1.0 Technical justification

The primary benefit from implementation of the project is the reduction of the risk of significant caused by potential overtopping and failure of the dam caused by a Probable Maximum Flood (PMF).

The need to improve the dam spillway for risk reduction was brought to light by a 2006 study that showed that the PMF flow was 126,600 cfs was higher than what was previously estimated. The Probable Maximum Flood Study by Northwest Hydraulics Consultants is provided in Attachment 7-1.

Table 13. PMF Comparison

Study	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Stage (ft msl)	72-hr PMP[a] (inches)
Baseline (2005 Study)	134,100	126,600	321.95	32.2
USACE 1971[b]	136,000	n/a	n/a	29.9
DSOD 1980	112,700	102,700	319.76	30.2[c]
DSOD 1996	86,900	86,200	318.2	29.8

[a]Basin average 72-hour PMP
[b]1971 study for Garden Bar Dam Site, 4.5 miles upstream
[c]33.8 in for upper basin (above Lake Combie), 27.1 in for lower basin

Figure 7.1 Excerpt from pg. 24 of NHC report

As shown in Figure 7.1, the PMF flow that was previously thought to be in the 86,200 cfs-102,700 cfs range was established to be 126,600 cfs in 2006. The significant uptick was the result of new Hydrometerological Report Nos. 58 and 59, which established new Probable Maximum Precipitation (PMP) data for the western slope of the Sierra-Nevada Mountain Range in California.

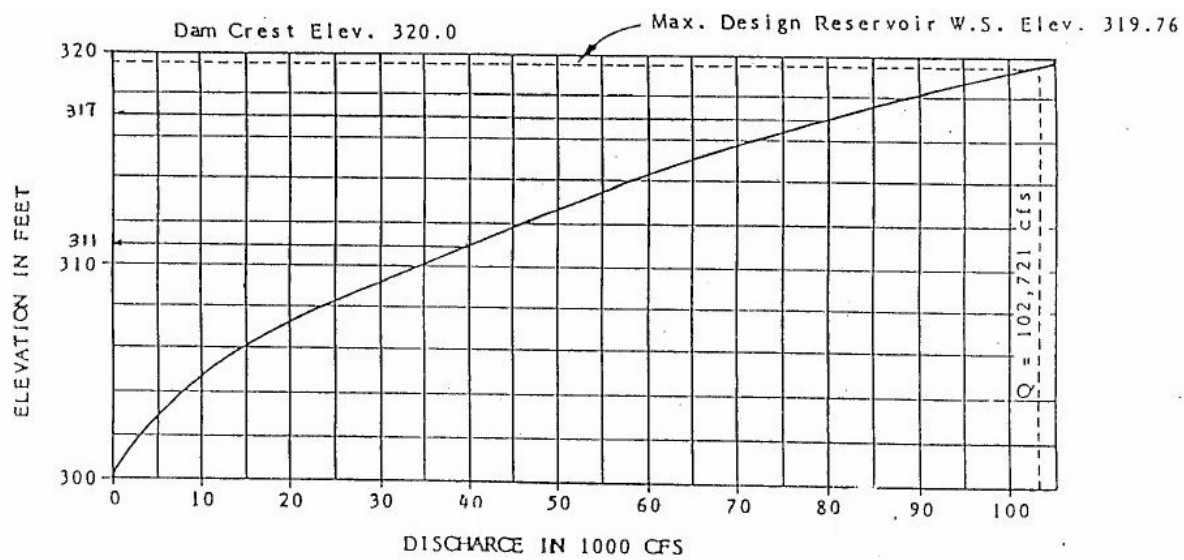


Figure 7.2 Excerpt from Appendix C of Mead & Hunt report

As compared to the PMF flow of 126,600 cfs, the maximum capacity of the Camp Far West Reservoir spillway is only approximately 106,500 cfs. A Spillway Adequacy Evaluation conducted by Mead & Hunt Inc. is provided in Attachment 7-2. Figure 7.2 shows an excerpt from the Mead & Hunt report showing the existing spillway capacity.

An Emergency Action Plan submitted by SSWD to the Federal Energy Regulatory Commission in 2006 shows that in a PMF event, the spillway would be overwhelmed, raising water elevation in the dam causing the dam embankment to be overtopped and cause flood damage in a number of communities in Yuba, Placer, Sutter, and Sacramento counties. Figure 7.3 shows the estimated inundation area in the event of PMF-induced dam failure.

The proposed project will increase the spillway capacity from 106,500 cfs to 126,500 cfs to be able to handle PMF flows. This is illustrated in Figure 7.4.

In addition to flood risk reduction, the project will lead to increased water storage and increased hydropower production.

2.0 Project physical benefits

2.1 Benefits claimed

Table 7.1 summarizes each benefit claimed from the project. Further explanation is provided under 2.2 and 2.3 below.

Table 7.1: Benefits claimed

No.	Benefit claimed	Magnitude of benefit
1	Flood risk reduction	Annual benefit of $5,134,254 per F-RAM model
2	Addition water storage	9,830 acre-feet

2.2 Flood risk reduction

In the “without project” scenario, an extreme meteorological event is projected to cause an outflow in the vicinity of 126,500 cfs (see Figures 7.1 and 7.5). Since the spillway capacity is only approximately 106,500 cfs (see Figure 7.2), it is expected that such an event would cause a rise in water elevation in the reservoir leading to overtopping of the dam embankment and dam failure. Such a failure would cause a significantly large flood wave to travel downstream.

A flood wave is more damaging than normal flooding being caused, without dam failure, by such a large event since flood waves travels at a much faster velocity and causes greater damage because of lack of adequate notice to evacuate.

An F-RAM model was run to calculate the avoided damages that would result from this project. The basis of the damages is the Emergency Action Plan map (see Figure 7.3) which shows the extents of flooding caused by dam failure. Based on the floodplain boundaries, the amount of land in each of the four affected counties (Yuba, Placer, Sutter, Sacramento) was calculated.

Using 2010 Census data, the number of residential units and agricultural land in each affected county was calculated (see Figure 7.6). This number was factored by the proportion of the county that was inundated by the flood. Subjective spatial factors were also introduced. For instance, the area in Sacramento county that is affected by the flood is known to be sparsely populated. Hence, the number of houses affected by the flood was adjusted to zero. Depreciation ratio was calculated using residential depreciation chart from HAZUS, assuming houses to be 45 years old and average condition. Note that for ease of calculations, all residential units were assumed to be “single story (no basement)”.

Three events were considered. The lowest event was a 100-year flood, which would not cause dam failure in either the without- or with-project scenario. The two higher floods were roughly equivalent to a 1-in-6,000 and 1-in-10,000 year events. The difference in damages between these two events is insignificant since dam failure occurs in both scenarios in the without-project scenario. Dam failure is avoided in both scenarios with the project in place.

Note that an extreme meteorological event will still cause some flood damage even if there is no dam failure. This is expected because the levees in the area are designed for a 100 to 200 year return interval flows while the meteorological events in question are expected to be of greater magnitude. However, damages from such flows alone (without dam failure) are projected to cause 75% of the damage expected from a dam failure. A failure-caused flood wave significantly exacerbates damage because of height and velocity of the wave. Each parameter of damage (residential units, agricultural area, road lengths, etc.) in the pre-project scenario was factored down to 75% to reflect reduced damages if the dam does not fail.

Based on the above methodology, the project will lead to eliminating flood risk from the abovementioned event for 2,750 residential units, 24,612 acres of agricultural land, and 10 miles of roadways. These benefits are tabulated in Tables 7.2 through 7.4.

2.3 Additional water storage

Installation of gates on the lowered spillway will allow SSWD to store additional water for irrigation. This water will only be stored during the summer season, and during the flood season the gates will be allowed to fold down to allow flood flows to pass through. Based on a study of 39 years of basin hydrology and reservoir inflows, it is projected that the average increase in surface water storage will be 9,830 acre-feet. Figure 7.7 shows the historic trend analysis; the difference between the blue line and red line represents the annual storage.

SSWD provides surface water irrigation to far fewer acreage that it is authorized to do. This is because, in most years, adequate surface water is not available. The lands that do not get surface water ordinarily resort to groundwater pumping. The availability of 9,830 additional acre-feet of storage will allow the groundwater users to switch to surface water, thus making an equivalent amount of groundwater available for drinking water purposes.

This benefit is tabulated in Table 7.5.

2.4 Annual Project Physical Benefits (Table 7 from PSP)

Table 7.2: Annual Benefits – Flood Reduction/Residential units

Annual Project Physical Benefits			
Project Name: Camp Far West Spillway Improvement Project			
Type of Benefit Claimed: Flood Risk Reduction			
Measure of Benefit Claimed (Name of Units): Number (of residential units that will be removed from flood risk area)			
Additional Information About this Measure: Typical lifetime of flood projects is 50 years.			
(a)	(b)	(c)	(d)
	Physical Benefits		
Year	No. of residential units affected in without-project scenario	No. of residential units affected in with-project scenario	Change Resulting from Project (b) – (c)
2012	11,000	8,250	2,750
2013	11,000	8,250	2,750
2014	11,000	8,250	2,750
--Continue benefits for the project lifetime of 50 years--			
2062	11,000	8,250	2,750
Comments: Details of the above calculation are available in the F-RAM model provided in Attachment 8			

Table 7.3: Annual Benefits – Flood Reduction/Agricultural area

Annual Project Physical Benefits			
Project Name: Camp Far West Spillway Improvement Project			
Type of Benefit Claimed: Flood Risk Reduction			
Measure of Benefit Claimed (Name of Units): Acres (of agricultural land removed from flood risk area)			
Additional Information About this Measure: Typical lifetime of flood projects is 50 years.			
(a)	(b)	(c)	(d)
	Physical Benefits		
Year	Acres of agricultural land affected in without-project scenario	Acres of agricultural land affected in with-project scenario	Change Resulting from Project (b) – (c)
2012	98,448	73,836	24,612
2013	98,448	73,836	24,612
2014	98,448	73,836	24,612
--Continue benefits for the project lifetime of 50 years--			
2062	98,448	73,836	24,612
Comments: Details of the above calculation are available in the F-RAM model provided in Attachment 8			

Table 7.4: Annual Benefits – Flood Reduction/Roadways

Annual Project Physical Benefits			
Project Name: Camp Far West Spillway Improvement Project			
Type of Benefit Claimed: Flood Risk Reduction			
Measure of Benefit Claimed (Name of Units): Miles (of roadways removed from flood risk area)			
Additional Information About this Measure: Typical lifetime of flood projects is 50 years.			
(a)	(b)	(c)	(d)
	Physical Benefits		
Year	Miles of roadways affected in without-project scenario	Miles of roadways affected in with-project scenario	Change Resulting from Project (b) – (c)
2012	38.5	28.9	9.6
2013	38.5	28.9	9.6
2014	38.5	28.9	9.6
--Continue benefits for the project lifetime of 50 years--			
2062	38.5	28.9	9.6
Comments: Details of the above calculation are available in the F-RAM model provided in Attachment 8			

Table 7.5: Annual Benefits – Additional water storage

Annual Project Physical Benefits			
Project Name: Camp Far West Spillway Improvement Project			
Type of Benefit Claimed: Additional water storage			
Measure of Benefit Claimed (Name of Units): Acre-feet (of additional water stored)			
Additional Information About this Measure: Typical lifetime of flood projects is 50 years. Water storage is an additional benefit of what is principally a flood risk reduction project.			
(a)	(b)	(c)	(d)
	Physical Benefits		
Year	Additional storage over baseline Without Project	Additional storage over baseline With Project	Change Resulting from Project (c) – (b)
2012	0	9,830	9,830
2013	0	9,830	9,830
2014	0	9,830	9,830
--Continue benefits for the project lifetime of 50 years--			
2062	0	9,830	9,830
Comments: The additional storage of water leads to a reduction of groundwater usage among SSWD's agricultural supply area			

