table 6** lists the estimated peak flows and associated one-day flows used in the revised flow frequency for Dry Creek at Vernon Street. **Table 6** includes a tabulation of the recorded peak, 1-, 3-, 5-, and 10-day annual flows for water years 2000 through 2009 for the USGS gage at Vernon Street.

4.2 <u>Arcade Creek Flow Duration Data</u>. Flow duration data for Arcade Creek at the Del Paso Heights USGS gage are available for water years 1964 to 1978 and 1996 to 2009. Observed and estimated peak flows for Arcade Creek at the Winding Way location (Sacramento County gage) are available for water years 1962 to 1963 and 1979 to 1995. No flow duration data are available for the Winding Way location.

Table 6
Dry Creek at Vernon Street Gage in Roseville
Annual Flow Duration Data

Water Year	Flow Duration and Average Flow (cfs)				
	Peak	1-Day	3-Day	5-Day	10-Day
1950	1,260	N/A	N/A	N/A	N/A
1951	1,980	N/A	N/A	N/A	N/A
1952	2,000	1,350	N/A	N/A	N/A
1953	2,839	2,060	N/A	N/A	N/A
1954	1,095	700	N/A	N/A	N/A
1955	1,230	674	N/A	N/A	N/A
1956	4,000	2,900	N/A	N/A	N/A
1957	1,130	868	N/A	N/A	N/A
1958	4,190	2,010	N/A	N/A	N/A
1959	748	582	N/A	N/A	N/A
1960	2,240	1,300	N/A	N/A	N/A
1961	1,212	800	N/A	N/A	N/A
1962	3,900	3,080	N/A	N/A	N/A
1963	5,800	N/A	N/A	N/A	N/A
1964	2,800	N/A	N/A	N/A	N/A
1965	3,800	2,100	N/A	N/A	N/A
1966	989	682	N/A	N/A	N/A
1967	4,800	N/A	N/A	N/A	N/A
1968	1,087	673	N/A	N/A	N/A
1969	3,700	N/A	N/A	N/A	N/A
1970	1,947	1,361	N/A	N/A	N/A
1971	2,200	N/A	N/A	N/A	N/A
1972	1,049	884	N/A	N/A	N/A
1973	3,000	N/A	N/A	N/A	N/A
1974	2,000	1,290	N/A	N/A	N/A
1975	1,541	1,181	N/A	N/A	N/A
1976	282	78	N/A	N/A	N/A
1977	131	N/A	N/A	N/A	N/A
1978	3,295	2,260	N/A	N/A	N/A
1979	1,392	938	N/A	N/A	N/A
1980	3,894	2,870	N/A	N/A	N/A
1981	1,243	790	N/A	N/A	N/A
1982	6,000	N/A	N/A	N/A	N/A
1983	7,000	N/A	N/A	N/A	N/A
1984	952	N/A	N/A	N/A	N/A
1985	1,300	N/A	N/A	N/A	N/A
1986	13,000	5,930	N/A	N/A	N/A
1987	1,600	N/A	N/A	N/A	N/A
1988	1,446	N/A	N/A	N/A	N/A
1989	1,720	N/A	N/A	N/A	N/A
1990	1,739	N/A	N/A	N/A	N/A
1991	2,128	N/A	N/A	N/A	N/A
1992	2,290	N/A	N/A	N/A	N/A

Water	Flow Duration and Average Flow (cfs)				
Year	Peak	1-Day	3-Day	5-Day	10-Day
1993	2,133	N/A	N/A	N/A	N/A
1994	787	N/A	N/A	N/A	N/A
1995	15,000	7,580	N/A	N/A	N/A
1996	2,215	1,417	N/A	N/A	N/A
1997	7,950	3,550	N/A	N/A	N/A
1998	7,521	4,434	N/A	N/A	N/A
1999	1,771	1,182	N/A	N/A	N/A
2000	4,010	3,020	1,740	1,339	893
2001	983	636	411	317	239
2002	1,120	817	533	464	371
2003	1,730	1,060	586	445	335
2004	1,910	1,220	718	505	437
2005	1,750	1,290	1,010	803	526
2006	7,200	4,200	2,067	1,424	966
2007	2,230	1,140	676	498	297
2008	2,620	1,200	765	530	322
2009	1,268	781	585	438	373

N/A = data not available or not estimated
One-day flows for 1986 and 1995 based on rainfall-runoff modeling for these two flood events. Peaks for 1950 to 1995 developed as detailed in Reference 2, Appendix 3 for Peer Review. One-day flows between 1968 and 1981 developed as described in Section 4.1 of this report.

The peak flow frequency curve developed for the Peer Review used data for the Winding Way and Del Paso Heights locations as if the locations were interchangeable. **Tables 2** and **5** list the 2- and 1% flood peak and associated one-day flow data for Arcade Creek at Winding Way and at the Del Paso Heights gage; the differences in magnitude are less than 1%. For this study, the differences in flow between upstream and downstream location are treated as negligible. **Table 7** tabulates the annual peak and flow duration data for Arcade Creek at Winding Way/Del Paso Heights gage used for the flow frequency analysis presented in this study.

5. DRY CREEK AT VERNON STREET FLOW FREQUENCY ANALYSIS

5.1 Regional Frequency Computation for Dry Creek. The annual peak flows for 60 years of recorded and estimated values for Dry Creek at Vernon Street, Roseville, gage are plotted on **Plate 5**, the annual rainflood frequency curves for Dry Creek at Vernon Street. Considering the lack of annual duration data in the record for Dry Creek, an approach was needed to determine the plotting positions of the previously recorded and estimated annual 1-day flow data in relation to the peak flows. The HEC program, REGFQ (Regional Frequency Computation (**Reference 7**)) was used to develop a reasonable estimate of the plotting positions for those one-day flows. The flows listed in **Table 6** were used as input to the REGFQ computer program. Output from the program is shown on **Plate 4** with the one-day flows from **Table 6** plotted using median plotting positions. The missing one-day flows are indicated as gaps where the REGFQ program made estimates of their magnitudes.

Table 7
Arcade Creek at Winding Way/Del Paso Heights Gage
Flow Duration Data

Water	Flow Duration and Average Flow (cfs)				
Year	Peak	1-Day	3-Day	5-Day	10-Day
1962	2,450	N/A	N/A	N/A	N/A
1963	2,500	N/A	N/A	N/A	N/A
1964	1,400	772	431.7	266.5	134.8
1965	1,450	897	593.3	419.8	250.3
1966	625	360	155.7	103.8	65.4
1967	2,000	1,020	574.7	471.4	360.8
1968	568	289	162.3	112.4	63.5
1969	1,570	1,280	937.0	664.0	517.9
1970	1,600	879	455.3	313.0	247.4
1971	1,630	1,090	537.7	413.6	288.9
1972	590	408	228.0	178.0	115.6
1973	2,170	771	508.7	412.8	363.7
1974	2,050	807	317.0	241.0	197.7
1975	1,300	829	449.7	311.2	206.7
1976	200	153	56.0	51.7	27.2
1977	345	281	69.5	49.4	25.7
1978	2,390	1,270	811.0	599.0	346.3
1979	1,200	N/A	N/A	N/A	N/A
1980	1,700	N/A	N/A	N/A	N/A
1981	800	N/A	N/A	N/A	N/A
1982	3,300	N/A	N/A	N/A	N/A
1983	2,900	N/A	N/A	N/A	N/A
1984	1,650	N/A	N/A	N/A	N/A
1985	700	N/A	N/A	N/A	N/A
1986	3,800	N/A	N/A	N/A	N/A
1987	1,500	N/A	N/A	N/A	N/A
1988	1,180	N/A	N/A	N/A	N/A
1989	1,550	N/A	N/A	N/A	N/A
1990	1,080	N/A	N/A	N/A	N/A
1991	1,650	N/A	N/A	N/A	N/A
1992	2,100	N/A	N/A	N/A	N/A
1993	2,300	N/A	N/A	N/A	N/A
1994	1,250	N/A	N/A	N/A	N/A
1995	4,100	N/A	N/A	N/A	N/A
1996	1,700	1100	589.7	358.6	212.5
1997	2,270	1090	591.3	678.6	381.7
1998	3,320	1910	1,069.3	714.8	462.5
1999	1,040	527	350.0	218.6	133.5
2000	2,430	1790	740.3	549.2	309.0
2001	1,030	281	181.7	141.0	73.8
2002	1,030	543	229.7	213.4	147.7
2003	1,150	578	340.0	250.8	173.7
2004	1,340	492	224.7	149.1	108.9

Water	Flow Duration and Average Flow (cfs)				
Year	Peak	1-Day	3-Day	5-Day	10-Day
2005	1,000	661	420.3	322.4	191.7
2006	3,460	1890	835.3	538.2	373.9
2007	1,030	438	300.7	192.2	100.1
2008	1,700	745	373.0	242.4	133.4
2009	N/A	388	208.0	140.4	125.8
Note: N/A = data not available Peak flows for 1962, 1963, 1982 to 1984, 1986 to 1995 from the Sacramento County Winding Way gage. Peak flows for 1972 to 1981 and 1985 estimated based on Dry Creek at Vernon gage.					