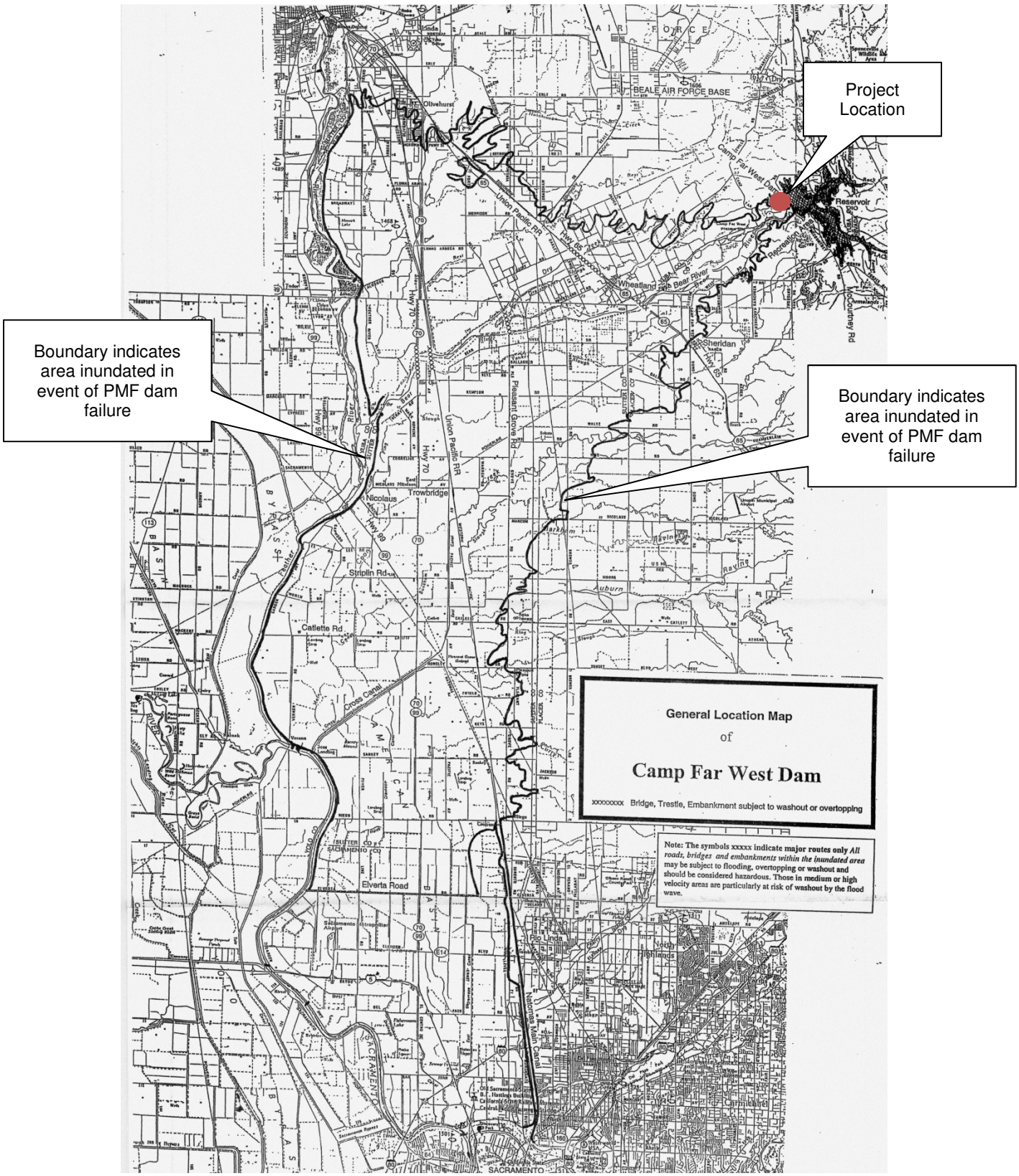


Figure 7.3 Estimated area affected by dam failure in PMF event

Source: Emergency Action Plan prepared by SSWD

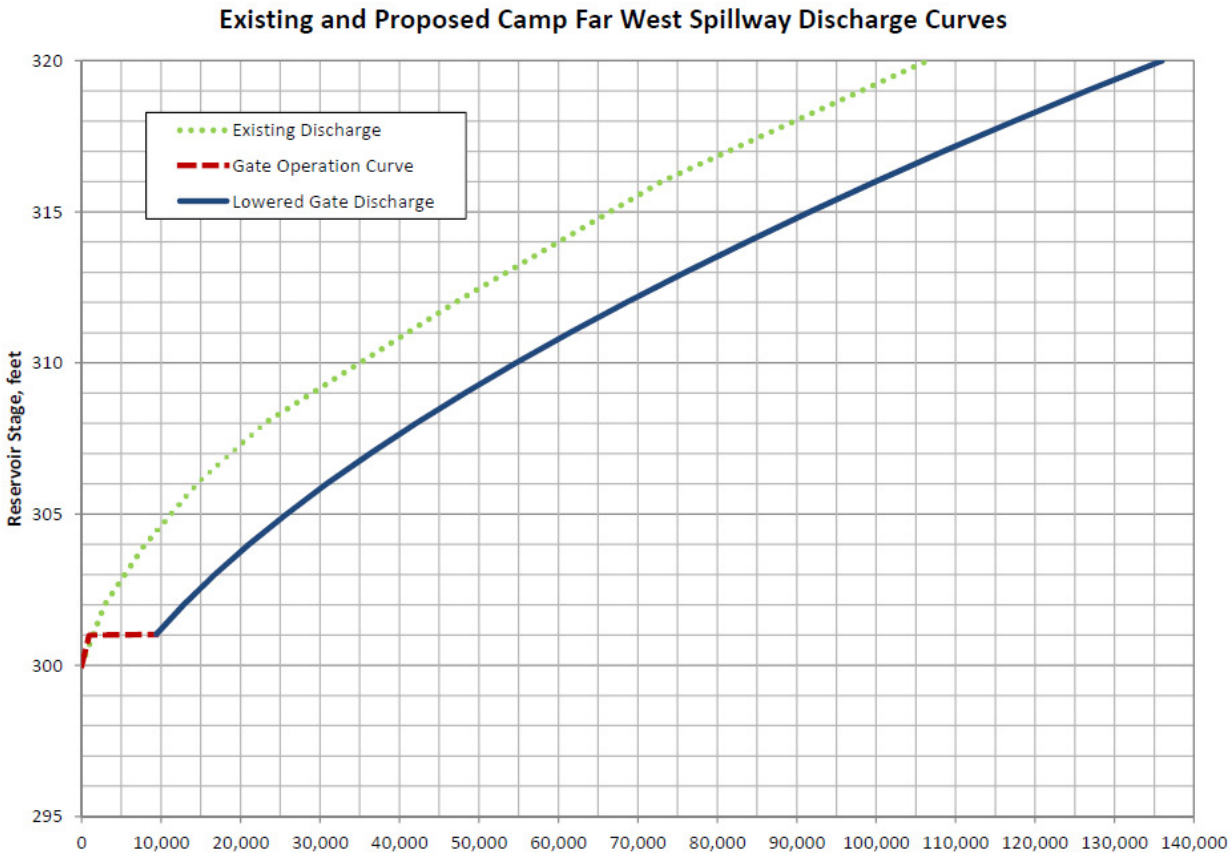


Figure 7.4 Increase in spillway capacity as result of project

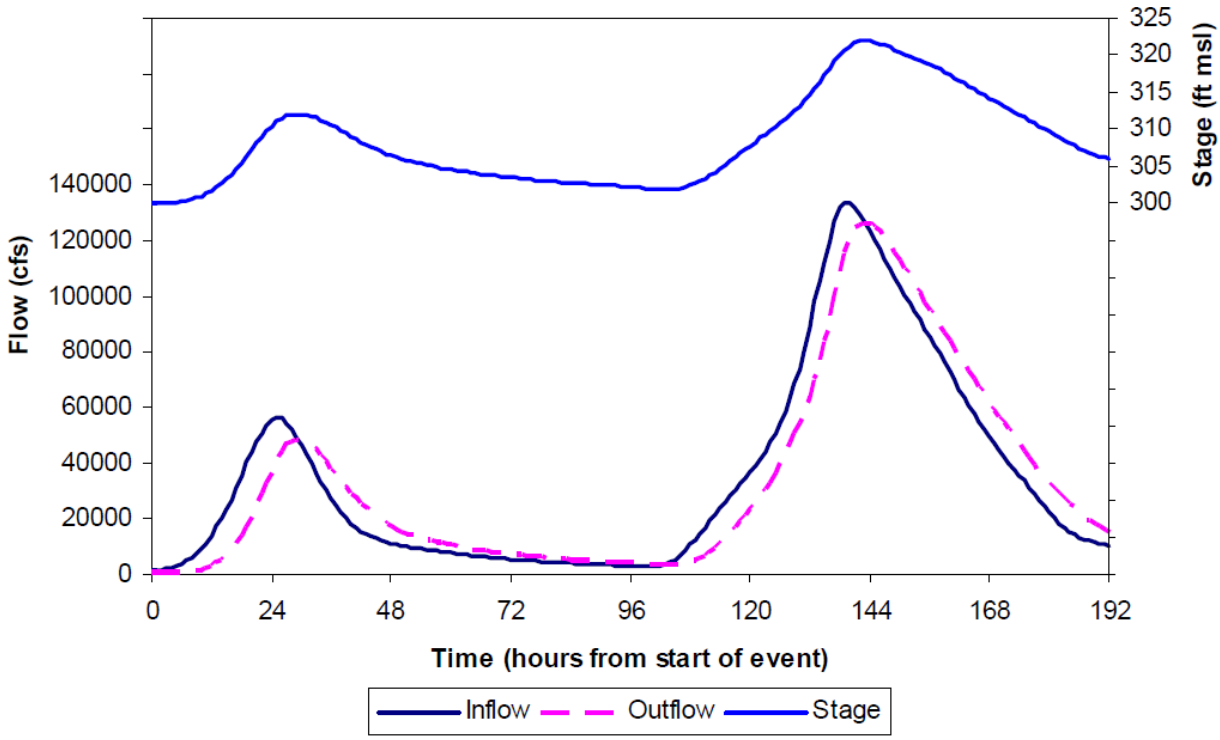


Figure 7.5 Outflow hydrograph from reservoir during PMF event
Source: NHC PMF Report (2006)

Summary of land area and residential units affected

County	Total Area (sq. miles)	Land area affected (sq. miles)	Total Housing units in County	Housing units affected by flood
Yuba	631.84	60.3	27527	2627
Placer	1407.01	12.9	154,365	1415
Sutter	602.41	123.7	33,886	6958
Sacramento	964.64	8.2	556,332	0
Total Area	**3605.9**	**205**	**772,110**	**11,001**
Agricultural area (acres)		**98,448**		

Notes:

1. *Since the area affected by Sacramento County is rural in nature, no housing units were considered in the county*
2. *75% of land area was assumed to be under cultivated agriculture*

Figure 7.6 Determination of affected residences and agriculture

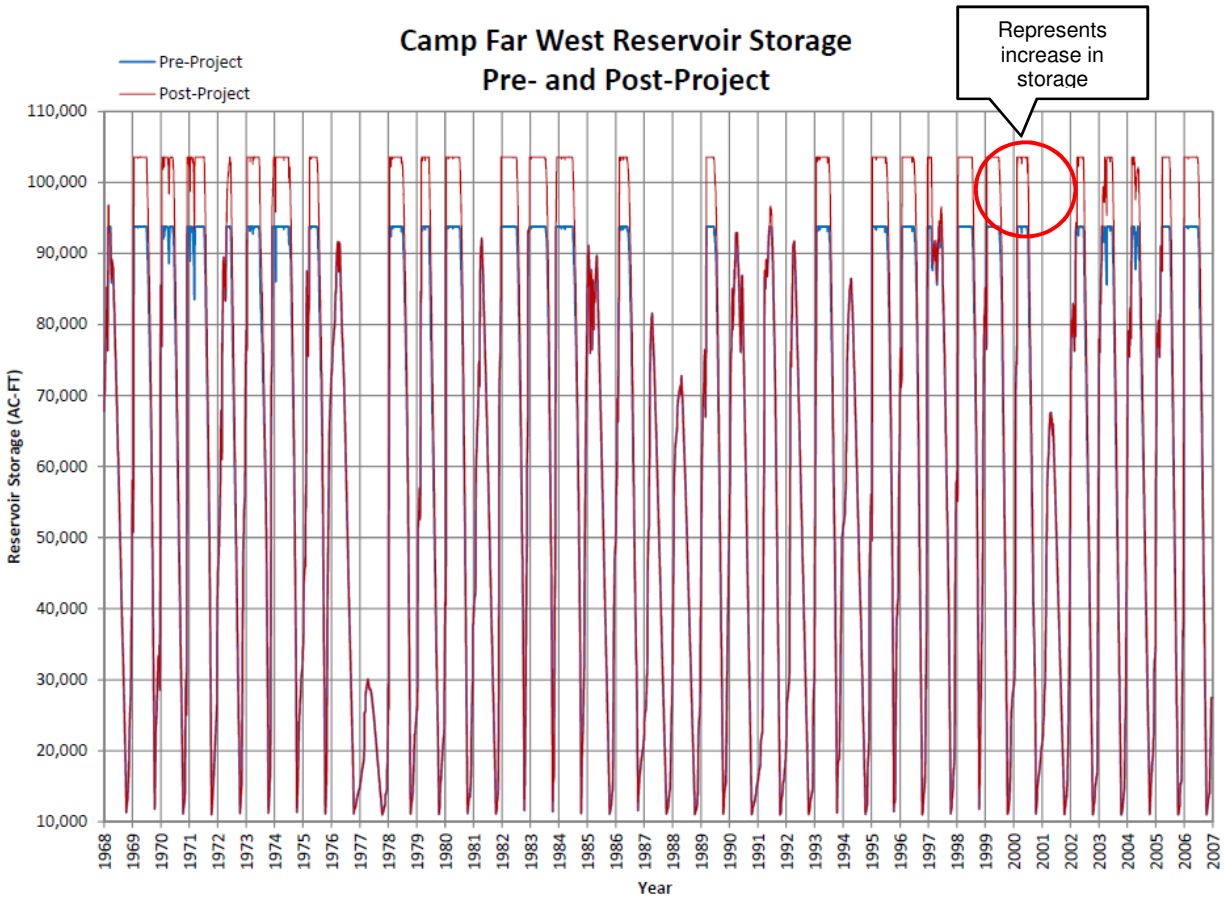


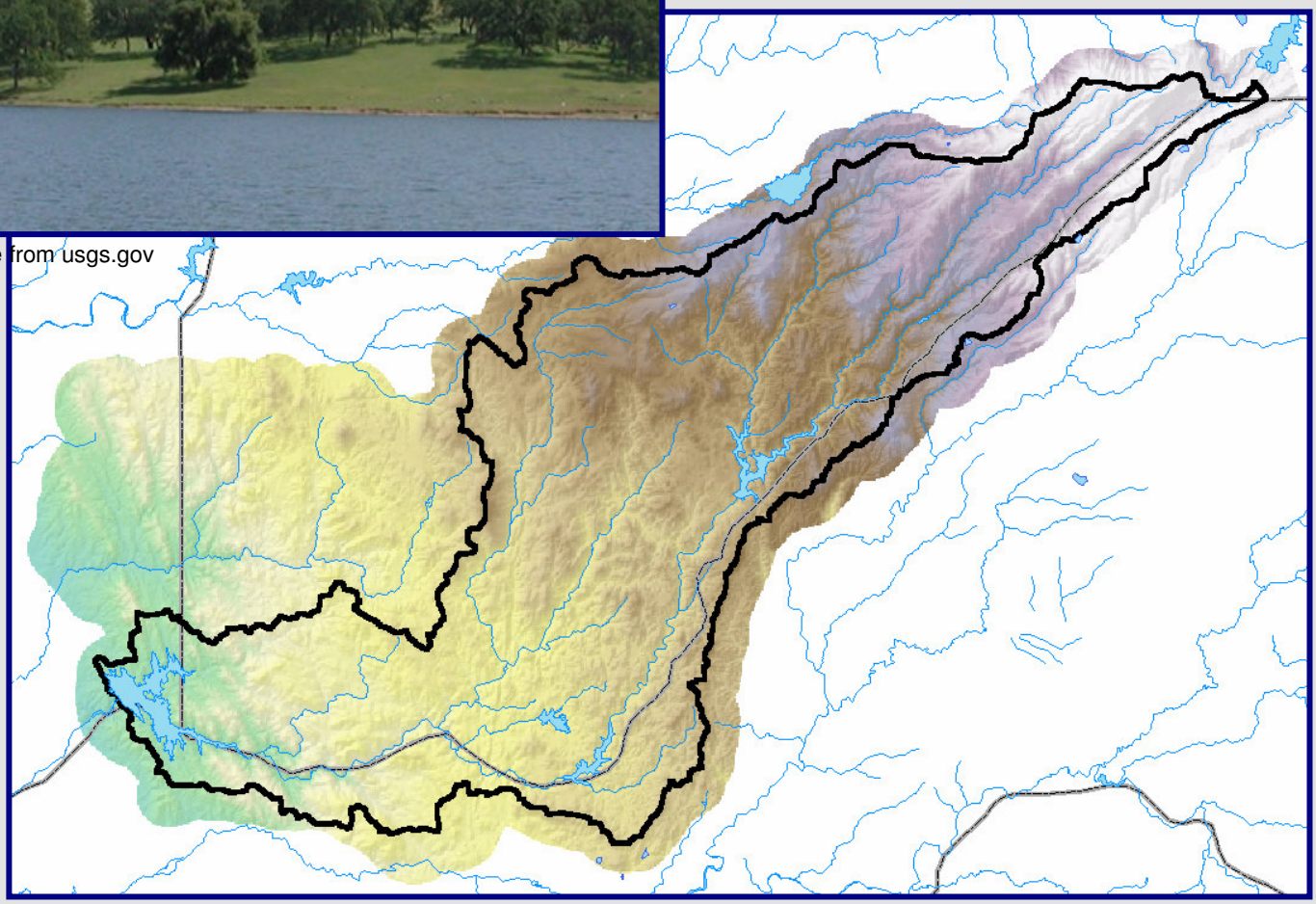
Figure 7.7 Determination of annual water storage benefits

Probable Maximum Flood Study for Camp Far West Dam (FERC No. 2997)

Prepared for:

South Sutter Water District

Image from usgs.gov



Prepared by:

nhc northwest hydraulic consultants

Leaders in water resource technology

March 2006

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I. Introduction

This report describes the methodologies, data development, and results of a Probable Maximum Flood (PMF) study for Camp Far West Dam (FERC No. 2997) on the Bear River in northern California. Northwest Hydraulic Consultants, Inc. (**nhc**) was retained by the South Sutter Water District to develop this revised PMP and PMF in response to a request by the Federal Energy Regulatory Commission (FERC). In general, the procedures followed in the development of this PMF were based on FERC's "Engineering Guidelines for the Evaluation of Hydropower Projects" (FERC, 2001).

II. Project Description

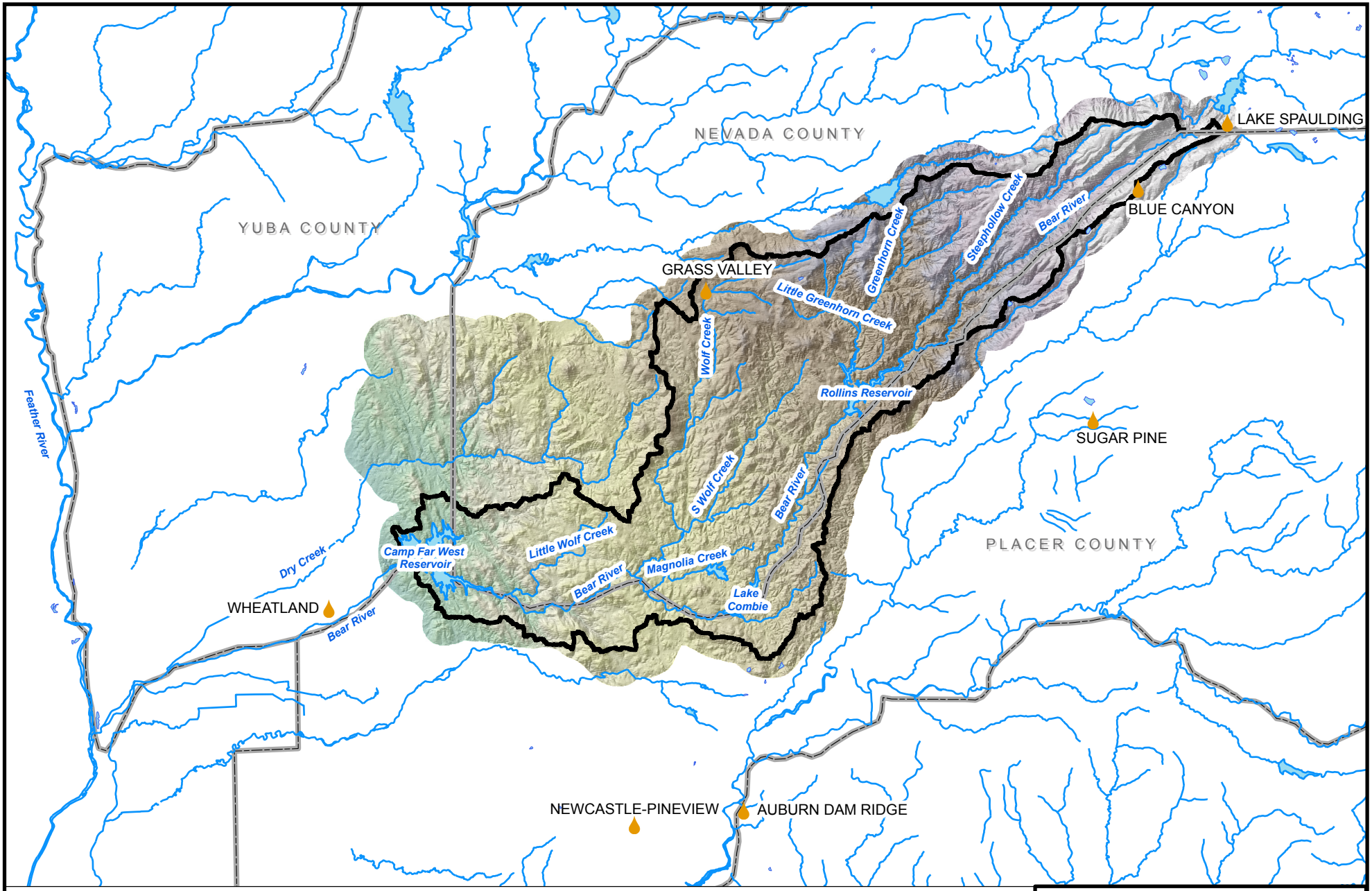
A. Project Information

Camp Far West is operated by South Sutter Water District (SSWD) for power generation, irrigation, and recreation. The facility is located on the Bear River, approximately 13 miles east and upstream of the confluence with the Feather River and 15 miles southeast of Marysville, California. The drainage area to Camp Far West dam is approximately 282 square miles. The largest tributary to the Bear River in the Camp Far West basin is Wolf Creek, which drains 78 square miles and joins the Bear River approximately 13 miles upstream of Camp Far West Dam. Figure 1 illustrates the Camp Far West drainage basin.

Camp Far West Reservoir was planned in the early 1950s to supplement the growing demand for irrigation water and to help reverse groundwater depletion. Construction began in June 1962 and was completed in January 1964. Camp Far West Dam is an earthfill structure with a height of 185 feet, measured from the original streambed to the dam crest at elevation 320 feet. The main dam crest is approximately 2,070 feet long. The total crest length, including the wing walls and northern dike, is approximately 6,550 feet. The spillway is an ungated, ogee-type reinforced concrete structure. The original spillway design capacity was based on Cooperative Report No. 12 to determine the Probable Maximum Flood (PMF). The design PMF inflow hydrograph had a peak of 78,400 cfs. The final spillway design provided a crest at elevation 300 and length of 300 feet, which passed 75,200 cfs at elevation 316.3 (3.7 feet of freeboard).

The storage capacity and spillway discharge rating curve for Camp Far West are summarized in Table 1. Storage data were obtained from a model developed by the Corps of Engineers for a 2004 study (USACE, 2004) and are based on data obtained from SSWD. The spillway rating curve was taken from as-built drawings for Camp Far West Dam. The Corps study used a different spillway rating curve dated January 1968, but that curve extended only to elevation 316 and appears to have been extended incorrectly above elevation 315. Since the discharges at lower water surface elevations are similar between the two curves, **nhc**, in consultation with the SSWD, elected to use the as-built curve, which provided more reasonable estimates of the discharges at higher elevations. The curve was graphically extrapolated to estimate discharges above 320 feet msl.

Camp Far West is not a flood control project. It is operated to fill and spill early in the rainy season. Flood control is provided only early in the season, while the reservoir is filling, and by surcharge storage under spillway flow conditions.



Precipitation Gages

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Camp Far West PMF	
Camp Far West Basin Map	
northwest hydraulic consultants	
Date: 11/28/05	Figure 1

Table 1. Camp Far West Storage Capacity and Spillway Rating

Reservoir Elevation (ft–msl)	Storage Capacity (acre-ft)	Spillway Discharge (cfs)
140	0	0
280	69,500	0
300	104,400	0
302	108,700	2,900
304	113,100	8,000
306	117,400	14,500
308	121,800	23,000
312	131,100	47,000
316	141,000	73,000
320	151,000	106,500
322	156,000	126,500

B. Basin Physiography and Geology

The Bear River basin is located on the western slope of the Sierra Nevada and is bounded on the north by the Yuba River basin and on the south by the American River basin. The Bear River drains portions of Placer, Nevada, and Yuba counties. The river flows in a southwesterly direction for about 65 miles from its headwaters near Lake Spaulding to its confluence with the Feather River about 13 miles downstream of Camp Far West Dam (see Figure 1).

Basin topography transitions from foothills in the western portion of the basin to steep mountainous terrain in the east. Elevations in the basin upstream of Camp Far West range from 300 feet near the dam to just over 5,800 feet. Slopes range from 2 to more than 30 percent. The basin is underlain predominantly by Triassic-Jurassic metavolcanic rocks. There is a wide variety of soils in the basin, but drainage characteristics are less variable. Most of the basin soils fall into NRCS hydrologic group C, indicating relatively poor drainage, though there are pockets of more well-drained soils. Most of the basin is forested, with cover ranging from oak woodland in the foothills to coniferous forest at higher elevations. Portions of the western part of the basin are covered by grasslands or scrub/rangeland, and there is some urban development, particularly the Nevada City area in the upper Wolf Creek basin.

The climate of the basin is characterized by hot, dry summers and mild, wet winters, with over 80 percent of annual precipitation falling between November and March. Mean annual precipitation for the Camp Far West basin is approximately 48 inches, ranging from 23 inches at lower elevations to over 70 inches in the basin headwaters. Most of the basin's precipitation falls as rain, though winter snow is common at higher elevations. Significant snow accumulation below 5,000 feet is rare, with any snowfall typically melting within a short time.

C. Upstream Dams

Flows into Camp Far West are affected by several upstream reservoirs and diversions. Rollins Reservoir, which captures inflow from the upper 101 square miles of the watershed, is the only other significant reservoir in the basin. It has a storage capacity of 66,000 acre-feet and a 300-foot long ungated spillway. The Rollins spillway crest is at elevation 2,171 feet. Rollins Dam is operated by the Nevada Irrigation District (NID). Storage and discharge data for Rollins Reservoir are summarized in Table 2 (USACE, 2004). Lake Combie, approximately 15 miles upstream of Camp Far West, has only 5,000 acre-feet of storage capacity and does not provide storage or flow attenuation during large events. Magnolia Dam, near Lake Combie, and Dutch Flat Afterbay, at the upper end of the basin, provide no significant storage or flow attenuation.

There are several inter-basin transfers between the Bear River and the Yuba and American Rivers. In the upper basin, the Drum Canal transfers water from Lake Spaulding in the Yuba River basin to the Drum Forebay; water used for power production then travels via penstocks to the Drum Afterbay upstream of Dutch Flat in the Bear River basin. Smaller flows are diverted from the South Yuba Canal to the Bear River headwaters and from the Low Boardman Canal to the Bear River below Dutch Flat Afterbay. Downstream of Rollins Reservoir, the Bear River Canal diverts water to the American River basin. At peak operation, the Drum Canal typically carries about 840 cfs, most of which ends up in the Bear River, while the Bear River Canal diverts about 440 cfs out of the basin. However, during the peak of the January 1997 event, flow in Drum Canal was about 100 cfs and flow in Bear River Canal was about 240 cfs. Because flow records do not indicate consistent net gain or loss to the Bear River during large events, and because diversion flows are sufficiently small compared to large event flows to be insignificant, these transfers were neglected for this study.

Like Camp Far West, Rollins Reservoir and Lake Combie are operated to fill as early as possible in the rainy season. Consequently, they would not be expected to provide significant attenuation of the PMF.

Table 2. Rollins Storage Capacity and Discharge Rating

Reservoir Elevation (ft–msl)	Storage Capacity (acre-ft)	Discharge (cfs)
1,970	2,600	0
2,005	6,200	155
2,105	25,900	600
2,165	61,200	870
2,171	66,000	900
2,173	67,700	3,400
2,175	69,400	10,000
2,181	74,600	38,000
2,187	80,200	81,000

D. Previous Studies

An earlier PMF study for the basin was produced by the Corps of Engineers in 1971, using Hydrometeorological Report No. 36 (HMR 36) to determine the probable maximum precipitation (PMP) amounts. That study, which used an energy budget method to determine snowmelt contributions, estimated the PMF flow at the Garden Bar Dam Site, 4.5 miles upstream of Camp Far West as 136,000 cfs. In 1980, the California Division of Safety of Dams (DSOD) performed a PMF study for Camp Far West Dam and determined a PMF inflow of approximately 112,700 cfs. The 1980 study, which also used PMP amounts calculated according to HMR 36, did not consider snowmelt. The 1971 and 1980 studies were reviewed in a 1987 Independent Consultant's Safety Inspection Report (Parsons Brinckerhoff, 1987), which judged the 1980 PMF analysis to be reasonable, despite the much lower peak. DSOD produced a revised PMF in 1996, also using HMR 36 to determine PMP and snowmelt temperatures. The PMF determined in this study was approximately 86,900 cfs (Tapia, 1996). Only limited documentation of the 1996 analysis is available.

In addition to the PMF studies described above, the Corps of Engineers performed an analysis of Bear River Hydrology in a 2004 floodplain mapping study (USACE, 2004). The Corps developed an HEC-HMS model that was calibrated to the New Year 1997 flood event, the second largest on record since Camp Far West was completed.

The large discrepancy in the previous PMF studies suggests that a new PMF analysis is called for. This report documents an independent PMF analysis performed by **nhc** using updated PMP guidance from HMR 59 (NOAA, 1999), best available geospatial data, and the most appropriate hydrologic modeling methodologies.

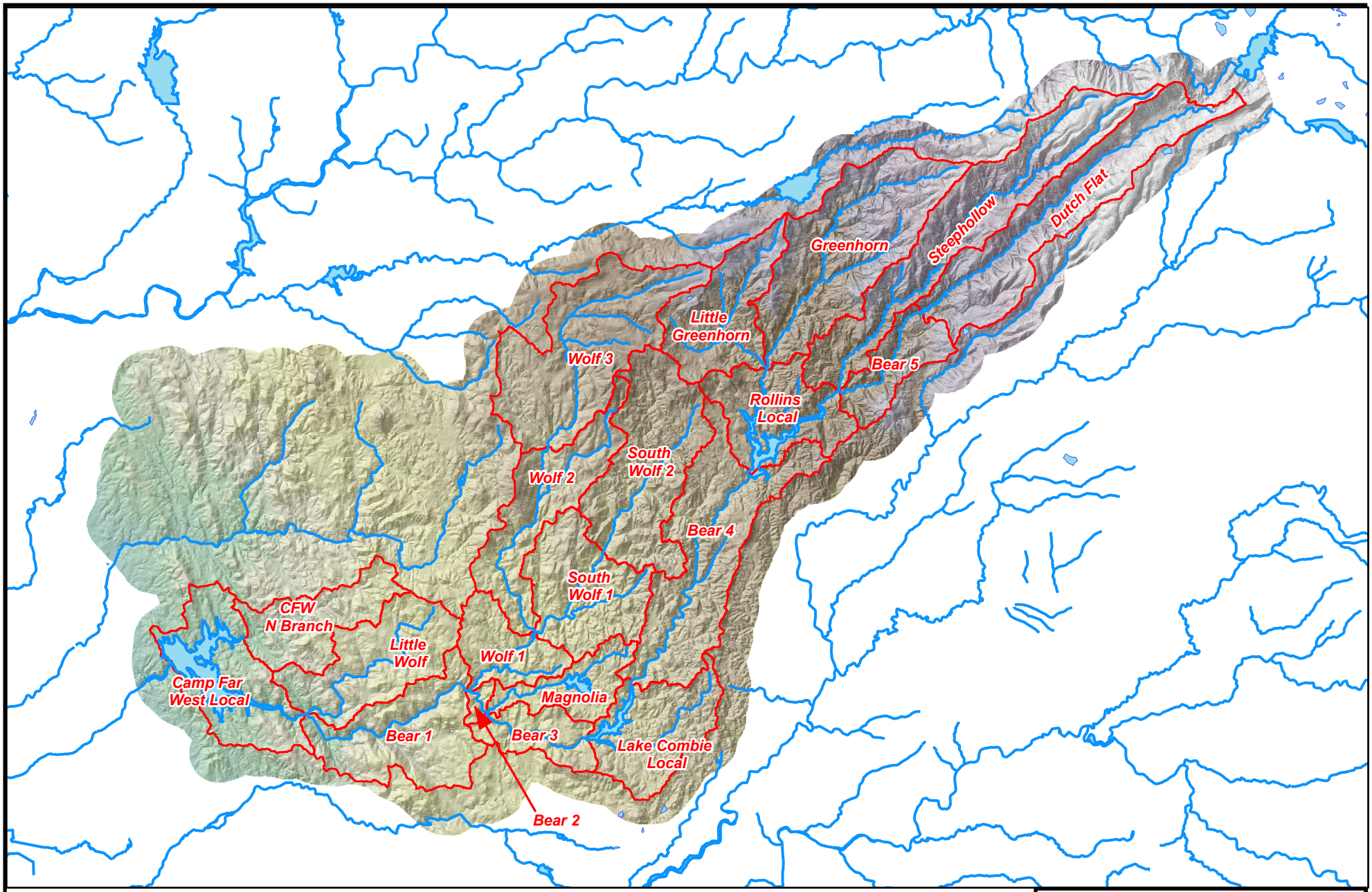
III. Watershed Model

A. Model Development

The Corps of Engineers HEC-HMS hydrologic modeling software was used to model the Camp Far West PMF. HEC-HMS, which replaces HEC-1, is a widely accepted hydrologic model that has been applied to numerous PMF studies. The newest version of HEC-HMS (version 3.0) offers algorithms for modeling snowmelt runoff in addition to rainfall-runoff modeling and basic reach and reservoir routing capabilities that have been available in previous releases. Application of the HEC-HMS model requires the development of geographic and meteorologic input data, and the model is typically calibrated using observed flow data. The model development is described below; model calibration is described in Section V.

B. Subbasin Definition

For the purpose of hydrologic modeling, the Camp Far West watershed was subdivided into 20 subbasins, which were delineated in GIS from the National Elevation Dataset (NED). Subbasin boundaries are shown in Figure 2. Although calibration data are only available for inflows to Rollins and Camp Far West reservoirs (reconstructed from recorded outflow and reservoir elevation data), the basin was subdivided to better represent the spatial variability in meteorologic and geographic characteristics and to provide a better means for evaluating hydrograph timing issues. Hypsometric (area-elevation) data, used in defining elevation bands for simulation of snow accumulation and melt, were also determined from the NED (Table 3).



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Camp Far West PMF

HEC-HMS Model Subbasins

northwest hydraulic consultants

Date: 11/28/05 | Figure 2

The HEC-HMS model for the Camp Far West basin uses the deficit and constant loss rate methodology and the Clark unit hydrograph approach for runoff modeling. Preliminary estimates of loss (infiltration) rates in the model were based on soil characteristics and the loss rates used in the Corps model (USACE, 2004). Loss rates were assumed to vary by subbasin with basin slope and soil drainage characteristics, represented by the surface runoff production attribute from NRCS SSURGO data. Monthly average evapotranspiration data, used in the deficit and constant loss method to recover soil storage capacity during dry periods, were obtained from the California Irrigation Management Information System (CIMIS) for the nearby Browns Valley evaporation station.

The T_c parameter for the Clark unit hydrograph was initially estimated for each subbasin as a function of flow path length and elevation change, using an empirical equation developed by the U.S. Bureau of Reclamation. The R parameter was determined by calibration, assuming a constant value for the ratio $R/(T_c+R)$ over the basin. Initial estimates for the loss rates and unit hydrograph parameters were updated by calibration to the New Year 1997 flood event (see Section V).