5.2 Updated Dry Creek Peak Flow Frequency Curve. **Table 3** lists the statistics for the peak flow frequency curve, for the Peer Review analysis and the FFA statistics for 60 years of estimated and observed peak flows. The peak flow frequency statistics did not change by much with the addition of 14 years of data. The decision was made not to change the peak flow frequency curve statistics used with the Peer Review adjusted gage measurement record for several reasons. The peak flow record includes many estimated peak flows. Also, the flow frequency curve for the adjusted gage measurement record was developed based on analysis by engineers from several government agencies and engineering firms. Further analysis should be done before making the decision to change the statistics.

5.3 Dry Creek One-Day Flow Frequency Curve. The previously recorded and estimated annual one-day flows for Dry Creek at Vernon Street listed in **Table 6** were plotted on **Plate 4** using the plotting positions estimated from the REGFQ run. Statistics were tested to develop a one-day flow frequency curve that was representative of the plotted one-day data points above the 50% chance exceedence on **Plate 4**. Guidance for the upper end of the frequency curve came from the “Excel spreadsheet” with the one-day flows associated with the 2-, 1-, 0.5-, and 0.2% flood hydrographs for Dry Creek at Vernon Street. These “Excel spreadsheet” one-day flows for Vernon Street are listed in **Table 5.** While the Peer Review peak flow frequency curve has a positive skew, the volume frequency curves developed for the current analysis have zero or negative skews, more typical of flow frequency curves for the region. A zero skew is used for the one-day flow frequency curve. The mean and standard deviation selected for the straight line curve produce a one-day flow frequency curve that fits very well to the observed and estimated one-day flows plotted on **Plate 4** as well as to the “Excel spreadsheet” one-day flows listed in **Table 5.** The final statistics selected for the one-day flow frequency curve are listed on **Plate 4**.

5.4 Dry Creek Five- and Ten-Day Flow Frequency Curves. As discussed in Section 7 below, the synthetic 8-flood series hydrographs for Dry Creek at NEMDC were developed as part of the AR CF GRR. The preliminary 8-flood series hydrographs for Dry Creek at NEMDC were flood runoff from 10-day storms using methodology in the Sacramento City/County Drainage Manual, **Reference 8**. Development of these hydrographs is discussed in the Natomas General Reevaluation Report Hydrology

Appendix, **Reference 9**. The 10-day flood hydrographs were later reshaped into a main 5-day wave preceded by a smaller 5-day wave, as discussed in the AR CF GRR Synthetic Hydrology Technical Documentation Appendix (**Reference 1**)**.** The flood hydrographs were reshaped to conform to the valley-wide flood hydrographs developed for the Sacramento and San Joaquin River Basins Comprehensive Study (**Reference 10**). While the flood hydrograph shapes changed, the 5- and 10-day flood volumes for Dry Creek at NEMDC did not. **Tables 13** and **17** in **Reference 1** list the 5- and 10-day volumes, respectively, of the synthetic 8-flood series hydrographs for Dry Creek at NEMDC**. Table 8** below lists these flood volumes in acre feet. Flood volumes listed in other tables in this report are in average day cfs.

Computer modeling was used to develop a flood reproduction of the New Year January 1997 (NY ’97), 29 December 1996 to 3 January 1997) storm and flood event for Dry and Arcade creeks as part of the AR CF GRR Synthetic Hydrology Technical Documentation (**Reference 1**). The reshaped 8-flood series 10-day flood hydrographs for Dry Creek, with the main 5-day wave and smaller 5-day wave, are based on the shape of the NY ’97 5-day flood reproduction hydrographs for Dry Creek. The computer model for the NY ’97 flood reproduction computed a flood hydrograph for each Dry Creek subbasin and index point. **Figure 1** displays the NY ’97 flood hydrograph computed for Dry Creek at Vernon Street. The 5-day volume for the NY ’97 flood hydrograph for Dry Creek at Vernon Street is 12,459 ac-ft, and the corresponding 5-day flood hydrograph for Dry Creek down at NEMDC is 17,387 ac-ft.

Each of the 8-flood series 5-day volumes for Dry Creek at Vernon Street is computed by multiplying the 8-flood series 5-day flood volume for Dry Creek at NEMDC in **Table 8** by the ratio of the NY ’97 5-day flood volume at Vernon Street to the NY ’97 5-day flood volume at NEMDC. For example, the 50% 5-day flood volume for Dry Creek at Vernon Street is computed by multiplying the 50% flood 5-day volume at NEMDC (9,250 ac-ft in **Table 8**) by the ratio 0.717 (12,460 ac-ft divided by 17,400 ac-ft). The 50% 5-day flood volume for Dry Creek at Vernon Street is about 6,628 ac-ft or 668 average cfs. Each of the 8-flood series 5-day volumes was computed the same way.

The 8-flood series 10-day volumes for Dry Creek at Vernon Street are computed by multiplying the 8-flood series 10-day flood volume for Dry Creek at NEMDC in **Table 8** by the same ratio as above. For example, the 50% 10-day flood volume for Dry Creek at Vernon Street is computed by multiplying the 50% 10-day flood volume at NEMDC (11,000 ac-ft in **Table 8**) by the ratio 0.717. The 50% 10-day flood volume for Dry Creek at Vernon Street is about 7,882 ac-ft or 397 average cfs. Each of the 8-flood series 10-day volumes was computed the same way.

Table 8
Five- and Ten-Day Flood Volumes for Synthetic 8-Flood Series

	8-Flood Series Five-Day Volumes (ac-ft)							
	50%	20%	10%	4%	2%	1%	0.50%	0.20%
Dry Cr. at NEMDC	9,250	15,450	19,800	26,600	31,000	35,600	39,800	47,200
Arcade Cr. at NEMDC	3,400	5,310	6,650	8,430	9,710	11,050	12,300	14,260
	8-Flood Series Ten-Day Volumes (ac-ft)							
	50%	20%	10%	4%	2%	1%	0.50%	0.20%
Dry Cr. at NEMDC	11,000	18,300	23,600	32,700	38,200	43,900	49,100	58,700
Arcade Cr. at NEMDC	4,220	6,570	8,190	10,300	11,900	13,600	15,100	17,600

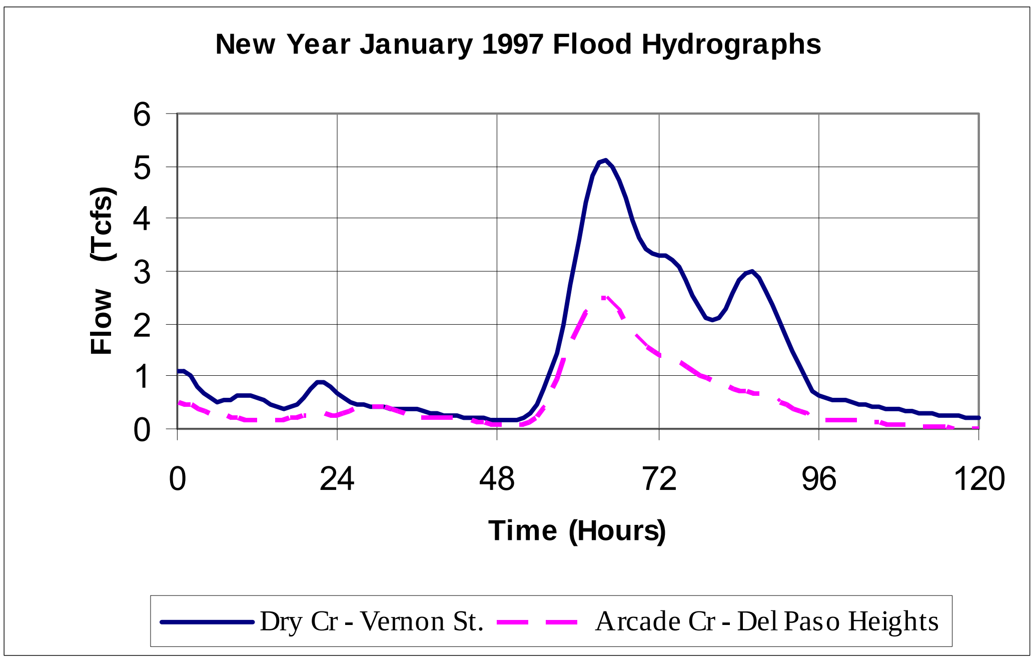

Figure 1. New Year January 1997 Flood Hydrographs Modeled for Dry Creek at Vernon Street and Arcade Creek at Del Paso Heights Gage