Table 3. Camp Far West Area-Elevation Summary

Subbasin	Percent of Basin Area (%) by Elevation Band (ft)					
	0-1000	1000-2000	2000-3000	3000-4000	4000-5000	5000-6000
Bear 1	36.5%	63.5%	0.0%	0.0%	0.0%	0.0%
Bear 2	2.4%	97.6%	0.0%	0.0%	0.0%	0.0%
Bear 3	0.0%	98.3%	1.7%	0.0%	0.0%	0.0%
Bear 4	0.0%	25.1%	74.2%	0.7%	0.0%	0.0%
Bear 5	0.0%	0.0%	50.5%	49.5%	0.0%	0.0%
Camp Far West Local	100.0%	0.0%	0.0%	0.0%	0.0%	0.0%
CFW North Branch	55.7%	44.3%	0.0%	0.0%	0.0%	0.0%
Dutch Flat	0.0%	0.0%	1.9%	35.4%	50.8%	11.9%
Greenhorn	0.0%	0.0%	33.7%	49.4%	16.8%	0.0%
Lake Combie Local	0.0%	75.3%	24.7%	0.0%	0.0%	0.0%
Little Greenhorn	0.0%	0.0%	66.4%	33.6%	0.0%	0.0%
Little Wolf	28.6%	71.4%	0.0%	0.0%	0.0%	0.0%
Magnolia	0.0%	99.5%	0.5%	0.0%	0.0%	0.0%
Rollins Local	0.0%	0.0%	99.2%	0.8%	0.0%	0.0%
South Wolf 1	0.0%	84.5%	15.5%	0.0%	0.0%	0.0%
South Wolf 2	0.0%	24.9%	75.0%	0.0%	0.0%	0.0%
Steephollow	0.0%	0.0%	20.1%	35.7%	31.8%	12.4%
Wolf 1	0.2%	99.8%	0.0%	0.0%	0.0%	0.0%
Wolf 2	0.0%	55.8%	44.0%	0.2%	0.0%	0.0%
Wolf 3	0.0%	2.3%	89.7%	7.9%	0.0%	0.0%
Whole Basin	10.8%	30.0%	35.8%	13.4%	8.0%	2.0%

C. Channel Routing Method

Muskingum routing was used for river reaches, with Muskingum 'k' values determined from calculations of travel time using Manning's equation. Reach lengths and slopes were computed from GIS data, and separate channel characteristics were estimated for headwaters/tributary channels and main stem channels. The latter were refined during calibration. Travel time through the

reservoirs (Rollins Reservoir, Lake Combie, and Camp Far West) was represented and applied in the model using the lag routing method. Outflows from Rollins and Camp Far West were then determined from storage-discharge curves.

IV. Historic Flood Records

A. Stream Gages

Only limited streamflow data are available for the Camp Far West watershed. The USGS publishes data for stream gages on the Bear River below Rollins Dam (station 11422500) and on the Bear River at Wheatland (station 11424000) downstream from Camp Far West Dam. A number of the canals and diversions in the basin are also gaged, but those data were not applicable to this study.

The Nevada Irrigation District (NID) maintains a reservoir stage record for Rollins Reservoir (originally established by the USGS and reported as station 11421800), which was used to reconstruct inflows for modeled historical events for which data were available. Limited operations log records for Camp Far West Dam were obtained for the 1997 and 1986 events. Flows from the Wheatland gage and the Camp Far West stage-storage-discharge relationship were used to reconstruct inflows to Camp Far West for the 1995 event.

B. Historic Floods

The largest historical floods on the Bear River since the construction of Camp Far West Dam occurred in February 1986 and January 1997. Peak (regulated) flows for these events below Rollins and Camp Far West, as reported by the USGS, are summarized in Table 4. The relative magnitude of the peaks at the two locations can vary significantly between events depending on precipitation distribution and snow accumulation and melt at higher elevations.

Table 4. Historic Flood Event Peaks

Event	Bear River below Rollins (cfs)	Bear River at Wheatland (cfs)
Feb 1986	22,500	48,000
Mar 1995	n/a	17,500
Jan 1997	34,300[a]	34,900
Feb 1998	5,310	22,000

[a]Data from this gage appear anomalous for the Jan 1997 event; gage broke during event and flows are reported as estimated for Jan 3 – Feb 12 (after peak).

Considering the quality and availability of discharge and hydrometeorological data, the historic flood events selected for model calibration and verification, respectively, were:

- January 1997
- February 1986
- March 1995

Each of these events occurred during the season when the critical PMF would occur (November through March) and each event is typical of winter rain-on-snow events associated with large-scale

frontal systems in this region. The January 1997 event was selected as the primary calibration event because a) it is the largest storm for which substantial data are available, b) there are sufficient data to reconstruct sub-daily inflows to both major reservoirs, and c) spatial mapping of precipitation (produced by the Oregon Climate Service) was available for the event. Reliable short-interval data were not available for Rollins for the 1986 or 1995 events, nor could information on precipitation distribution within the Camp Far West basin be obtained.

C. Precipitation Associated with Historic Floods

Hydrologic modeling of the calibration events necessitated the development of precipitation data in the form of event total precipitation and the temporal distribution of these totals. Subbasin event total precipitations for the January 1997 event (December 29 – January 3) were estimated from GIS mapping of precipitation developed by the Oregon Climate Service (OCS). The temporal distribution of precipitation was determined using available gage records. **nhc** examined precipitation data for several recording gages in or near the Bear River basin and assigned temporal gages to individual subbasins based on proximity to the gage and total event precipitation. The precipitation gage sites used in the simulation were Newcastle, Auburn Dam Ridge, Sugar Pine, Blue Canyon, and Lake Spaulding (see Figure 1). The Corps model (USACE, 2004) used the same precipitation gages for simulation of the 1997 event.

Precipitation data for the February 1986 and March 1995 events were determined from hourly gage records at Wheatland, Grass Valley, and Blue Canyon (see Figure 1). Spatial distribution of precipitation for the 1986 event was assumed to be similar to the 1997 event, based on ratios of gage totals. An American River PMF study produced by the Corps (USACE, 2001) reports that isohyetal mapping of this event was performed by the National Weather Service, but **nhc** was unable to obtain this mapping. The spatial distribution for the 1995 event was assumed to be similar to mean annual precipitation, as all three hourly gages recorded approximately 20 percent of their mean annual rainfall according to OCS mapping.

D. Snowpack/Snowmelt Associated with Historic Floods

Snow water equivalent (SWE) data for the 1997 and 1995 events were available for two SNOTEL sites in the vicinity and elevation range of the basin, Blue Canyon (5,280 feet) and Greek Store (5,600 feet). Snow depth data from 11 lower elevation stations were used to provide additional estimates. Initial SWE for each 1000-foot elevation band was estimated from these data and past relationships for the snow gage sites used to develop the initial SWE for the PMF (see Section VIII). For the 1997 event, additional information was obtained from the Corps' simulation of American River snowmelt (USACE, 2001). For the 1986 event, SWE data were available for eight stations at elevations ranging from 5,200 feet to 6,900 feet, and snow depth data were obtained from 11 lower elevation stations (see Table 9). A regression line through these points was used to determine initial SWE for the model elevation bands. The initial snowpack, by elevation band, used in the simulations of the three historic events are shown in Table 5.

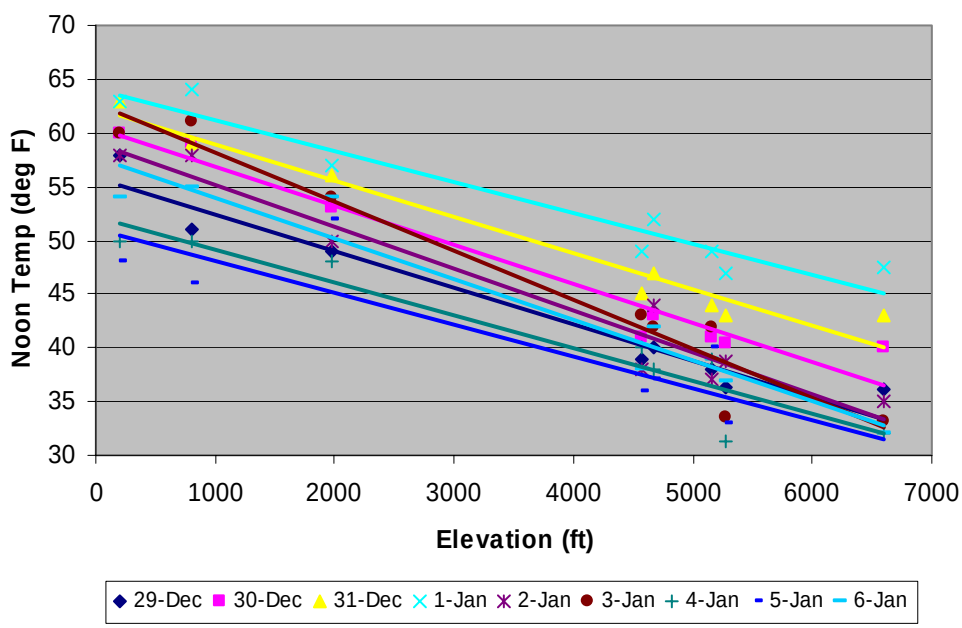
Table 5. Initial Snow Water Equivalent for Historic Events

Elevation Band (feet)	Snow Water Equivalent (inches) February 1986[a] (12 Feb)	March 1995 (8 March 1995)	January 1997 (29 Dec 1996)
0-1000	0	0	0
1000-2000	0	0	0
2000-3000	0	0	0
3000-4000	0	0	0.8
4000-5000	0	2	3.3
5000-6000	8.9	15	9.8

[a]Snowmelt was not simulated for this event due to lack of available hourly temperature data and the fact that less than 2% of the basin lies in the highest elevation band.

Hourly temperature data were obtained from CDEC for eight stations for the 1997 event and for five stations for the 1995 event. The maximum and minimum daily temperatures and noon temperatures were plotted versus gage elevations to estimate an average lapse rate of 3.5 ºF per 1,000 feet of elevation change. Figure 3 shows the noon temperatures by elevation for each day of the 1997 event. Only low elevation data were available for the 1995 event, but those data showed a lapse rate similar to the curves shown in Figure 3. Due to their proximity to the basin, the Lincoln (200 feet) and Lake Spaulding (5,156 feet) stations were selected as representative low and high elevation temperature series, respectively, and temperature patterns in each subbasin were assumed to follow one of those two stations. The model uses the specified gage elevations and elevation band data for each subbasin to lapse the temperature series appropriately. Subbasin average values for initial SWE and temperature are then determined based on the hypsometric relationships for the individual subbasins (see Table 3).

Figure 3. January 1997 Noon Temperatures



There were no hourly temperature data available for the 1986 event. Because SWE data indicated that snow would only be present in the uppermost elevation band, which contains less than two percent of the basin area, it was judged to be reasonable to simulate the 1986 event with no snowmelt component.

V. Model Calibration

A. Approach

The HEC-HMS model was calibrated to the January 1997 storm event, one of the largest events on the Bear River since the construction of Camp Far West Dam. Hydrometeorological input data (precipitation, temperature, and initial snow water equivalent) for the calibration event were developed from gage data as described in Section IV.

For the 1997 event, the model was disconnected below Rollins Reservoir and a source added to represent Rollins reservoir outflows to the lower basin. This eliminates propagation of modeling errors in the upper basin and facilitates calibration in the lower basin. Hourly outflows from Rollins Dam were computed from hourly reservoir stages using the modeled stage-discharge rating, as there appear to be problems with the gage record downstream of Rollins.

The HEC-HMS model was calibrated to the Rollins and Camp Far West inflow hydrographs by adjusting the Clark unit hydrograph (T_c and R) and loss rate parameters. Rather than adjusting parameters for each subbasin independently, which could not be justified by the limited data available, consistent adjustments were made based on subbasin characteristics. Loss rates were adjusted based on surface runoff potential, an attribute of the NRCS SSURGO data. Subbasin loss rates were then calculated as a weighted average of the assigned rates for the soil types in the subbasin. Loss rates for the steep upper basin subbasins were reduced by 60 percent, as the upper basin has generally thinner soils and numerous rock outcrops. Reach lag times and Muskingum parameters were also modified during calibration. Adjustments to reach parameters were applied consistently by the type of reach, differentiating between upper (headwater/tributary) and lower (main stem) reaches.

B. Baseflow

Baseflow parameters (initial flow per unit area, baseflow threshold as percent of peak, and exponential recession) were initially adopted from the 2004 Corps model and adjusted during calibration. Baseflow parameters varied by subbasin depending on the relative basin slope (steep versus moderately sloped).

C. Unit Hydrograph Development

The T_c parameter for the Clark unit hydrograph was initially estimated for each subbasin as a function of flow path length and elevation change, using an empirical equation developed by the U.S. Bureau of Reclamation. The R parameter was calculated to maintain a consistent value of $R/(T_c+R)$ for all subbasins. A ratio value of 0.7 was found to provide a good match for hydrograph shape and is consistent with values **nhc** has determined for other studies in northern California. This correspondence is not unexpected, as the value of this ratio tends to be constant in regions with similar physiographic and hydrologic characteristics.

D. Loss Rates

Loss rates were based on soil characteristics published in the NRCS SSURGO data. By assigning loss rates to each category of the surface runoff potential attribute, **nhc** calculated a loss rate for each mapped soil unit. Subbasin loss rates were determined by calculating weighted averages of the individual soil unit loss rates. The Camp Far West basin is dominated by soils with high surface runoff production potential, with areas of exposed bedrock particularly in the upper basin. The National Land Cover Dataset (NLCD) was used to determine forested areas, and different soil storage rates were defined for forest versus non-forest (pasture or rangeland) areas. Soil storage is essentially an initial condition in an event model, and the amount of available storage, which was adjusted during calibration, does not significantly impact peaks. Impervious area for each subbasin was also determined from analysis of the NLCD.

During calibration, it was found to be necessary to apply lower loss rates in the steep upper basins to match flow peaks and volumes into Rollins Reservoir. Although soils mapping does not indicate significant differences in soil groups or runoff potentials, the steep upper basins have typically shallower soils and exposed bedrock, which would be expected to have much lower soil storage and infiltration capacity.

The watershed is extensively forested, and the extent and effects of frozen soil are unknown, though assumed to be negligible since only a small fraction of the basin experiences regular snow cover and freezing conditions.

E. Snowmelt Parameters

The snowmelt algorithm in HEC-HMS is a temperature index method based on that developed for the SSARR model. It is significantly more complex than the degree-day method previously available in HEC-1, employing a variable melt rate depending on precipitation and antecedent temperatures and capturing some of the complexities of snowpack dynamics. The general parameters for the snowmelt model were taken from a study of the Sacramento and San Joaquin River basins that optimized parameter results for the entire western Sierra (Daly et al, 2000). Because snowpack was fairly insignificant in the calibration events, the validity of these parameters for the specific basin could not be assessed. Ground melt was also neglected for these events at the recommendation of the model's creator (Daly, personal communication).

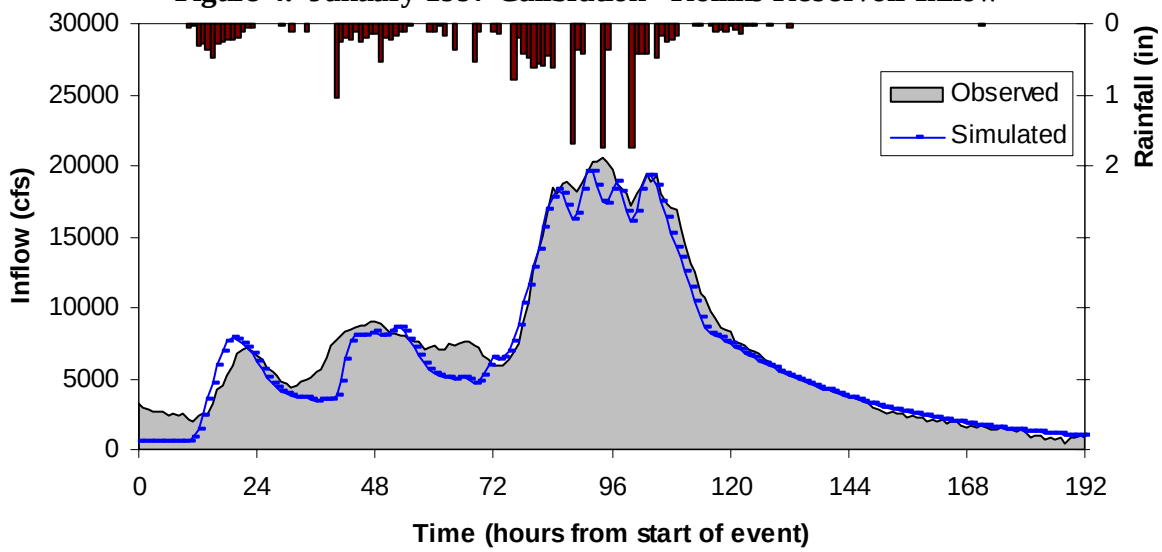
F. Calibration Results

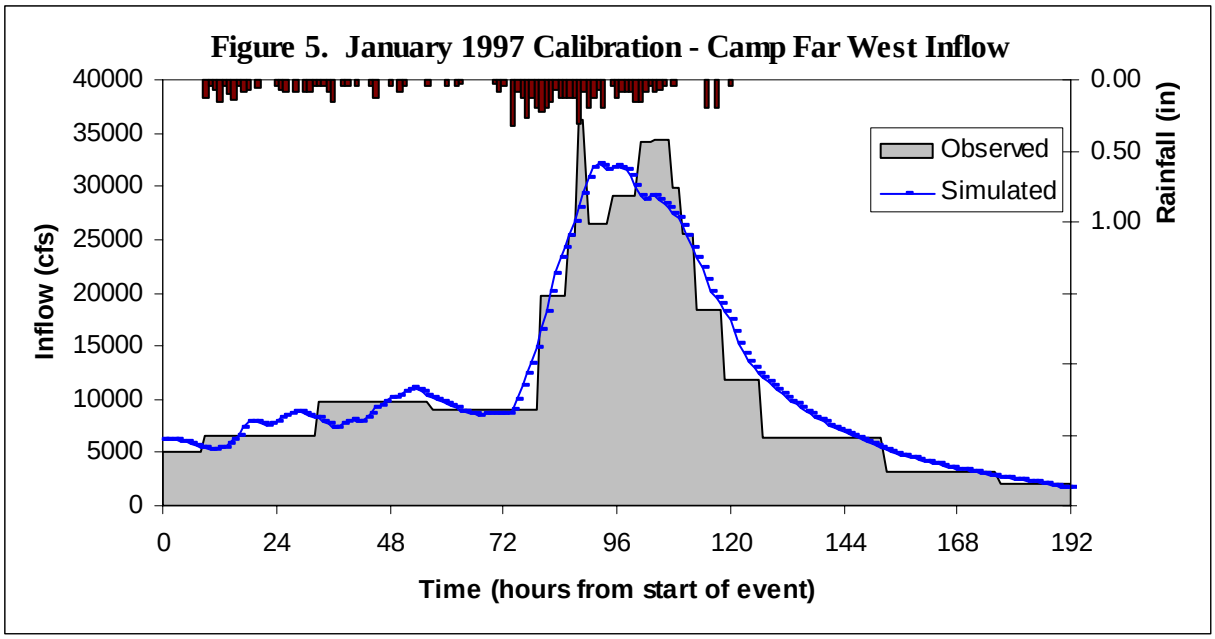
Table 6 shows the final calibrated values for key model parameters. The calibrated hydrographs for Rollins and Camp Far West are shown compared to the reconstructed inflow hydrographs in Figures 4 and 5, respectively. For the 1997 event, simulated peak flows are lower than observed entering both Rollins and Camp Far West (-3.7 and -11 percent, respectively). Event volumes are simulated well, with the model slightly undersimulating inflow to Rollins (-7.8 percent) and oversimulating inflow to Camp Far West (6.5 percent).

Table 6. Calibrated HEC-HMS Parameters

Subbasin	Area (sq mi)	Clark Unit Hydrograph		Rainfall Abstractions	
		T_c (hr)	R (hr)	Soil Storage (in)	Constant Loss (in/hr)
Bear 1	15.4	2.38	5.56	0.97	0.13
Bear 2	0.9	0.63	1.47	1.00	0.16
Bear 3	5.9	1.64	3.83	0.98	0.14
Bear 4	22.7	3.76	8.77	0.95	0.12
Bear 5	7.0	1.42	3.31	0.28	0.08
Camp Far West Local	15.1	2.15	5.01	0.89	0.13
CFW North Branch	10.1	2.01	4.70	0.95	0.09
Dutch Flat	21.4	3.34	7.80	0.29	0.05
Greenhorn	24.3	2.94	6.86	0.30	0.04
Lake Combie Local	12.0	2.22	5.17	0.95	0.12
Little Greenhorn	12.5	1.87	4.37	0.98	0.04
Little Wolf	14.4	2.94	6.86	0.94	0.10
Magnolia	6.6	2.08	4.86	0.88	0.10
Rollins Local	12.0	1.69	3.95	0.28	0.04
South Wolf 1	13.9	1.64	3.84	0.98	0.10
South Wolf 2	19.5	2.28	5.31	0.94	0.10
Steephollow	23.7	4.01	9.36	0.29	0.04
Wolf 1	7.2	1.67	3.90	0.96	0.11
Wolf 2	14.0	3.50	8.17	0.86	0.10
Wolf 3	23.5	3.03	7.07	0.85	0.12

Figure 4. January 1997 Calibration - Rollins Reservoir Inflow



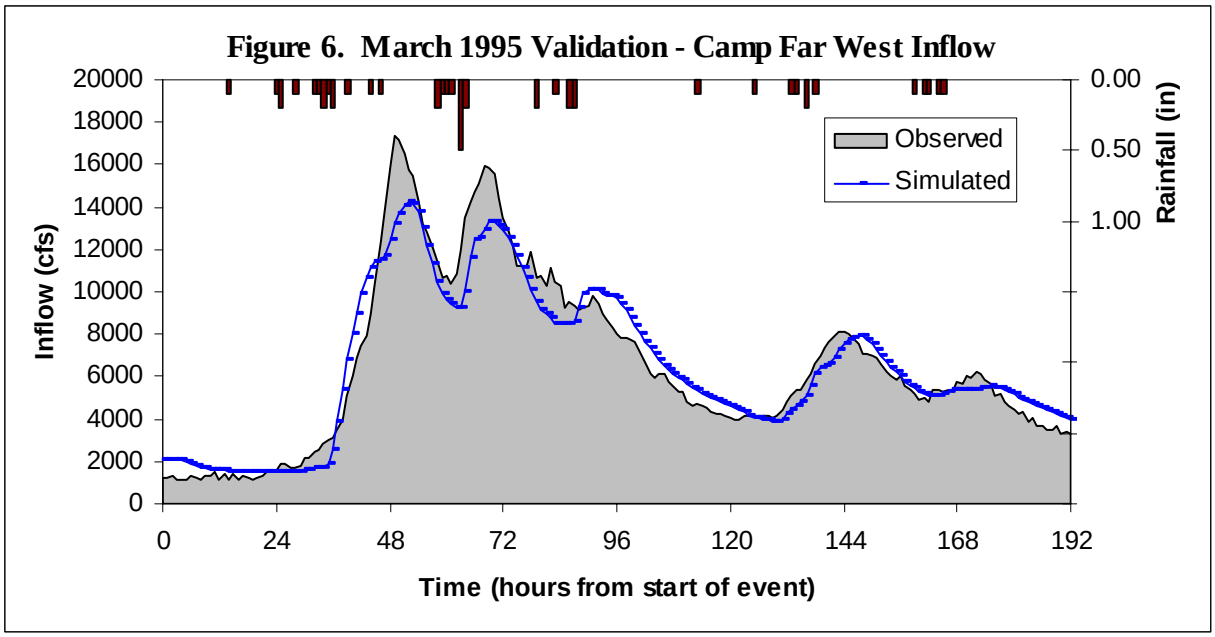


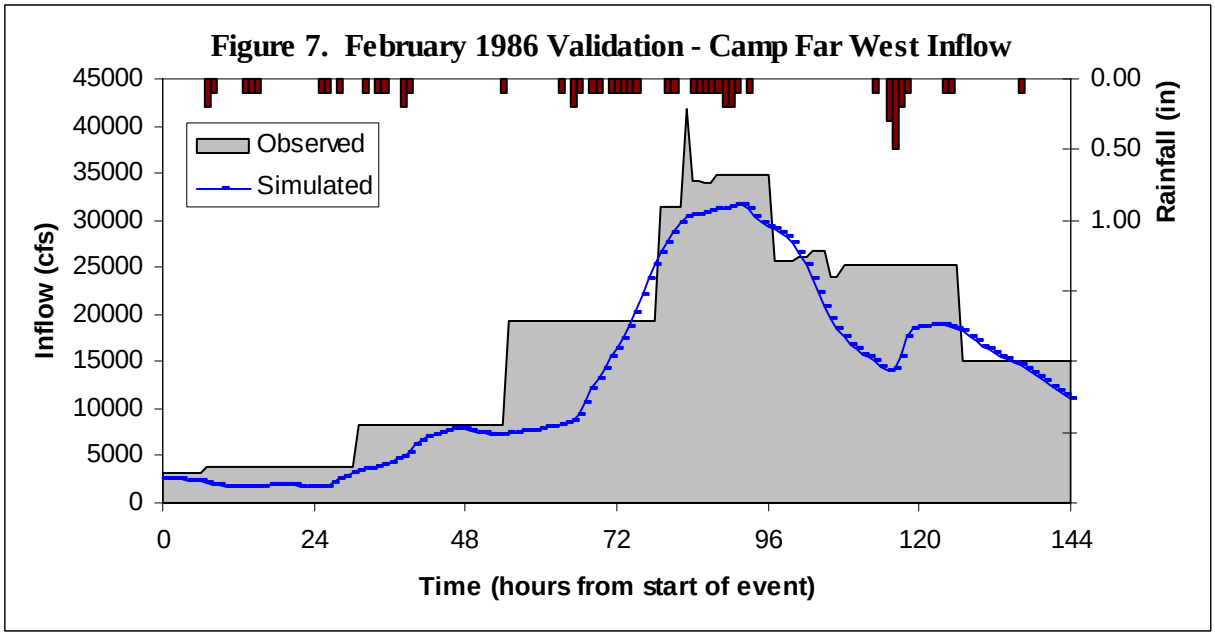
VI. Model Verification

The model calibration was validated using the February 1986 and March 1995 storm events. Meteorologic inputs for these events were developed as described in Section IV. It should be noted that, unlike the January 1997 event, there is limited information available to characterize the spatial distribution of rainfall within the basin for these events. Precipitation data for the 1986 and 1995 events were available for only three rainfall gages, all of which are outside or on the perimeter of the basin. Thus, the spatial distribution of precipitation for the validation events is subject to considerably greater uncertainty than the calibration event, for which detailed precipitation mapping was available.

Inflows to Camp Far West for March 1995 agreed very well with computed inflows (within 2.5 percent), though the peak is somewhat low. For the February 1986 event, flow volume is significantly undersimulated (-20 percent). As discussed above, the precipitation distribution for these two events was estimated from limited data, introducing greater uncertainty compared to the 1997 event. However, the additional simulations suggest that the model provides a reasonable representation of basin hydrologic response. Therefore, **nhc** believes the model calibration to be reasonable for a range of large events.

Figures 6 and 7 show simulated versus observed inflows to Camp Far West Reservoir for the March 1995 and February 1986 events.





VII. Probable Maximum Precipitation

Probable maximum precipitation (PMP) amounts were determined following the procedures of HMR 59 (NOAA, 1999), which covers PMP estimation for California. HMR 59 supersedes the PMP estimation procedures from HMR 36.

A. PMP Storm Type and Month of Occurrence

PMP amounts for the Camp Far West basin were determined for the "general storm" conditions outlined in HMR 59. General storms, which cover large areas for durations of up to 72 hours, approximate the frontal storms typical of the winter rainy season (November through March) that

generate the largest floods in the region. Thunderstorms, or other "local storm" events, are not critical for a basin of this size.

Seasonal variables have significant effects on both PMP and the resulting runoff, so the PMF depends on the time of year in which the PMP occurs. Differences in PMP depths, snowpack conditions, antecedent soil moisture, and meteorological conditions throughout the year require that the critical time for the PMF be identified. The critical flood period for the Camp Far West basin was identified as November through March for the following reasons:

- PMP depths are greatest for November through March, with the entire basin subject to the "all-season" PMP. PMP amounts throughout the rest of the year are reduced from this maximum amount over all or part of the basin.
- During the winter, antecedent conditions are more likely to lead to higher levels of surface runoff. The higher frequency of storms during the winter results in higher moisture levels in the soil column.
- Snowmelt, which can contribute significantly to runoff, will be greatest in months for which the snowpack is well developed (typically December through the spring melt)

PMP depths are the same over the entire November through March period. However, the associated antecedent and coincident hydrometeorological conditions (as determined from HMR 59) vary by month, so separate monthly simulations were required to identify the critical PMF for Camp Far West Dam.

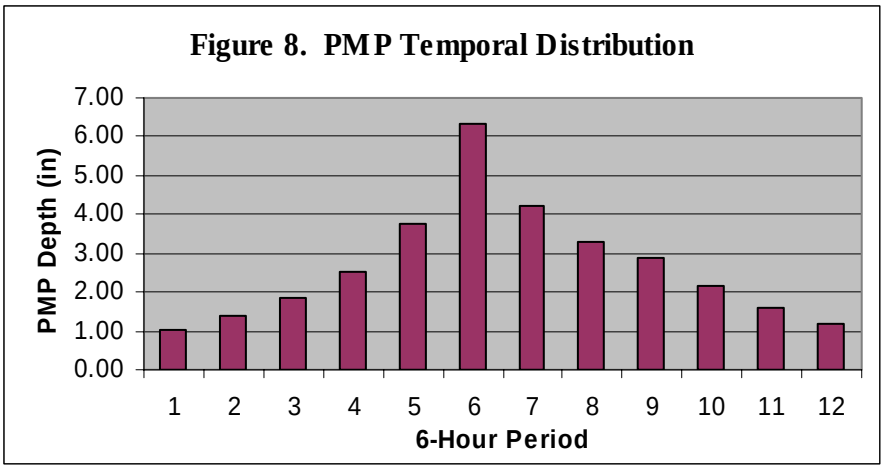
B. Estimation of PMP Depths

PMP depths were computed according to the procedures of HMR 59. The basin average 10-square mile, 24-hour PMP depth was computed as 21.4 inches using GIS techniques and a digitized version of the HMR 59 General Storm-PMP Index Map. Approximately two thirds of the Camp Far West basin lies in the Sierra depth-area-duration (DAD) region (region 5), with the remaining third in the Central Valley region (DAD region 4). November through March PMP amounts for the basin are shown in Table 6. To determine hourly PMP amounts, a curve was fit to the six duration points from the HMR analysis, and hourly amounts were calculated from the curve. This may result in slight differences (less than five percent) in the simulated PMP for some durations, compared to the calculated values in Table 7.

Table 7. November – March PMP Amounts

Basin Average PMP (inches) for Duration (hrs)					
1	6	12	24	48	72
2.11	6.75	10.82	17.18	27.21	32.16

HMR 59 does not provide detailed guidance on spatial distribution of PMP. For this analysis, the spatial distribution of the PMP was assumed to match the January 1997 storm, using the ratio of the 1997 subbasin total to basin total precipitation to determine subbasin PMP totals. The PMP was distributed temporally according to the pattern shown in Figure 8.



VIII. Antecedent and Coincident Conditions for the Probable Maximum Flood

A. Reservoir Operations

The water surface elevation in Camp Far West Reservoir was assumed to be at the spillway crest elevation of 300 feet msl four days prior to the PMP event, at the start of the antecedent 100-year storm. Based on operations during the 1986 and 1997 events and information from SSWD, a turbine discharge of 600 cfs was assumed throughout the PMF.

The starting water surface for Rollins Reservoir at the start of the antecedent event was also assumed to be at the spillway crest elevation, 2,171 feet msl.

B. Baseflow

The same initial baseflow per unit area used for the 1986 and 1995 historic events was assumed at the start of the antecedent event. (Due to an antecedent event, flows and reservoir elevations at the start of the 1997 event were higher than those for other events investigated.) However, with the reservoir assumed to be full, the PMF is not sensitive to the initial baseflow assumption.

C. Precipitation

To ensure conservative antecedent basin soil conditions and appropriate reservoir levels at the onset of the PMP, a 100-year, 24-hour rainfall event was assumed to start four days prior to the start of the PMP event. OCS mapping of the 100-year, 24-hour precipitation totals was used to determine the basin average point 100-year, 24-hour rainfall amounts. The average hourly rainfall amounts were then determined in a manner similar to the PMP depth estimates, using the same depth-duration and depth-area multipliers. The 100-year, 24-hour rainfall amounts for each subbasin are shown in Table 8.

Table 8. 100-year, 24-hour Rainfall Amounts

Basin Average Precip (in) for Duration (hrs)			
1	6	12	24
0.99	3.17	5.08	8.06

D. Snowpack

A 100-year snowpack was assumed to be in place four days prior to the start of the PMP event, i.e. at the start of the antecedent 100-year, 24-hour rainfall event. SWE data were obtained from eight SNOTEL and snow course gages (Table 9) and analyzed to determine the monthly 100-year SWE at each gage site. In addition, snow depth data were obtained for 11 National Weather Service stations at lower elevations.