The synthetic 8-flood series hydrographs for Dry Creek at Vernon Street, Roseville, were rebalanced to produce higher peak flows. The 8-flood series 5- and 10-day hydrograph volumes remain unchanged. The 8-flood series 5- and 10-day flood volumes, computed as described in the above paragraphs, were plotted as average flows in cfs on **Plate 4**, the flow frequency curves for Dry Creek at Vernon Street. Statistics were tested to develop flow frequency curves that passed smoothly through these flood volumes. The final statistics and flow frequency curves for the 5- and 10-day flood volumes are displayed on **Plate 4**. There are only ten years (2000 – 2009) of observed annual 5- and 10-day flows for the Vernon Street gage. This time period is insufficient to plot the observed flows on **Plate 4**. The ten annual data points for 5- and 10-day flows, as distributed by the REGFQ program, do not match the 5- and 10-day flow frequency curves and are not shown on **Plate 4**.

5.5 Dry Creek Three-Day Flow Frequency Curve. There are only ten years of recorded data for Dry Creek at Vernon Street for which annual 3-day flows could be computed. This is not a long enough record on which to base a flow frequency curve. The statistics for the 3-day flow frequency curve needed to be somewhere in-between the statistics for the 1-day and the 5-day flow frequency curves, in order for develop reasonable 3-day flood volumes that would not be too difficult to balance as part of the 5-day flood waves for the 8-flood synthetic series at Vernon Street. A preliminary set of statistics for the 3-day flow frequency curve was selected such that the mean peak flow, standard deviation, and skew were between those for the 1-day and 5-day statistics. During the process of balancing the 8-flood series hydrographs, the 3-day volumes needed to be changed by minor amounts to create realistically shaped hydrographs. The 3-day flow frequency statistics on **Plate 4** are those used for the 3-day volumes of the final balanced hydrographs. By coincidence, the plotting positions from the REGFQ program for the ten annual 3-day flows fit along the 3-day frequency curve pretty well and are included on **Plate 4**.

6. ARCADE CREEK AT WINDING WAY/DEL PASO HEIGHTS FLOW FREQUENCY ANALYSIS

6.1 Regional Frequency Computation for Arcade Creek. The annual peak flows for 47 years of record for Arcade Creek at Winding Way/Del Paso Heights are plotted on **Plate 5**, the rainflood frequency curves for Arcade Creek. While more annual duration data are available for Arcade Creek than for Dry Creek, 19 years of duration data are missing for the years that the USGS gage at Del Paso Heights was not in operation. The REGFQ program (**Reference 7**) was also used to develop reasonable estimates of the annual 1-, 3-, 5-, and 10-day flows for the missing years. The annual flows listed in **Table 7** for Arcade Creek were used as input to the Regional Frequency Computation program. **Plate 5** shows the median plotting positions for the annual duration data listed in **Table 7**. Estimates for duration data for the missing years are indicated as gaps between the recorded data points.

6.2 Updated Arcade Creek Peak Flow Frequency Curve. **Table 4** lists the statistics for the Arcade Creek peak flow frequency curve, for the Peer Review analysis and the FFA statistics for 47 years of peak flows. Most of the peak flows were recorded at the Del Paso Heights gage, some were recorded at the Sacramento County gage at Winding Way, and a few were estimated. Updating the peak flow record with 13 more years of data at the Del Paso Heights gage did not make much difference in the frequency curve. It was decided to use the Peer Review statistics, from the FFA analysis for 34 years of record.

6.3 Arcade Creek 1-Day Flow Frequency Curve. An FFA analysis could not be performed for the one-day flow duration with 19 years missing from the record. The FFA analysis for the Arcade Creek peak flow record showed that 1976 was a low outlier. The REGFQ program was used for the Arcade Creek peak and 1-day flow data with low outlier 1976 removed. The adjusted frequency statistics for the one-day duration matched the plotted data points and were used for the one-day flow frequency curve. The flow frequency statistics, one-day flow frequency curve, and recorded one-day flows for Arcade Creek at the Del Paso Heights gage are shown on **Plate 5**.

6.4 Arcade Creek Five- and Ten-Day Flow Frequency Curves. The frequency curves for the 5- and 10-day volumes for Arcade Creek at Winding Way/Del Paso Heights gage were developed in the same manner as the 5- and 10-day frequency curves for Dry Creek at Vernon Street, Roseville. **Table 8** lists the synthetic 8-flood series 5- and 10-day flood volumes for Arcade Creek at NEMDC, which were developed for the Natomas GRR Hydrology Appendix, **Reference 9**. These flood volumes are still used for the present analysis.

The computer model for the NEMDC tributaries was used to develop a flood reproduction of the NY ’97 flood hydrograph for Arcade Creek as well as for Dry Creek (in **Reference 1**). The computer model developed a flood hydrograph for each Arcade Creek subbasin and index point. **Figure 1** displays the NY ’97 flood hydrograph computer for Arcade Creek at the Del Paso Heights gage location. The 5-day volume for the NY ’97 flood hydrograph for Arcade Creek at the Del Paso Heights gage is 5,300 ac-ft, and the corresponding 5-day flood hydrograph for Arcade Creek down at NEMDC is 6,098 ac-ft.

Each of the 8-flood series 5-day volumes for Arcade Creek at Del Paso Heights gage is computed by multiplying the 8-flood series 5-day flood volume for Arcade Creek at NEMDC in **Table 8** by the ratio of the NY ’97 5-dayflood volume at Del Paso Heights gage to the NY ’97 5-day flood volume at NEMDC. For example, the 50% 5-day flood volume for Arcade Creek at Del Paso Heights gage is computed by multiplying the 50% 5-day volume at NEMDC (3,400 ac-ft in **Table 8**) by the ratio 0.869 (5,300 ac-ft divided by 6,098 ac-ft). The 50% 5-day flood volume for Arcade Creek at Del Paso Heights gage is about 5,300 ac-ft or 300 average cfs. Each of the 8-flood series 5-day volumes was computed the same way.

The 8-flood series 10-day volumes for Arcade Creek at Del Paso Heights gage are computed by multiplying the 8-flood series 10-day flood volume for Arcade Creek at NEMDC in **Table 8** by the same ratio as above. For example, the 50% 10-day flood volume for Arcade Creek at Del Paso Heights gage is computed by multiplying the 50% 10-day flood volume at NEMDC (4,220 ac-ft in **Table 8**) by the ratio 0.869. The 50% 10-day flood volume for Arcade Creek at Del Paso Heights gage is about 3,667 ac-ft or 185 average cfs. Each of the 8-flood series 10-day volumes was computed the same way.

The synthetic 8-flood series hydrographs for the Del Paso Heights gage location were rebalanced to produce higher peak flows, but the 5- and 10-day hydrograph volumes were not changed in the process. The 8-flood series 5- and 10-day flood volumes, computed as described in the above paragraphs, were plotted as average flows in cfs on **Plate 5,** the flow frequency curves for Arcade Creek at Winding Way/Del Paso Heights. Statistics were tested to develop flow frequency curves that passed smoothly through these flood volumes. The final statistics, 5- and 10-day flow frequency curves, and recorded 5- and 10-day flows for Arcade Creek at the Del Paso Heights gage are shown on **Plate 5.**

The annual 5-day duration data observed for Arcade Creek fit along the 5-day flow frequency curve on **Plate 5**. The observed annual 10-day volumes for Arcade Creek at Del Paso Heights gage are slightly higher than the 10-day flow frequency curve. The 10-day volumes for the synthetic 8-flood series hydrographs were based on rainfall-runoff modeling of a series of 10-day storms for the NEMDC tributaries, not on analysis of flow frequency data for Arcade Creek. The 10-day storms were based on criteria in the Sacramento City/County Drainage Manual, Volume 2, Hydrology Standards (**Reference 8**). The development of the 10-day storms and runoff hydrograph volumes was presented in the Natomas General Reevaluation Report Hydrology Appendix (**Reference 9**).