Table 9. Snow Gages Used in PMF SWE Regression

CDEC ID	Station Name	Data Type (SWE or snow depth)	Gage Location		
			Latitude (deg min)	Longitude (deg min)	Elevation (ft msl)
DNS	Donner Summit	SWE	39 19	-120 20	6900
BOM	Bowman Lake	SWE	39 27	-120 36	5650
CMC	Chapman Creek	SWE	39 38	-120 32	5850
RBV	Robbs Valley	SWE	38 55	-120 23	5600
SPD	Lake Spaulding	SWE	39 19	-120 39	5200
TBC	Talbot Camp	SWE	39 12	-120 23	5750
BLC	Blue Canyon	SWE	39 17	-120 42	5280
GKS	Greek Store	SWE	39 05	-120 33	5600
n/a	Colgate Powerhouse	Snow Depth	39 19	-121 11	595
n/a	Englebright Dam	Snow Depth	39 14	-121 16	800
n/a	Dobbins 1 S	Snow Depth	39 21	-121 12	1640
n/a	Grass Valley No. 2	Snow Depth	39 12	-121 04	2400
n/a	Colfax	Snow Depth	39 05	-120 57	2400
n/a	Challenge Ranger Station	Snow Depth	39 29	-121 13	2570
n/a	Nevada City	Snow Depth	39 14	-121 00	2781
n/a	Foresthill Ranger Station	Snow Depth	39 00	-120 50	3015
n/a	Iowa Hill	Snow Depth	39 07	-120 50	3100
n/a	Gold Run 2 SW	Snow Depth	39 09	-120 51	3320
n/a	Deer Creek Forebay	Snow Depth	39 18	-120 50	4455

For January through March, frequency analyses were performed on beginning of month SWE values obtained from SNOTEL and snow course records, and the 100-year value was extracted from the theoretical distribution. Because snow course data are not available for November or December, equivalent record lengths were not available for those months. Based on analysis of early season data from two recording stations, 100-year SWE for December was assumed to equal that for January. November SWE for the records of the available stations was negligible. Although there

are likely to be rare occasions where some high elevation snowpack is present in November, snowmelt contribution would not be significant compared to the other winter months, so November will not produce the critical PMF.

For the low elevation snow depth stations, snow depths were identified for each year of record for December 15, January 1, February 1, and March 1 (for consistency with snow course data). As frequency distributions are not well-defined with numerous zero values, the 100-year snow depth was assumed to be the maximum over the period of record (over 50 years more most stations, up to 75 years). Snow depth was converted to SWE assuming 30 percent water equivalent. Following the determination of the monthly 100-year SWE for each gage, an elevation-based regression equation was developed to estimate the initial SWE for each elevation band (Figure 9 and Table 10).

Figure 9. PMF Initial Snow Water Equivalent

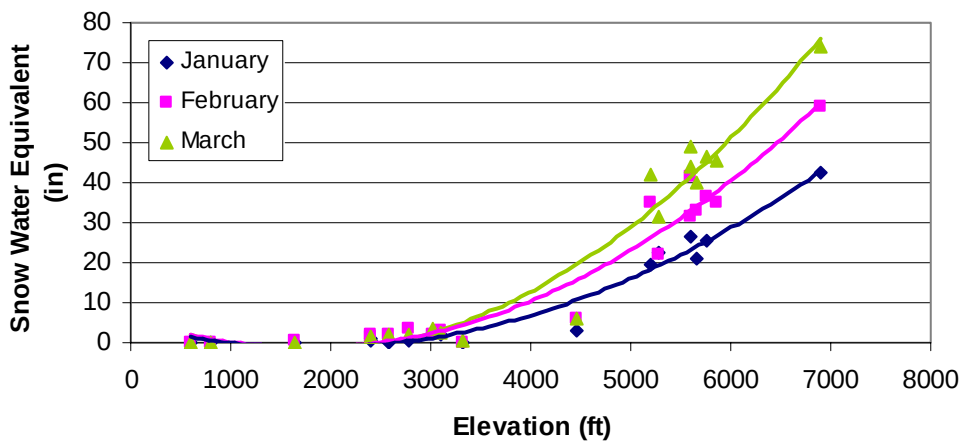


Table 10. PMF Initial Snow Water Equivalent

Elevation Band (feet)	January PMF	February PMF	March PMF
0-1000	0.0	0.0	0.0
1000-2000	0.0	0.0	0.0
2000-3000	0.0	0.1	0.0
3000-4000	3.5	5.9	6.8
4000-5000	10.9	16.2	20.1
5000-6000	21.8	31.2	39.3

The air temperatures prior to and during the PMP for each of the winter months were calculated based on the procedures of HMR 59. A pseudo-adiabatic lapse rate of 2.6 ºF per 1,000 feet was used for consistency with graphs and procedures in HMR 59.

HMR 59 provides curves for upper and lower bounds of temperature progressions for two days leading up to the PMP, and runs were made for both scenarios. For both scenarios, the 24-hour storm event four days prior to the PMP was assumed to occur at the starting PMP temperature. Temperatures the following day were interpolated to reach the two day temperatures indicated by the curves in HMR 59. The cool period prior to the PMP produced higher PMF flows, due to less

snowmelt before the PMP, so this was selected as the critical scenario. Results for the warmer scenario are reported in Section IX along with other sensitivity analyses.

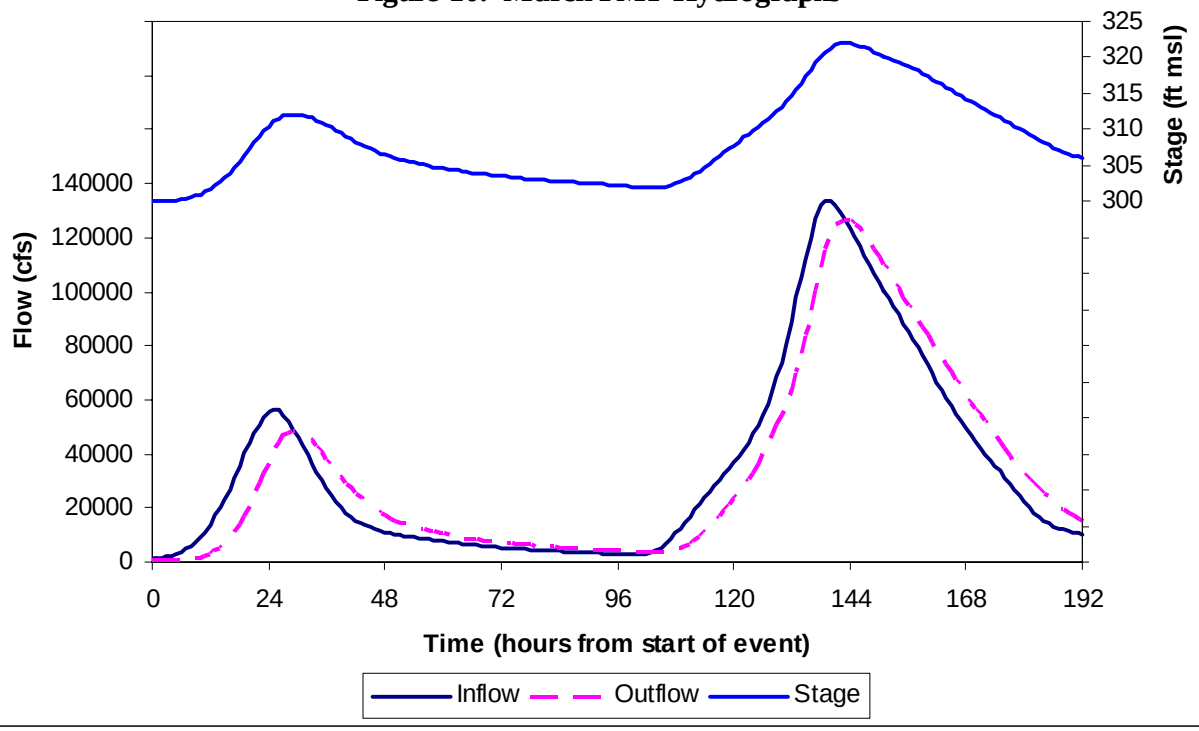
An alternative to the approach taken herein would be to simulate the PMP without an antecedent event using the 100-year snowpack as the initial snow condition. This scenario is described in Chapter 8 of FERC's PMF guidelines (FERC, 2001). However, for the Camp Far West basin, this scenario leaves significant uncertainty with regard to starting reservoir elevations and soil moisture conditions (i.e. initial loss), as very cold, dry conditions would need to be in place to maintain the low elevation snowpack going into the PMP. In such a case, it is possible that there would be soil storage available and that the reservoir level would be lower than simulated using the antecedent event approach. Because of these uncertainties, the antecedent storm approach was selected, though results for a scenario with no antecedent event are included in the sensitivity analyses in Section IX.

IX. PMF Hydrographs

A. Inflow PMF Hydrograph

The HEC-HMS model was run for the PMP and associated meteorological conditions for each month from December through March. November was not run because there is no significant snowmelt contribution for that month. Because temperatures and snowpacks are similar, differences in the PMF between months are very small. The March PMF, which has the highest starting SWE, is considered the critical storm with a peak inflow to Camp Far West Reservoir of approximately 134,100 cfs. The inflow hydrograph for the March PMF is shown in Figure 10. The peak inflow to the upstream Rollins Reservoir for the March PMF is 72,500 cfs. Rollins Dam has spillway capacity to pass this flow at a maximum water surface elevation of 2185.7 feet, leaving 1.3 feet of freeboard.

Figure 10. March PMF Hydrographs



B. Reservoir Outflow PMF

Maximum outflow from the Camp Far West spillway is approximately 126,600 cfs for the PMF. The peak reservoir water surface elevation reaches 321.95 feet msl (see Figure 10). This indicates approximately 2.0 feet of overtopping of the dam. The simulated water surface elevation exceeds the dam crest elevation for 13.7 hours during the PMF (with discharges computed assuming no flow over the dam crest). The Camp Far West Dam discharge capacity (spill plus turbine releases) is 107,100 cfs at the dam crest elevation of 320 feet, or 85 percent of the peak PMF outflow.

It should be emphasized that simulated stages above the dam crest are purely hypothetical, as discharge and volume were extrapolated and do not consider the effects of dam overtopping on discharges or actual topography on reservoir volumes. (Assuming weir flow over the length of the dam crest, including wing walls, above elevation 320, the maximum water surface elevation for the PMF reaches 320.70 feet msl.)

In addition to the PMF, FERC requires consideration of the effects of wind waves on estimated peak water surface elevations at the dam. Following the procedures described in Corps of Engineers EM 1110-2-1420 (Hydrologic Engineering Requirements for Reservoirs), **nhc** computed a design wind speed (over water) of approximately 60 mph with a minimum duration of 20 minutes. This design wind speed translates to an overland velocity of about 52 mph. The 1987 Safety Inspection Report included a detailed wind wave analysis and reported wave run-up heights for a range of wind velocities (Parsons Brinckerhoff, 1987). Camp Far West Reservoir has significant fetch length only for northeast winds, and the 1987 analysis found that wave run-up could exceed three feet for northeast winds in excess of 50 mph. Table 11, taken from the 1987 analysis, shows the maximum wave height at the dam for a range of northeast wind velocities. The velocities and durations used in the 1987 calculations are consistent with the procedures of EM 1110-2-1420. The 1987 report also noted that the 100-year northeast gust speed for nearby Beale Air Force Base was 52 mph.

Table 11. Wave Run-up Analysis

NE Overland Wind Velocity (mph)	Wave Height at Dam (ft)
15	0.89
20	1.20
25	1.51
30	1.83
35	2.15
40	2.44
45	2.77
50	3.06
55	3.36

Due to atmospheric circulation patterns, northeast winds typically do not occur until after a storm has moved through the area, so it is unlikely that maximum wind conditions would coincide with the PMF peak. However, if the design wind occurs within 19 hours of the peak, when the reservoir elevation drops to 317 feet, wind waves could contribute to dam overtopping. If the design wind occurs within 13 hours of the peak, wind waves would push the water surface elevation at the dam higher than the peak PMF reservoir elevation. The 1987 Safety Inspection Report suggests that, in

an extreme case, strong northeast wind conditions could develop 12 hours after the peak; the maximum elevation at the dam for this scenario would be 322.4 feet.

C. Sensitivity Analysis

Several assumptions required for simulation of basin runoff and flow routing of the PMF are somewhat more uncertain than others, or are of critical importance to the estimation of the PMF. These include:

- calibrated model loss rates,
- initial snowpack,
- temperature sequence prior to the PMP, and
- spatial and temporal distribution of PMP.

Sensitivity analyses were performed to assess the effects of variations in these parameters. Results are shown in Table 12.

Table 12. PMF Sensitivity Analysis

Simulation	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Stage (ft msl)
Baseline (Cold prior to PMP)	134,100	126,600	321.95
No turbine releases	134,100	126,600	322.01
Increase loss rates by 20%	131,200	123,500	321.64
Decrease loss rates by 20%	136,600	129,100	322.20
No snowmelt simulation	130,100	122,600	321.55
Warm period preceding PMP	131,000	123,400	321.63
No antecedent storm	138,600	130,200	322.31
PMP spatial alt 1	133,700	126,400	321.93
PMP temporal alt 1	138,400	130,800	322.37
PMP temporal alt 2	133,100	125,400	321.83

The sensitivity results indicate that the PMF simulation is fairly robust and not overly sensitive to any one parameter. All sensitivity analyses produced peak inflows to Camp Far West within 3.5 percent of the baseline PMF peak and peak stages within half a foot of the baseline. None of the alternative scenarios produced a PMF that could be passed at a water level below the crest of the dam.

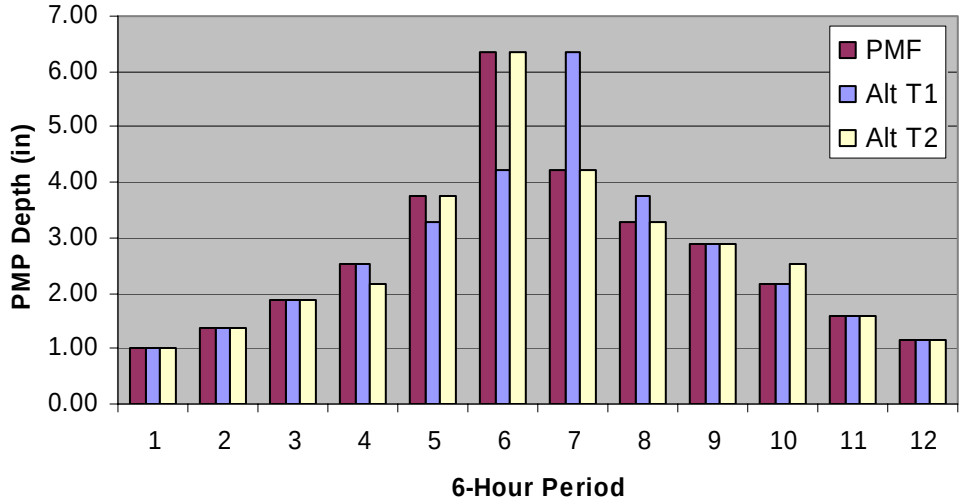
The largest inflow was simulated for the scenario without an antecedent storm event, with the PMP occurring after two days of warming temperatures starting with a basin-wide 100-year snowpack. As discussed in Section VIII, however, there is considerable uncertainty in initial conditions for this scenario. The scenario run for this analysis assumes starting reservoir elevations for Camp Far West and Rollins at the spillway crests and no initial losses, which is probably conservative given the conditions required to maintain lower elevation snowpack in this basin.

The distribution of the PMP in space and time has some impact on the resulting PMF. The PMF was not overly sensitive to an alternative spatial distribution—based on annual precipitation rather than the 1997 event distribution—as both scenarios put the majority of the rainfall on the relatively

impervious upper basin. A distribution centered on the lower portion of the basin would produce a lower PMF because loss rates are generally higher in the lower basin. However, that is not a typical storm pattern for the study area.

Changes in the temporal distribution of the PMP, i.e. how the defined 6-hour periods are ordered, can have a larger impact on simulated peak inflows. HMR 59 does not provide guidance on the ordering of the 6-hour PMP amounts. However, the FERC guidelines indicate that the peak 6-hour period would typically occur between the half and two-thirds point of the 72-hour PMP, with the remaining 6-hour amounts distributed alternately around the peak. Figure 11 shows the PMP distribution for two temporal alternatives compared with the PMP distribution selected for the PMF. As shown in Table 12, the inflow peak increases somewhat with the peak rainfall intensities moved later in the storm, when the whole basin is saturated and fully contributing runoff.

Figure 11. PMP Temporal Distribution Alternatives



D. Comparison with Previous Studies

As discussed in Section I, there have been several previous PMF studies for Camp Far West Dam. Table 13 shows a comparison of the PMF determined in this study with previous results. Unfortunately the reporting describing the previous PMF studies did not provide sufficient information on the assumptions and methods used to allow a detailed evaluation of the reasons for differences between the current study and the past efforts. However, as demonstrated by the simulated peak inflows and estimated PMPs in Table 13, the current study is apparently quite conservative in comparison to these previous efforts. Some of this difference is undoubtedly due to differences in the PMP estimation procedures in HMR 58. Other differences may include better model calibration, inclusion of snowmelt in the PMF simulations, more refined spatial distribution of rainfall and snow, and more conservative assumptions used in the hydrologic modeling.