6.5 <u>Arcade Creek Three-Day Flow Frequency Curves</u>. The recorded annual 3-day volumes for Arcade Creek at Del Paso Heights gage were plotted on **Plate 5** using the plotting positions output from the REGFQ program. The statistics for the flow frequency curve needed to be somewhere in-between the statistics for the 1-day and the 5-day flow frequency curves, in order to develop reasonable 3-day flood volumes that would not be too difficult to balance as part of the 5-day flood waves for the 8-flood synthetic series at the Del Paso Heights gage. A preliminary set of statistics for the 3-day flow frequency curve was selected such that the mean peak flow, standard deviation, and skew were between those for the 1-day and 5-day statistics and were representative of the plotted annual data points. During the process of balancing the 8-flood series hydrographs, the 3-day volumes needed to be changed by minor amounts to create realistically shaped hydrographs. The 3-day flow frequency statistics on **Plate 5** are those used for the 3-day volumes of the final balanced hydrographs.

7. BALANCED HYDROGRAPH DEVELOPMENT FOR DRY AND ARCADE CREEKS

This section discusses development of the balanced hydrographs to the flow frequency curves displayed on **Plates 4 and 5** for the synthetic 8-flood series at Dry Creek at Vernon Street and at Arcade Creek at the Del Paso Heights gage. For consistency with the Comprehensive Study, the computed New Year January 1997 flood hydrographs for Dry Creek at Vernon Street and Arcade Creek at Del Paso Heights gage were used as the pattern hydrographs for the synthetic 8-Flood Series.

7.1 <u>Peak Flows</u>. The balanced flood hydrographs include the peak flows listed below in **Tables 9 and 10**. The peak flows for Dry Creek (**Table 9**) are the same as the Adjusted Gage Measurement peak flows on **Table 1** and the same as those on the flow frequency curve defined by the Adjusted Gage Measurement flow frequency statistics on **Table 3**. The peak flows for Arcade Creek (**Table 10**) are the same as the Peer Review FFA Program Results on **Table 2** and those on the flow frequency curve defined by the Peer Review FFA Statistics on **Table 4**. Hydrographs and peak flows for the downstream tributaries and local subbasins on Dry and Arcade creeks were not changed from those previously provided to Hydraulic Design Section.

Table 9

Peak and Volume Tabulation for Synthetic 8-Flood Series

Balanced Hydrographs for Dry Creek at Vernon Street (Roseville)

8-Flood Event	**Peak** (cfs)	**24-Hour** (avg cfs)	**3-Day** (avg cfs)	**5-Day** (avg cfs)	**10-Day** (avg cfs)
50%	2,010	1,360	843	665	407
20%	3,900	2,500	1,420	1,080	659
10%	5,640	3,500	1,880	1,400	854
4%	8,500	4,900	2,560	1,860	1,130
2%	11,200	6,340	3,110	2,220	1,350
1%	14,400	7,390	3,720	2,590	1,560
0.50%	18,300	8,620	4,340	2,970	1,790
0.20%	24,500	11,300	5,260	3,530	2,120

Table 10

Peak and Volume Tabulation for Synthetic 8-Flood Series

Balanced Hydrographs for Arcade Creek at Del Paso Heights Gage

8-Flood Event	**Peak** (cfs)	**24-Hour** (avg cfs)	**3-Day** (avg cfs)	**5-Day** (avg cfs)	**10-Day** (avg cfs)
50%	1,540	945	425	304	187
20%	2,420	1,460	677	491	302
10%	3,010	1,790	842	613	377
4%	3,730	2,200	1,050	771	474
2%	4,260	2,490	1,200	884	544
1%	4,770	2,780	1,350	995	613
0.50%	5,260	3,050	1,500	1,110	685
0.20%	5,900	3,410	1,680	1,250	769

7.2 Balancing to 1-, 3-, and 5-Day Durations. A spreadsheet was developed to balance the synthetic flood hydrographs to the 1-, 3-, and 5-day durations from the flow frequency curves, **Plates 4 and 5**. The synthetic hydrographs were balanced using the New Year 1997 flood hydrographs on **Figure 1**, for Dry Creek at Vernon Street and Arcade Creek at Del Paso Heights gage. A different flood hydrograph pattern was used for Dry Creek at Vernon Street for the 1-, 0.5-, and 0.2% floods; it is discussed in **Section 7.3** below.

a. 24-Hour Flow. The 1-day flow frequency curve is for the annual maximum 1-day volume, measured at the gage from midnight to midnight. The maximum 24-hour flow for the same event is almost always higher than the 1-day flow, because the maximum 24-hour flow does not normally occur exactly between midnight one day and midnight the next. 24-hour volumes were used to balance the hydrographs to prevent the peak flow from appearing too peaked with respect to the one-day volume. For the balanced hydrographs, the ratio used for 24-hour flow to maximum 1-day flow is less than 1.15. Historically, the ratio of 24-hour flow to 1-day flow is not known for Dry and Arcade creeks, because only 1-day flows were available for most flood events. The 24-hour flows used to balance the synthetic 8-flood series hydrographs are listed on **Tables 9 and 10**.

b. Three Day Flow. In the process of balancing the hydrographs at the upstream gaging stations to the 3-day volumes, the 3-day volumes were slightly modified from those volumes represented by the 3-day flow frequency curves. Except for the 50% flood hydrograph for Dry Creek at Vernon Street, the 3-day volumes listed in **Tables 9 and 10** are within 2% of the 3-day volumes for the flow frequency curves for Dry and Arcade creeks.

c. Five Day Flow. In the process of balancing the hydrographs at the Dry Creek at Vernon Street to the 5-day volumes, the 5-day volumes were slightly modified from those volumes represented by the 5-day flow frequency curves. The 5-day volumes listed in **Table 9** are between 0- and 3% of the flow frequency curve volumes. The 5-day volumes listed in **Table 10** for Arcade Creek at Del Paso Heights are the same as those represented by the 5-day flow frequency curves.

7.3 Dry Creek at Vernon Street Pattern for 1-, 0.5-, and 0.2% Event Floods. The New Year January 1997 flood hydrograph modeled for Dry Creek at Vernon Street, shown on **Figure 1** in Section 5.4 and **Figure 2** below, has a double peak. Not only is the double-peak pattern more difficult to balance, especially for the 1-, 0.5-, and 0.2% flood events, but the New Year January 1997 flood was only about a 12% chance event for Vernon Street. A flood hydrograph pattern needed to be developed that would be easier to balance for the rarer floods yet still be representative of the Dry Creek watershed.

Figure 2 shows how the composite flood hydrograph pattern was developed based on the NY ’97 flood hydrograph as well as the observed or computed flood hydrographs for the two largest floods at Vernon Street. **Figure 2** shows the NY ’97 flood hydrograph for Dry Creek at Vernon Street as well as the flood hydrographs for the February 1986 and January 1995 events. The peak flows for the three hydrographs were lined up to coincide. Using portions of the three existing flood hydrographs, the composite flood hydrograph was developed to have a reasonable shape for a single peak and recession. The composite flood hydrograph pattern displayed below balanced very well to the 1-, 0.5-, and 0.2% flood volumes.

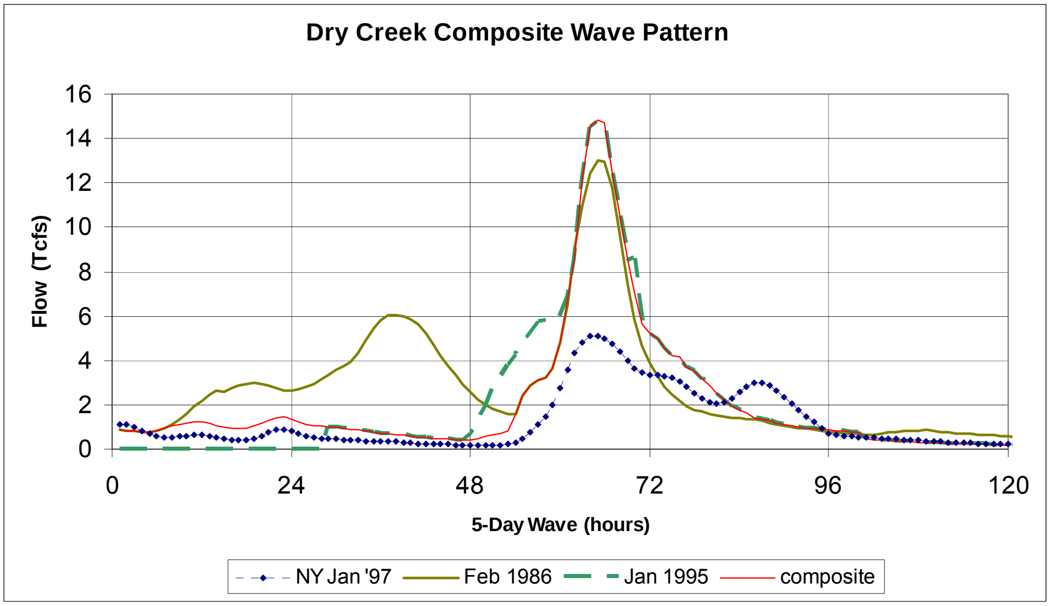

Figure 2. Development of Composite 5-Day Wave Pattern Hydrograph for Dry Creek at Vernon Street, for the 1-, 0.5-, and 0.2% Balanced Flood Hydrographs