Table 13. PMF Comparison

Study	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Stage (ft msl)	72-hr PMP[a] (inches)
Baseline (2005 Study)	134,100	126,600	321.95	32.2
USACE 1971[b]	136,000	n/a	n/a	29.9
DSOD 1980	112,700	102,700	319.76	30.2[c]
DSOD 1996	86,900	86,200	318.2	29.8

[a]Basin average 72-hour PMP
[b]1971 study for Garden Bar Dam Site, 4.5 miles upstream
[c]33.8 in for upper basin (above Lake Combie), 27.1 in for lower basin

X. References

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ATTACHMENT 7-2

Spillway Adequacy Evaluation

(final)

Camp Far West Hydroelectric Project

FERC Project No. 2997

Bear River

Report prepared for

South Sutter Water District

Trowbridge, California

Report prepared by

MEAD & HUNT

August 2006

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1. Introduction

The Camp Far West Dam Probable Maximum Flood (PMF) was recalculated by Northwest Hydraulic Consultants (NHC) and presented in the December 2005 revision to the December 2003 *Fifth Consultant's Safety Inspection Report, Camp Far West Hydroelectric Project*. This recalculation was requested by the Federal Energy Regulatory Commission (FERC) based on the new Hydrometerological Report Nos. 58 and 59, which established new Probable Maximum Precipitation (PMP) data for the western slope of the Sierra-Nevada Mountain Range in California. The spillway capacity of the Camp Far West Dam is 106,500 cubic feet per second (cfs) with zero freeboard. Previous reports have established the Probable Maximum Flood (PMF) as the Inflow Design Flood (IDF) for the project, and since this was less than the spillway capacity of 106,500 cfs, the spillway capacity had been considered adequate. Since the spillway capacity is less than the newly established PMF, the *Fifth Consultant's Safety Inspection Report* recommended that either an IDF study be performed to establish the IDF for the project, or that a site-specific PMP study should be performed.

In order to determine the IDF for the Camp Far West Dam, a dam failure analysis was performed by Mead & Hunt, Inc. Inflow conditions ranging from the just overtopping flood to the PMF condition were evaluated. Breach parameters were determined based on the current FERC *Engineering Guidelines for the Evaluation of Hydropower Projects* for dam failure analysis. Results of the analyses show that there are downstream impacts resulting from floods as high as the PMF. Based on these results, preliminary evaluations were made to determine the options available to meet the FERC guidelines for spillway capacity. Of the options considered, a selection of the IDF equal to the existing spillway capacity is recommended and discussed herein.

A. Certification

The undersigned, a registered professional engineer, does hereby certify and state that he is an employee of Mead & Hunt, Inc., Engineers, Architects, Scientists, Planners; that he has been designated as being in responsible charge of the preparation of this report for the Camp Far West Dam and Hydroelectric Project; that the engineering evaluation was performed in accordance with Part 12 of the FERC regulations; that he has read the FERC regulations and understands their intent and purpose; that the inspection work was done by him or under his direct supervision; that he approved this August 2006 *Spillway Adequacy Evaluation*; and that the conclusions and recommendations herein are based on his independent opinion and are made independently of the owner, its employees, and its representatives. No other independent consultant participated in this inspection and report.

(1) Reference to FERC Order 122 dated March 1, 1981 and paragraph 12.37(c)(7)

The conclusions and recommendations herein are based on the independent inspector's opinion and are made independently of the owner, its employees, and its representatives as required in FERC Order 122 Dated March 1, 1981, and Paragraph 12.37(c)(7).

(2) Signature of Part 12D Independent Consultant

Eric A. Van Deuren, P.E.

(3) PE Stamp

2. Description of the Project

The Camp Far West Dam was constructed in 1963 and was designed primarily for irrigation purpose. The dam is located on the Bear River in northern central California, approximately 5½ miles northeast of Sheridan, Placer County, and 6½ miles east of Wheatland, Yuba County. The Bear River joins the Feather River about 17 miles downstream from the Camp Far West Dam.

Main structures of the Camp Far West Dam include the south wing dam, the main dam, the powerhouse, the uncontrolled spillway, the north wing dam, and the north dike.[1] The south wing dam is a 1,060-foot-long earth-fill section with a maximum height of 45 feet and a crest elevation at 320 feet.[2] The main dam is 2,070 feet long and 185 feet high with a crest elevation at 320 feet. The upstream slope of the main dam varies from 2:1 to 3:1 (H:V), and the downstream slope is 2:1. The uncontrolled spillway is an ungated, ogee-type reinforced concrete structure with a crest length of 300 feet. The spillway crest is at elevation 300 feet. Maximum spillway capacity at the main dam crest elevation of 320 feet is approximately 106,500 cfs. The north wing dam is a 1,460-foot-long earth-fill section with a maximum height of 25 feet and a crest elevation at 320 feet. The north dike is a 1,450-foot-long earth-fill section with a maximum height of 12 feet and a crest elevation at 320 feet.

The Camp Far West Reservoir has a storage capacity of 104,400 acre-feet and a surface area of 2,020 acres at its maximum normal water surface elevation of 300 feet. The reservoir has a potential gross storage of 150,400 acre-feet at elevation 319.76 feet with a corresponding surface elevation of 2,650 acres. At the minimum pool level of 175 feet, the reservoir has a storage capacity of 2,200 acre-feet and a surface area of about 150 acres.

A vicinity map of the project location is shown in Appendix A, and plan views of the dam are shown in Appendix B. The spillway rating curve for the Camp Far West Dam is shown in Appendix C.

[1] The project structures are referred to in this report as left and right looking downstream.

[2] Elevations are referenced to mean sea level throughout this report.

3. Dam Breach Model

The latest version of HEC-RAS unsteady-state flow model (Version 3.1.3) developed by the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) was used for this dam breach study. The HEC-GeoRAS 4.1.1 extension was utilized to develop the geometric data for the HEC-RAS model using available geospacial data. An electronic copy of the HEC-RAS model accompanies this report.

A. Study reach and geometric data

The study reach extended from the upstream end of the Camp Far West reservoir to the junction of the Bear River and the Feather River with a total length of approximately 20 miles. Twenty-two cross sections were developed in the study reach. Geometry of these cross sections was obtained using HEC-GeoRAS tools.

The geospacial data for the study area was obtained from the U.S. Geological Survey (USGS) National Elevation Dataset (NED). It is the best available Digital Elevation Model (DEM) data provided by the USGS Seamless Data Distribution System (SDDS). The DEM data was later converted into the Triangulated Irregular Network (TIN) format for HEC-GeoRAS processing. The river alignment, cross sections, and levee locations were developed using the TIN and HEC-GeoRAS utilities.

The geometry data was then extracted and converted into a GIS Import File that was importable into HEC-RAS. Cross-section data was verified and completed based on the HEC-RAS model from the Lower Feather River Floodplain Mapping Study (USACE Sacramento District, February 2005) and the USGS topographic maps. Elevations of the levees were also obtained from the Lower Feather River Floodplain Mapping Study and the USGS topographic maps. Geometry of the Camp Far West Dam was obtained from project drawings in the *Supporting Technical Information Document* (STID) for the Camp Far West Dam (Mead & Hunt, December 2003).

Index maps are included in this report as Appendix D.

B. Manning's roughness coefficients

The Manning's roughness coefficients (Manning's "n") for the river channel and its overbank areas were estimated based on aerial photos of the study area, and adjusted by calibrating the model to match the known stage–discharge relationship at the USGS gage station 11424000 near Wheatland on the Bear River, approximately 5½ miles downstream from the Camp Far West Dam. The Manning's "n" value was determined to be 0.03 for the main channel and 0.06 for the overbank areas in the reservoir, and 0.03 and 0.05 for the main channel and overbank, respectively, downstream of the dam.

C. Inflow conditions and lateral flow(s)

The PMF hydrograph for the Camp Far West Dam was developed in March 2006 as part of the new PMF study. The peak of the new PMF inflow was determined to be approximately 134,000 cfs. The overtopping inflow was determined to be about 110,500 cfs, corresponding to a spillway outflow of approximately 106,700 cfs at elevation 320.03 feet. The overtopping flow hydrograph was obtained by applying a multiplier of 0.825 to the PMF hydrograph. The PMF and overtopping inflow hydrographs at the Camp Far West Dam are shown in Appendix E.

Rivers and tributaries in the study area other than the Bear and Feather Rivers are the Dry Creek, the Yankee Slough, and the Best Slough. Both the Yankee Slough and the Best Slough have flows less than 10 percent of the flow in the Bear River, and, therefore, were not considered in the HEC-RAS model. Flow in the Dry Creek was approximately 20 percent of that in the Bear River at a 100-year event, and, hence, was treated as a lateral flow to the Bear River, but the physical creek was neglected in the model in order to obtain reasonably conservative results. The Dry Creek was assumed to experience the 100-year flood when the Camp Far West Dam was passing the overtopping or the PMF flow. Inflow hydrograph for the Dry Creek was obtained from the *Lower Feather River Floodplain Mapping Study*. The 100-year flood hydrograph for the Dry Creek is shown in Appendix F.

D. Boundary condition

The Bear River is a tributary to the Feather River. The 100-year floods on the Bear and Feather Rivers and the PMF on the Bear River are compared below:

100-year flood on the Bear River at Wheatland: 44,300 cfs
PMF on the Bear River at Camp Far West Dam: 134,000 cfs
100-year flood on the Feather River at Nicolaus: 323,000 cfs

It can be noted that the PMF at the Camp Far West Dam is approximately 41 percent of the 100-year flood on the Feather River. Therefore, it was assumed that the water surface elevation at the downstream end of the study reach (junction of the Bear and Feather Rivers) remained at the 100-year flood level of 52.78 feet. The value of the 100-year flood elevation on the Feather River was obtained from the *Lower Feather River Floodplain Mapping Study* and was assumed to be the downstream boundary condition for the HEC-RAS model.

E. Model calibration

The USGS gage station 11424000 is located near Wheatland on the Bear River. It is a real-time gage station and has recorded flow data since 1929. The water surface elevations at this location were calibrated to be within 2 feet of the difference with the measured stages at the gage station at floods ranging from 3,240 to 48,000 cfs between the years 1982 and 1992.

Storage of the Camp Far West reservoir was calibrated to match the maximum storage capacity at the pool elevation of 319.76 feet. The weir coefficient of the Camp Far West Dam was calibrated based on the spillway rating curve of the dam.

F. Breach assumptions

The breach was assumed to occur in the highest part of the main dam when the water level in the reservoir dam reached the peak of the no-failure stage hydrograph. The breach parameters were selected based on FERC criteria, to represent a severe dambreak scenario. The dam was assumed to fail down to the foundation elevation of 135 feet, with an average breach width (740 feet) of four times the height of the dam (185 feet) and a breach side slope of 1:1. Because of the large size of the reservoir, the water level in the reservoir would draw down slowly and the peak outflow is not sensitive to breach formation time. Therefore, the breach time was assumed to be 1 hour. Table 1 summarizes dam breach assumptions used in the analysis. A downstream view of the breach is shown in Appendix G.

Table 1
Assumed Breach Parameters for Camp Far West Dam

	Overtopping Failure	PMF Failure
Location of breach	Main Dam	Main Dam
Bottom elevation	135 feet (Rock foundation)	135 feet (Rock foundation)
Breach height	185 feet	185 feet
Breach bottom width	555 feet (Average breach width = four times breach height)	555 feet (Average breach width = four times breach height)
Side slopes	1:1	1:1
Time to develop maximum size	1 hour	1 hour
Reservoir elevation to initiate breaching	320 feet (top of main dam crest; 0.03 foot below peak stage)	320.9 feet (0.04 foot below peak stage)

4. Results of Analysis

The overtopping inflow is approximately 84 percent of the PMF. Model run results showed that most of the levees along the Bear River would be overtopped if the dam failed and, therefore, the HEC-RAS model runs for the dam failure scenarios assumed that all of the levees were failed, while for the no-failure scenarios, the levees were assumed to stay intact for all inflow conditions.

Table Nos. 2 and 3 summarize results from the model runs. The model output summaries are attached as Appendix H. An electronic version of the HEC-RAS model accompanies this report.

Table 2
Summary of Downstream Flow Conditions at Overtopping Flow (84-percent PMF)
Peak Inflow = 110,500 cfs

Cross Section No.	Location Downstream of Dam (miles)	Levee Elevation (feet)	Peak Elevation With Failure (feet)	Peak Elevation Without Failure (feet)	Incremental Rise (feet)	Peak Flow With Failure (cfs)
94971 (Camp Far West Dam TW)	0.14		250.7	162.7	88.0	2,482,500
80961	2.79		140.6	123.0	17.6	1,440,900
68898 (Sheridan on left bank)	5.08	109.09 (L) 109.09 (R)	113.8	109.0	4.8	1,774,300
64526 (Sheridan on left bank)	5.90	109 (L) 108.54 (R)	104.5	107.9	-3.4	1,405,700
60526 (Wheatland on right bank)	6.66	104.18 (L) 105.03 (R)	96.9	103.7	-6.8	1,565,900
55349 (Wheatland on right bank)	7.64	99 (L) 100 (R)	91.5	98.2	-6.7	1,216,100
51931	8.29	93.9 (L) 96.55 (R)	89.4	93.5	-4.1	966,400
42954	9.99	84.06 (L) 84.12 (R)	83.3	84.2	-0.9	1,003,400
27087 (Dry Creek and Bear River junction)	12.99	66.28 (L) 65 (R)	70.1	61.9	8.2	907,600
19841 (Rio Oso on left bank)	14.37	61.23 (L) 60.91 (R)	68.7	57.2	11.5	895,200

Table 2
Summary of Downstream Flow Conditions at Overtopping Flow (84-percent PMF)
Peak Inflow = 110,500 cfs

Cross Section No.	Location Downstream of Dam (miles)	Levee Elevation (feet)	Peak Elevation With Failure (feet)	Peak Elevation Without Failure (feet)	Incremental Rise (feet)	Peak Flow With Failure (cfs)
13315 (Rio Oso on left bank)	15.60	62.27 (L) 59.36 (R)	68.2	55.1	13.1	657,000
5290 (Bear River and Feather River junction)	17.12	60.95 (L) 58.93 (R)	52.8	52.8	0	556,700

Table 3
Summary of Downstream Flow Conditions at PMF
Peak Inflow = 133,900 cfs

Cross Section No.	Location Downstream of Dam (miles)	Levee Elevation (feet)	Peak Elevation With Failure (feet)	Peak Elevation Without Failure (feet)	Incremental Rise (feet)	Peak Flow With Failure (cfs)
94971 (Camp Far West Dam TW)	0.14		251.6	166.2	85.4	2,816,400
80961	2.79		140.8	123.9	16.9	1,609,700
68898 (Sheridan on left bank)	5.08	109.09 (L) 109.09 (R)	114.0	109.3	4.7	1,724,200
64526 (Sheridan on left bank)	5.90	109 (L) 108.54 (R)	104.7	108.4	-3.7	1,646,900
60526 (Wheatland on right bank)	6.66	104.18 (L) 105.03 (R)	97.2	104.0	-6.8	1,388,500
55349 (Wheatland on right bank)	7.64	99 (L) 100 (R)	91.7	98.4	-6.7	1,229,300
51931	8.29	93.9 (L) 96.55 (R)	89.7	93.7	-4.0	1,084,700
42954	9.99	84.06 (L) 84.12 (R)	83.4	86.7	-3.3	991,400
27087 (Dry Creek and Bear River junction)	12.99	66.28 (L) 65 (R)	70.9	67.7	3.2	845,900

Table 3
Summary of Downstream Flow Conditions at PMF
Peak Inflow = 133,900 cfs

Cross Section No.	Location Downstream of Dam (miles)	Levee Elevation (feet)	Peak Elevation With Failure (feet)	Peak Elevation Without Failure (feet)	Incremental Rise (feet)	Peak Flow With Failure (cfs)
19841 (Rio Oso on left bank)	14.37	61.23 (L) 60.91 (R)	69.6	62.9	6.7	720,700
13315 (Rio Oso on left bank)	15.60	62.27 (L) 59.36 (R)	69.1	61.4	7.7	544,400
5290 (Bear River and Feather River junction)	17.12	60.95 (L) 58.93 (R)	52.8	52.8	0	511,100

Table 2 shows that, at the overtopping inflow without a dam failure, only one location (Cross Section 42954) approximately 10 miles downstream from the dam experienced levee overtopping by less than 0.1 foot. However, after the dam failed, almost all of the levees were overtopped and failed. The incremental rise in the water surface elevation that resulted from the dam breach was much higher than 2 feet in areas immediately below the dam and below the Dry Creek junction on the Bear River. In the area between Sheridan and the Dry Creek junction, the water level dropped after the dam breach because of a much larger flow area resulting from overtopping and the assumed levee breach along the Bear River.

Table 3 shows that, at the PMF condition, levees at various locations were already overtopped even before the dam failed, but some levees near the Sheridan and Wheatland area were still slightly higher than the water level in the Bear River before the dam failure. All of the levees were overtopped and failed after the dam failure. Similar to the overtopping condition, water level in the Bear River increases significantly at some locations and decreased between Sheridan and the Dry Creek junction due to the levee failure.