7.4 <u>Ten- and Thirty-Day Flood Hydrographs</u>. For the Comprehensive Study, the basic pattern of all synthetic flood hydrographs was a 30-day hourly time series consisting of 6 waves, each 5 days in duration. The highest wave volume was distributed into the fourth, or main, wave. The second highest volume preceded the main wave, so the two highest waves are in the middle ten days of the 30-day hydrograph. The volume of the fourth, or main, wave for each n-flood hydrograph at NEMDC is that listed for the 5-day volume in **Table 8**. For the hydrographs at upstream index points Dry Creek at Vernon Street and Arcade Creek at Del Paso Heights gage, the 5-day main wave volumes are those listed in **Tables 9 and 10**, based on the flow frequency curves on **Plates 4 and 5**. The 5-day wave hydrographs are patterned after the modeled New Year 1997 floods, except for the Dry Creek 1-, 0.5-, and 0.2% floods. Those floods use the composite pattern shown on **Figure 2**. The volume for the second highest wave for each n-flood hydrograph is the difference between the 5-day volume and corresponding 10-day volume in **Tables 9 and 10**.

Flows on the NEMDC tributaries can be high during and immediately after a rainstorm. Without additional rainfall, the flows drop to base flow or to urban runoff levels. The NEMDC tributary flows for the four smaller waves, waves 1 and 2, 5 and 6, would be so minor that zero runoff is assumed for the 30-day hydrographs, except for the middle 10 days (Waves 3 and 4). **Figure 3** displays the 6-wave 30-day pattern balanced hydrographs for the 1% floods for Dry Creek at Vernon Street and Arcade Creek at Del Paso Heights gage.

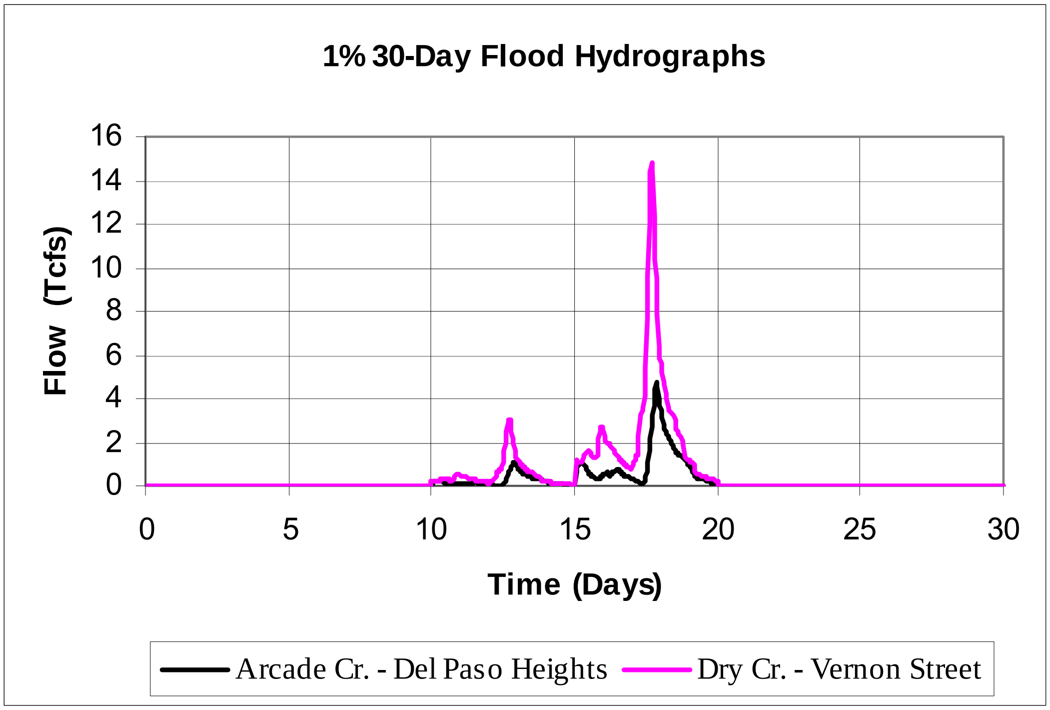

Figure 3. Synthetic 1% Flood 30-Day Wave Hydrographs for Dry Creek at Vernon Street and Arcade Creek at Del Paso Heights Gage

7.5 Routing Balanced Flood Hydrographs to NEMDC. The HEC-1 model was used to route the balanced 30-day synthetic flood hydrographs for Dry Creek at Vernon Street and Arcade Creek downstream to the NEMDC index points, combined with the local flow hydrographs along the way. The 8-flood volumes for Dry and Arcade creeks at NEMDC closely match the 5- and 10-day volumes listed on **Table 8**. The peaks and flood volumes for the flood hydrographs for Dry and Arcade creeks at NEMDC are listed on **Tables 11 and 12** below.

Table 11

Peak and Volume Tabulation for Synthetic 8-Flood Series

Dry Creek at NEMDC from Upstream Balanced Hydrographs

8-Flood Event	**Peak** (cfs)	**24-Hour** (avg cfs)	**3-Day** (avg cfs)	**5-Day** (avg cfs)	**10-Day** (avg cfs)
50%	2,170	1,840	1,170	949	543
20%	3,980	3,330	1,990	1,520	887
10%	5,330	4,520	2,620	1,960	1,150
4%	7,280	6,240	3,560	2,580	1,540
2%	8,900	7,670	4,290	3,060	1,830
1%	11,500	9,230	5,050	3,530	2,110
0.50%	14,000	10,700	5,820	4,010	2,410
0.20%	18,800	13,500	7,020	4,760	2,860

Table 12

Peak and Volume Tabulation for Synthetic 8-Flood Series

Arcade Creek at NEMDC from Upstream Balanced Hydrographs

8-Flood Event	**Peak** (cfs)	**24-Hour** (avg cfs)	**3-Day** (avg cfs)	**5-Day** (avg cfs)	**10-Day** (avg cfs)
50%	1,810	938	477	321	213
20%	2,380	1550	777	525	341
10%	2,930	1900	982	662	426
4%	3,600	2350	1230	837	535
2%	4,100	2690	1410	964	614
1%	4,620	3010	1580	1090	692
0.50%	4,970	3320	1750	1220	772
0.20%	5,570	3740	1970	1380	872

7.6 Peak Flow Attenuation. The balanced flood hydrographs with higher peaks at the upstream gaging stations on Dry and Arcade creeks do generate higher peak flows downstream at their confluences with NEMDC. With the routing process and addition of local flows, peak flows for the 50- and 20% flood events may increase in magnitude down at NEMDC. For the 10% and rarer floods, peak flows on Arcade Creek may attenuate somewhat as they travel down to NEMDC. In the modeling process, the peak flows for Dry Creek at Vernon Street for the 10% and rarer events appear to attenuate more in proportion to their magnitude. In the HEC-1 model, the 0.2% flood peak for Arcade Creek at NEMDC is 94% of the peak flow at the Paso Heights gage (5,570 cfs compared with 5,900 cfs upstream), while the Dry Creek peak flow at NEMDC is 77% of the peak flow at Vernon Street (18,800 cfs compared with 24,500 cfs upstream).

For the prior hydrology analysis of the NEMDC tributaries (**Reference 1**), peak flows for Arcade Creek at the "near Del Paso Heights" gage increased slightly downstream at NEMDC. Peak flows for Dry Creek at Vernon Street were attenuated downstream at NEMDC, but by no more than 8%, not by greater than 20%. All of the subbasin hydrographs for Dry and Arcade creeks were ratios of the computed HEC-1 subbasin flows for the modeled NY '97 historical flood. The hydrographs for Dry Creek at Vernon Street and Arcade Creek at Del Paso Heights gage were not balanced, nor were the peak flows adjusted to match existing flow frequency curves.

8. RESULTS

The Dry and Arcade creeks 30-day hydrographs for the synthetic 8-Flood Series were provided to Hydraulic Design Section. The hydrographs for the Dry Creek/Vernon Street and Arcade Creek/Del Paso Heights index points have higher peaks but the same volumes as the 8-flood series hydrographs documented in **Reference 1**. These hydrographs will be used in a hydraulic stage frequency analysis for NEMDC. They will also be used for additional hydraulic routing to upstream index points on Dry and Arcade creeks.

The synthetic 8-flood series hydrographs provided to Hydraulic Design Section are for the locations listed in **Table 13**. These locations are also shown on **Plates 2 and 3** for Dry and Arcade creeks.

Table 13

List of Locations for Balanced Synthetic 8-Flood Series Hydrographs
Provided to Hydraulic Design Section

Subbasin #	Subbasin or Index Pt. Location	D.A. (sq mi)
Dry Creek:		
511140	Dry Cr. At Sacramento-Placer County Line	88.58
512320	Sierra Cr. At Mouth	3.00
512110	Dry Cr. Local at Q Street	5.74
591010	Robla Cr. At Mouth	5.70
591011	Magpie Div. above Robla Cr.	8.90
510930	Dry Cr. Local at Rio Linda Blvd.	2.59
590620	Dry Cr. Local at NEMDC	1.97
590620	Dry Cr. Total Flow at NEMDC	116.48
Arcade Creek:		
HC15	Arcade Cr nr Del Paso Heights Gage	31.83
40	Del Paso Park Subbasin	1.91
50	North Town & Country Subbas	1.81
60	Interior Drainage above Pump 103	1.51
64	Water from Pump 103	1.51
70	Interior Drainage above Pump 159	1.22
72	Water from Pump 159	1.22
80	Interior Drainage above Pump 158	0.78
82	Water from Pump 158	0.78
90	Interior Drainage above Pump 154	1.08
92	Water from Pump 154	1.08
92C	Arcade Cr. Total Flow at NEMDC	40.14

9. LIST OF REFERENCES

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2. Dry Creek Hydrology Draft Appendix 3, "Peak Flow Frequency Relationships for Dry Creek at Vernon Street and Arcade Creek at American River." Dry Creek Hydrology/Hydraulics Peer Review and Consensus Effort. 22 May 1996.

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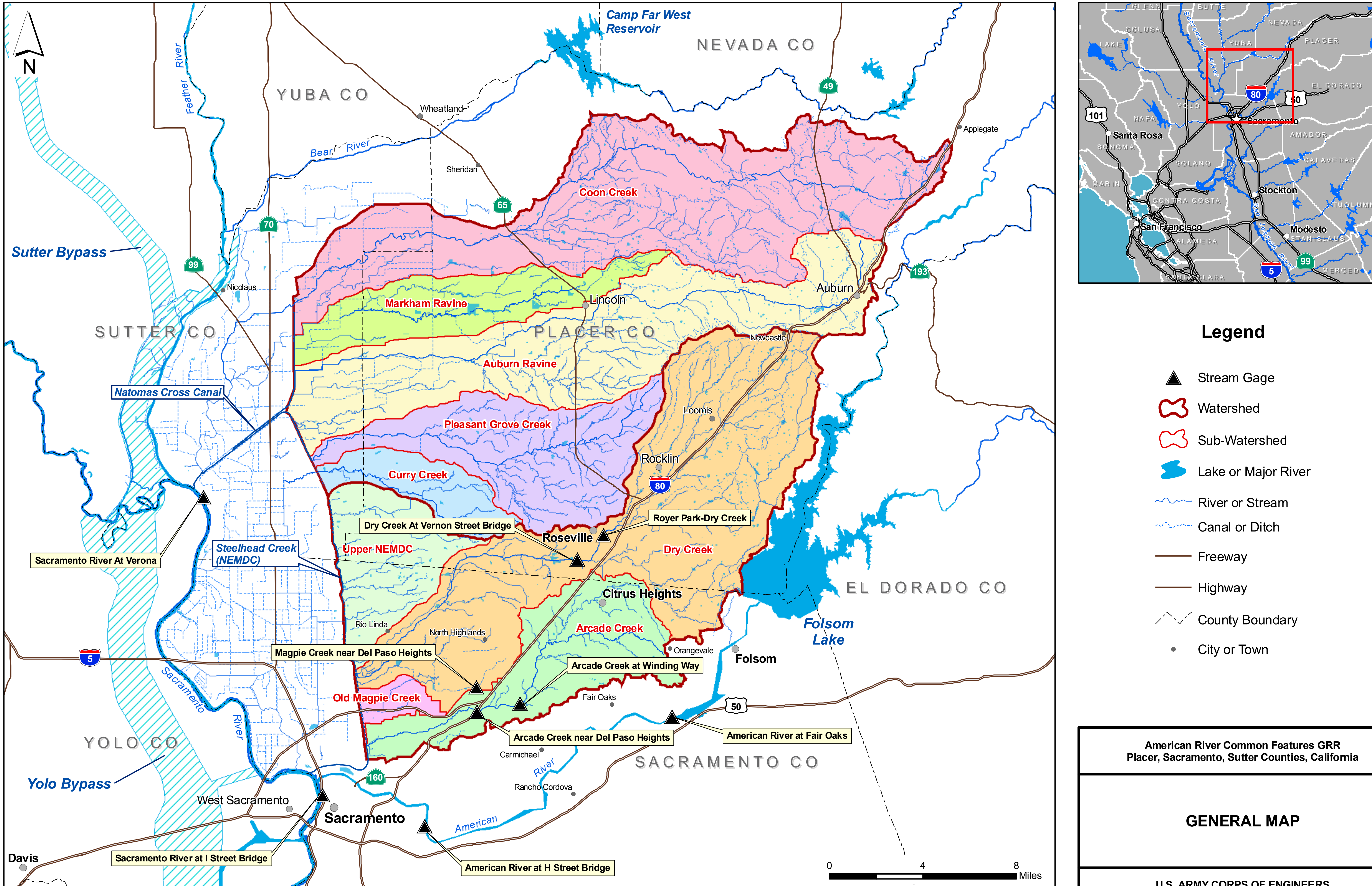

Prepared by J.S.M

American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

GENERAL MAP

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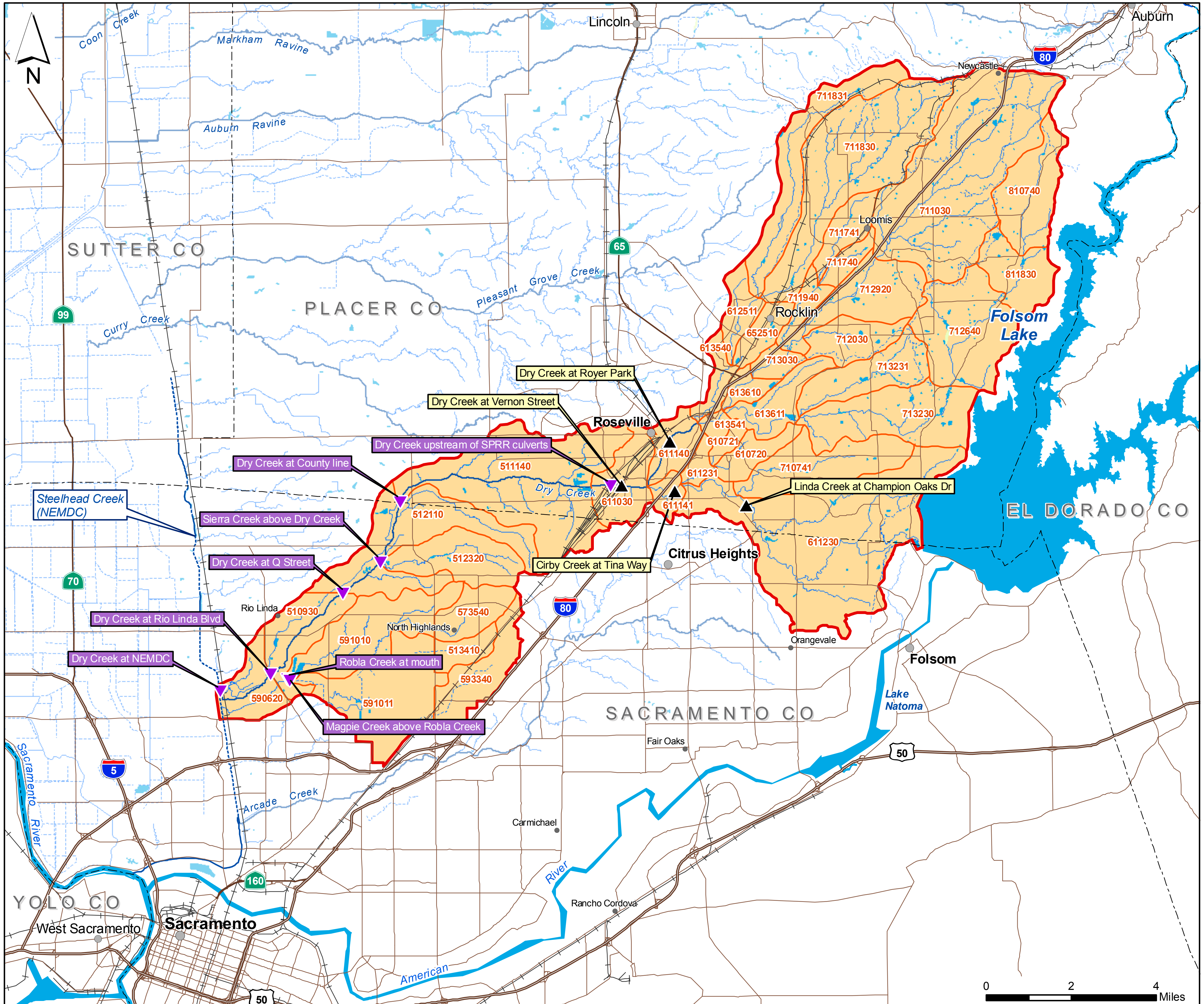
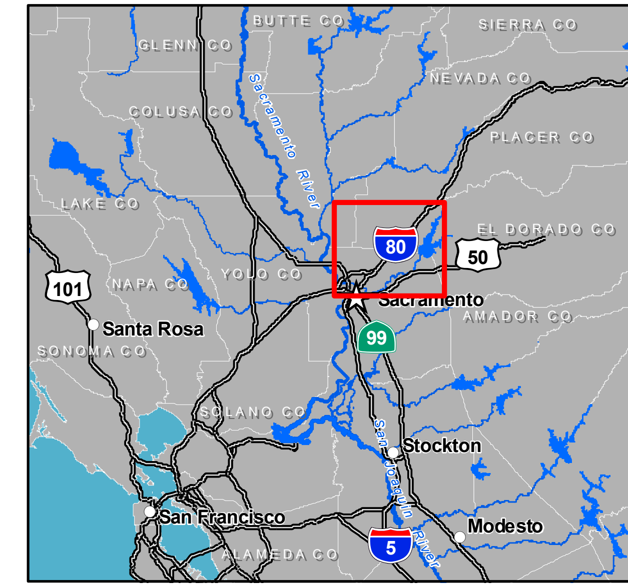
SACRAMENTO DISTRICT

NOV 2009 PLATE 1



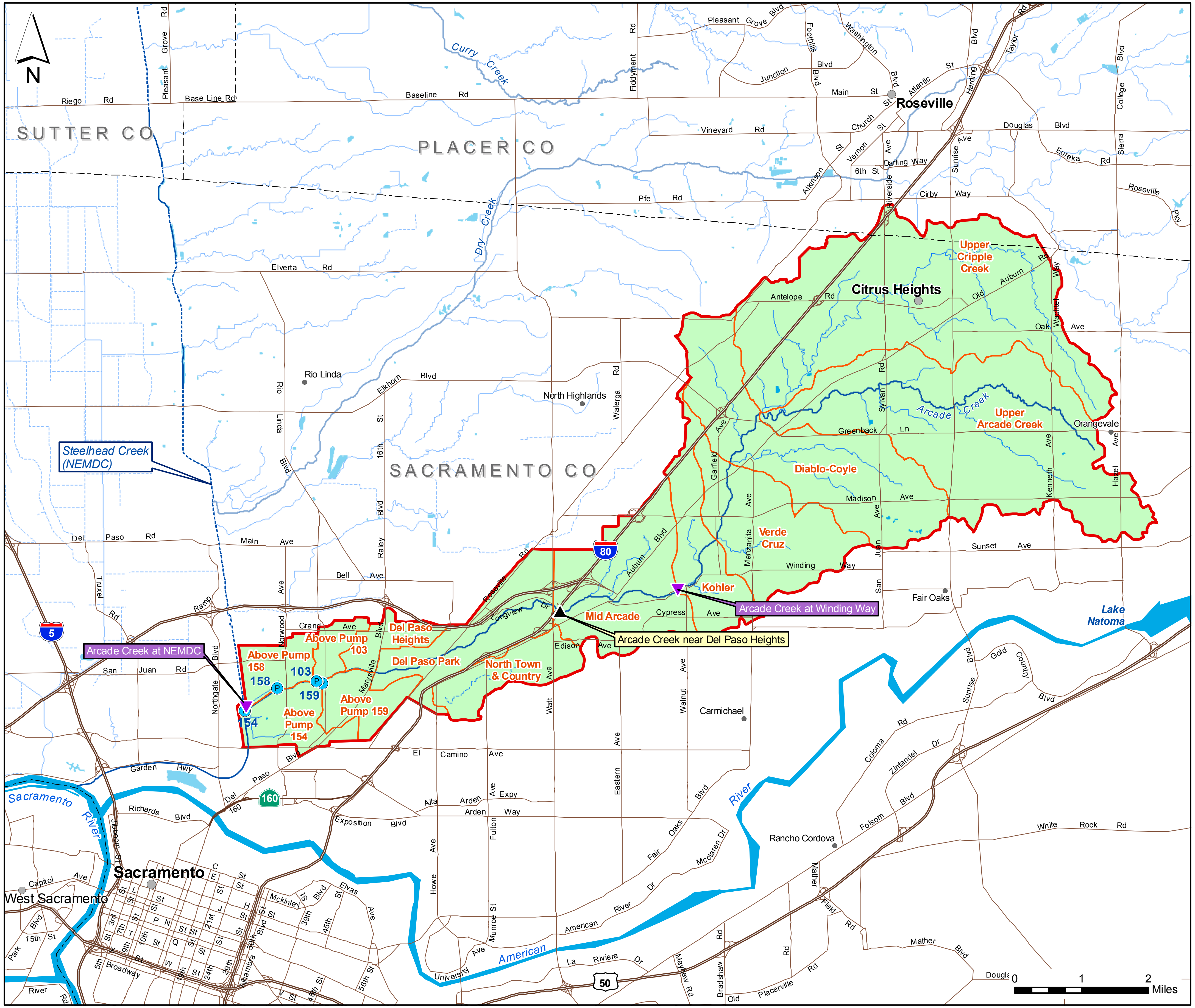


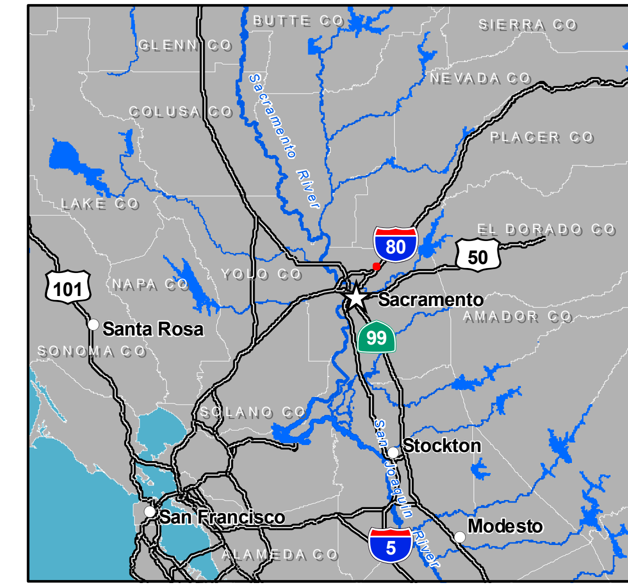
American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

DRY CREEK WATERSHED

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

NOV 2009 PLATE 2





Legend

- Stream Gage
- Model Index Point
- Pump Station
- Watershed
- Subbasin
- Waterbody
- River or Stream
- Canal or Ditch
- Freeway
- Highway
- Major Road
- County Boundary
- City or Town

American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

ARCADE CREEK WATERSHED

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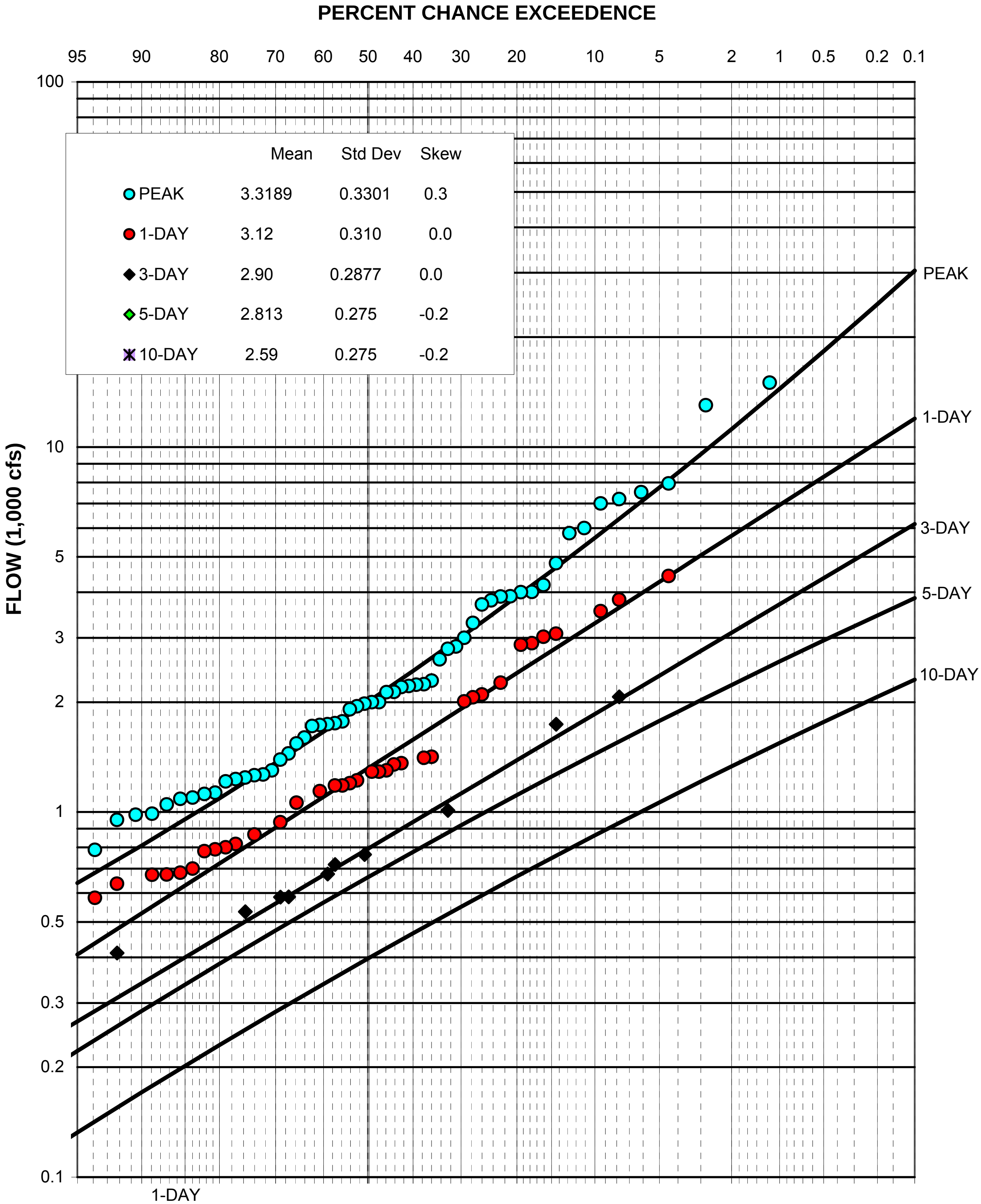
SACRAMENTO DISTRICT

NOV 2009

PLATE 3



NOTES:

1. Drainage area 78.20 sq. mi.
2. Median plotting positions, 60 years record.
3. Peak and 1-day plotting positions for WY 1950 to 2009, observed and estimated flows.
4. 1977 identified as low outlier peak.

Developed by LLW and prepared by JLB

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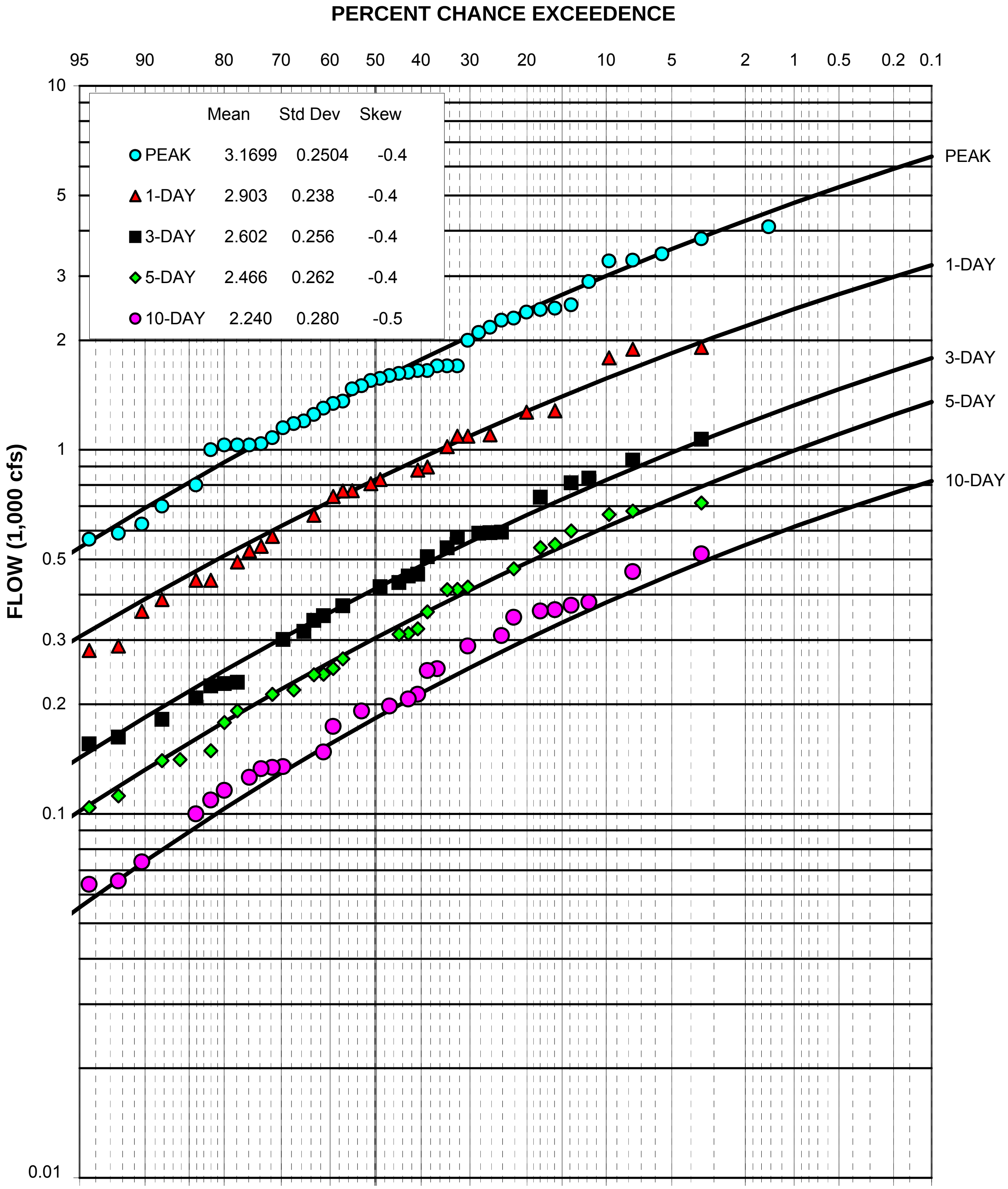
PLACER, SACRAMENTO, SUTTER COUNTIES, CA

RAINFLOOD FREQUENCY CURVES
DRY CREEK AT VERNON ST., ROSEVILLE
USGS STATION 11447293
WATER YEARS 1950 TO 2009

U.S. ARMY CORPS OF ENGINEERS
WATER CONSERVATION DISTRICT

NOV 2009



NOTES:
1. Drainage area 31.5 sq. mi.
2. Median plotting positions, 48 yrs record.
3. Period of record WY 1964-1978, 1996-2009 for 1-day to 10-day flows.
4. FFA peak flow record for 1962 to 2008.
5. 1976 identified as low outlier peak.

Developed by LLW and prepared by JLB

AMERICAN RIVER COMMON FEATURES GRR
PLACER, SACRAMENTO, SUTTER COUNTIES, CA

RAINFLOOD FREQUENCY CURVES
ARCADE CREEK NR DEL PASO HEIGHTS
USGS STATION 11447360
WATER YEARS 1962-2009

U.S. ARMY CORPS OF ENGINEERS
WATER CONSERVATION DISTRICT

NOV 2009