5. Downstream Development

Downstream developments along the Bear River were identified based on the USGS topographic maps and aerial photos of the study area. Major development areas along the Bear River below the Camp Far West Dam included Sheridan, Wheatland, and Rio Oso. Exact values of the first-floor elevations of structures in the development areas were not indicated on the topographic maps; however, estimates of the structures' elevations based on contours on the maps were sufficient for the purpose of this study.

Table 4 compares the approximate first-floor elevations of the structures to the flood elevations after the dam failure.

Table 4
Comparison of First-Floor Elevations and Flood Elevations After Dam Failure at Developments Downstream of the Camp Far West Dam

Cross Section No.	Location Downstream of Dam (miles)	First-Floor Elevation (feet)	Peak Elevation With Failure @ Overtopping (feet)	Peak Elevation With Failure @ PMF (feet)
94971 (Camp Far West Dam TW)	0.14		250.7	251.6
80961	2.79	175 ~ 260 (L)	140.6	140.8
68898 (Sheridan on left bank)	5.08	100 ~ 130 (L) 110 ~ 125 (R)	113.8	114.0
64526 (Sheridan on left bank)	5.90	100 ~ 115 (L) ~ 90 (R)	104.5	104.7
60526 (Wheatland on right bank)	6.66	90 ~ 97 (L) 80 ~ 85 (R)	96.9	97.2
55349 (Wheatland on right bank)	7.64	~ 75 (L) 75 ~ 85 (R)	91.5	91.7
51931	8.29	75 ~ 80 (L) 65 ~ 80 (R)	89.4	89.7
42954	9.99	67 ~ 71 (L) ~ 68 (R)	83.3	83.4
27087 (Dry Creek and Bear River junction)	12.99	50 ~ 55 (L) ~ 55 (R)	70.1	70.9

Table 4
Comparison of First-Floor Elevations and Flood Elevations After Dam Failure at Developments Downstream of the Camp Far West Dam

Cross Section No.	Location Downstream of Dam (miles)	First-Floor Elevation (feet)	Peak Elevation With Failure @ Overtopping (feet)	Peak Elevation With Failure @ PMF (feet)
19841 (Rio Oso on left bank)	14.37	~ 45 (L)	68.7	69.6
13315 (Rio Oso on left bank)	15.60	40 ~ 50 (L) ~ 42 (R)	68.2	69.1
5290 (Bear River and Feather River junction)	17.12	35 ~ 45 (L)	52.8	52.8

The above table shows that, a dam failure under either the overtopping or the PMF condition would cause severe inundation of structures at all three major development areas below the Camp Far West Dam.

6. Dam Failure Analysis Discussions and Conclusions

Results from the dam failure analysis indicated that if the Camp Far West Dam failed immediately upon overtopping (with an inflow of 110,500 cfs), the incremental rise in the Bear River below the dam would be greater than 2 feet at developed areas such as Sheridan, Wheatland, and Rio Oso. The analysis results also showed that if the dam failed at the PMF, the incremental rise downstream will still be greater than 2 feet. For all floods between just overtopping and the PMF, most of the downstream levees would not be overtopped without a dam failure, but would be overtopped from the flood wave resulting from a dam failure. Based on these analyses, the spillway capacity requirements under the FERC guidelines would be to be able to safely pass the PMF.

Based on the discussions above regarding the results of the dam failure analysis, alternatives are addressed in the following section of the report to address the apparent spillway capacity issues.

7. Spillway Modification Alternatives

Since the results of the dam failure analysis show that the current spillway capacity does not meet FERC guidelines, several alternatives are investigated and discussed below. In addition to the traditional structural alternatives to increase spillway capacity, there are non-traditional options that include modifications to the dam to protect the high sections from failing, as well as further evaluation of the option to do nothing and look at the mitigating factors affecting the selection of the IDF. The following alternatives are addressed in this report:

Alternative 1 – Raise the dam
Alternative 2 – Lower critical sections of the dam and allow lower hazard sections to overtop
Alternative 3 – Adding an auxiliary spillway
Alternative 4 – Lower the crest of the existing uncontrolled spillway
Alternative 5 – Do nothing and set the IDF equal to the spillway capacity

Sketches of the alternatives and associated cost estimates are presented in Appendix J.

A. Alternative 1: Raising the dam

As stated in the previous sections, an overtopping flow without a dam failure would not overtop the levees along the Bear River; however, a flood equal to the PMF without a dam failure would result in overtopping of most of the levees along the Bear River below the dam. As such, any dam failure at a flow greater than the spillway capacity, even if it occurs at a shallow section of the dam, would likely cause an increase in the outflow from the dam that would result in the overtopping of downstream levees that would not have been overtopped had the dam not failed. Therefore, this alternative consists of providing a parapet wall on the upstream face of the dam to prevent overtopping of the dam up to the PMF.

To prevent overtopping of the dam up to the PMF the dam would have to be raised approximately 3 feet. This alternative accomplishes this objective by adding a 3-foot-high parapet wall on the upstream crest of the embankment. The parapet may actually have to be somewhat higher to provide some freeboard during the PMF. The HEC-RAS model output is summarized in Appendix H.

Estimated construction cost for Alternative 1 = $4.1 Million

B. Alternative 2: Lowering critical section of the dam

This alternative consists of lowering the entire 1,450-foot-long north dike by 4 feet. This would allow the dam to pass the PMF without overtopping other sections of the dam. Since the north dike is only 12 feet high at its maximum section, a failure of this section of the dam would only result in a small increase in the outflow from the dam.

A key component of this alternative is routing of the flow passing over the north dike. Topography dictates that this water will not be directed back into the Bear River without a long diversion through downstream developments that would not have otherwise been flooded. Therefore, a training wall will

need to be constructed to direct the water flowing over the north dike back into the Bear River just downstream of the dam.

For this alternative, at the maximum pool elevation of 320 feet, the total spillway capacity will consist of two portions – the flow going over the uncontrolled spillway and the flow going over the lowered north dike crest. The two flows are summarized below:

Flow over uncontrolled spillway: 104,500 cfs
Flow over north dike (weir coefficient = 2.6): 30,200 cfs
Total flow from dam: 134,700 cfs

The values above show that the total spillway capacity after the modification is slightly higher than the PMF at the maximum pool elevation of 320 feet, demonstrating that only the north dike will be overtopped when the inflow is equal to the PMF. With this configuration, the crest of the north dike is at 316 feet, and the overtopping flow for the north dike will occur when the spillway discharge exceeds approximately 74,300 cfs, or about 56 percent of the PMF. The flood- frequency curve for the Bear River indicates that 74,300 cfs is approximately equal to the 350-year flood.

A set of HEC-RAS runs were performed to simulate a breach of the north dike at overtopping. The breach was assumed to occur at the deepest section of the north dike with an average breach width (40 feet) of five times the breach depth (8 feet). The breach formation time was assumed to be 0.5 hour. Results of the HEC-RAS runs are summarized in Table 5 and the model output is included in Appendix H.

Table 5 indicates that, if the north dike fails at the overtopping flow, the incremental rise downstream from the dam will be less than 0.3 foot, and the additional outflow due to the north dike failure will be less than 4 percent of the total outflow from the dam. Therefore, a failure occurring at the north dike with this alternative will not increase the downstream flooding hazard significantly.

Estimated construction cost for Alternative 2 = $2.7 Million

Table 5
Summary of Downstream Flow Conditions at Overtopping Flow at North Dike
Peak Inflow = 74,300 cfs

Cross Section No.	Location Downstream of Dam (miles)	Levee Elevation (feet)	Peak Elevation With Failure (feet)	Peak Elevation Without Failure (feet)	Incremental Rise (feet)	Peak Flow With Failure (cfs)	Peak Flow Without Failure (cfs)
94971 (Camp Far West Dam TW)	0.14		156.4	156.1	0.3	72,100	69,400
80961	2.79		121.2	121.1	0.1	72,100	69,400
68898 (Sheridan on left bank)	5.08	109.09 (L) 109.09 (R)	108.6	108.5	0.1	70,400	69,600
64526 (Sheridan on left bank)	5.90	109 (L) 108.54 (R)	106.9	106.8	0.1	68,600	67,900
60526 (Wheatland on right bank)	6.66	104.18 (L) 105.03 (R)	101.1	101.0	0.1	68,600	67,900
55349 (Wheatland on right bank)	7.64	99 (L) 100 (R)	95.3	95.3	0	68,600	67,900
51931	8.29	93.9 (L) 96.55 (R)	92.6	92.6	0	68,600	67,900
42954	9.99	84.06 (L) 84.12 (R)	84.0	84.0	0	67,500	67,100
27087 (Dry Creek and Bear River junction)	12.99	66.28 (L) 65 (R)	58.0	58.0	0	66,800	66,700
19841 (Rio Oso on left bank)	14.37	61.23 (L) 60.91 (R)	54.8	54.8	0	66,800	66,700
13315 (Rio Oso on left bank)	15.60	62.27 (L) 59.36 (R)	53.5	53.5	0	66,700	66,600
5290 (Bear River and Feather River junction)	17.12	60.95 (L) 58.93 (R)	52.8	52.8	0	66,700	66,700

C. Alternative 3: Auxiliary spillway

For this alternative an uncontrolled ogee-shaped auxiliary spillway will be constructed parallel to the existing spillway to allow the dam to pass the PMF without overtopping. The parallel spillway will be approximately 85 feet long with a crest elevation of 300 feet. Flows over two uncontrolled spillways at the maximum pool elevation of 320 feet are summarized below:

Flow over existing spillway: 104,500 cfs
Flow over parallel spillway: 29,600 cfs
Total flow from dam: 134,100 cfs

Estimated construction cost for Alternative 3 = $3.2 Million

D. Alternative 4: Lowering existing uncontrolled spillway

For this alternative the spillway capacity will be increased to pass the PMF without overtopping the dam by lowering the existing uncontrolled ogee-shape spillway. There are two ways to lower the existing uncontrolled spillway crest:

- Construct a lower ogee-shaped spillway.
- Flatten the existing spillway crest down to the required level.

The two sub-alternatives are discussed below. The HEC-RAS model output is summarized in Appendix H.

(1) Sub-alternative 4-1 – Lower ogee-shaped spillway

If an ogee-shaped spillway crest is to be maintained after the modification, the discharge coefficient will remain the same. The HEC-RAS model run shows that, due to the high efficiency of an ogee-shaped spillway crest, the existing spillway crest only needs to be lowered by about 3 feet in order to increase the maximum spillway capacity to the PMF. After the modification, 3-foot-high flashboards will need to be installed in order to keep the current maximum normal pool elevation of 300 feet prior to spilling and to maintain the usable storage capacity of the reservoir.

Estimated construction cost for Alternative 4-1 = $1.2 Million

(2) Sub-alternative 4-2 – Lower spillway with flat crest

This sub-alternative simplifies the spillway modification procedure by flattening the crest down without keeping the existing ogee shape. However, since a flat spillway crest cannot pass the flow as efficiently as an ogee-shaped crest, a lower discharge coefficient was used in the HEC-RAS run for this sub-alternative. The HEC-RAS results show that the spillway crest needs to be lowered by about 7 feet in order to increase the maximum spillway capacity to the PMF. For this option, a 7-foot-high flashboard will need to be installed after the modification in order to maintain the current maximum normal pool and usable storage capacity of the reservoir.

Estimated construction cost for Alternative 4-2 = $1.4 Million

E. Alternative 5: Do nothing and accept existing spillway capacity as adequate

This alternative presents discussions for accepting the existing spillway capacity as adequate. For this alternative, no additional spillway capacity would be added to the dam and the IDF is proposed to be equal to the PMF. Although this is a deviation from the FERC guidelines, there are several mitigating factors that warrant consideration of this alternative:

- One option to mitigate for the spillway capacity at Camp Far West Dam would be to conduct a site-specific PMP study. These types of studies have been done around the country with resulting PMP values being reduced between 15 and 25 percent from those presented in the respective Hydrometerological reports, which tend to use more conservative regional parameters in the determination of the PMP maps. A 15-percent reduction in the PMP would drop the PMF inflow from 133,900 cfs to 113,800 cfs, which is only slightly greater than the inflow of 110,500 cfs required to just overtop the dam. It is therefore reasonable to assume that a site-specific PMP study has a good probability of eliminating the need for additional spillway capacity.

- The annual peak flood-frequency curves at the Bear River near Wheatland were developed by the USACE Sacramento office. Extrapolation of the flood-frequency curves shows that the current spillway capacity of 106,500 cfs is approximately equal to the 1,200-year flood. The original flood-frequency curves developed by the USACE and the extrapolated curve are shown in Appendix I. The spillway capacity flood is also equal to approximately two times the 100-year flood and approximately three times the flood of record. This exercise demonstrates that the spillway capacity flood is a very extreme flood with a very low probability of occurrence.

- The dam failure analyses show that the incremental hazard resulting from a dam failure is controlled by overtopping of the Bear River levees and downstream Feather and Sacramento River levees. Overtopping of these levees resulting from a dam failure will be equally as dependent on the downstream flooding conditions as it will be on the dam failure flood wave itself. This makes the determination of the IDF somewhat more subjective than normal, where if the downstream flooding is less than that assumed in the model, those levees may not fail during a failure of the Camp Far West Dam. Further, if the downstream flooding is greater, those levees may have already failed, thus reducing the impact of the Camp Far West Dam failure. Following this logic and applying it to the

proposed alternatives to increase the spillway capacity of the dam, it puts into question the true benefits of completing the dam modifications.

8. Conclusions and Recommendations

Based on the dam failure analyses performed for this study, the spillway capacity at Camp Far West Dam does not currently meet the criteria set forth in the FERC guidelines. The spillway capacity is 106,500 cfs, or 84 percent of the PMF outflow of 126,600 cfs. Dam failure analyses show that, depending on the downstream flooding conditions, failures of the Camp Far West Dam could result in incremental increases in the river levels that would overtop levees protecting developed areas.

Based on this conclusion, an alternatives evaluation was performed to look at potential conceptual designs to mitigate for the spillway capacity deficiencies. Several structural modifications were evaluated to modify the dam to safely pass the PMF and select the PMF as the IDF for the project along with the option to accept the current spillway capacity as the IDF for the project. This report recommends the option of accepting the current spillway capacity as the IDF for the following reasons:

- If a site-specific PMP study were performed for the dam, a reduction in the PMP between 15 and 25 percent could be reasonably expected. A 15-percent reduction applied to the PMF inflow would drop the PMF inflow from 133,900 cfs to 113,800 cfs, which is only slightly greater than the inflow of 110,500 cfs required to just overtop the dam.

- The current spillway capacity of 106,500 cfs is approximately equal to a 1,200-year flood, two times the 100-year flood and three times the flood of record, demonstrating that the spillway capacity flood is a very extreme event with a very low probability of occurrence.

- The dam failure analyses show that the incremental hazard resulting from a dam failure is controlled by overtopping of the Bear River levees and downstream Feather and Sacramento River levees. Overtopping of these levees resulting from a dam failure will be equally as dependent on the downstream flooding conditions as it will be on the dam failure flood wave itself. This makes the determination of the IDF somewhat more subjective than normal, where if the downstream flooding is less than that assumed in the model, those levees may not fail during a failure of the Camp Far West Dam. Further, if the downstream flooding is greater, those levees may have already failed, thus reducing the impact of the Camp Far West Dam Failure. Following this logic and applying it to the proposed alternatives to increase the spillway capacity of the dam, it puts into question the true benefits of completing the dam modifications.

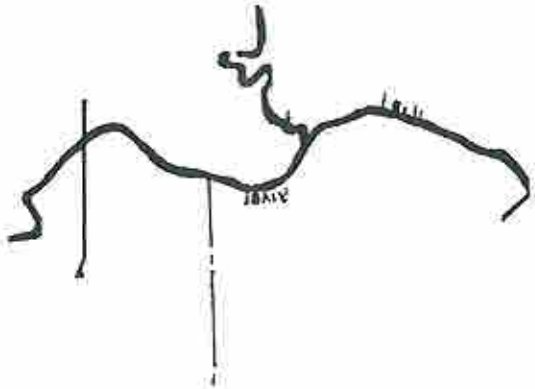
References

Mead & Hunt, Inc., *2003 Fifth Consultant's Safety Inspection Report, Camp Far West Dam and Hydroelectric Project*, FERC Project No. 2997, South Sutter Water District, Trowbridge, California, December 2003 (revised December 2005).

Mead & Hunt, Inc., *Supporting Technical Information Document, Camp Far West Dam Hydroelectric* Project, South Sutter Water District, Trowbridge, California, December 2003.

U. S. Army Corps of Engineers Sacramento District, *Amended Draft, Lower Feather River Floodplain Mapping Study*, State of California Department of Water Resources, February 17, 2005.

Appendix A. Project Location Map



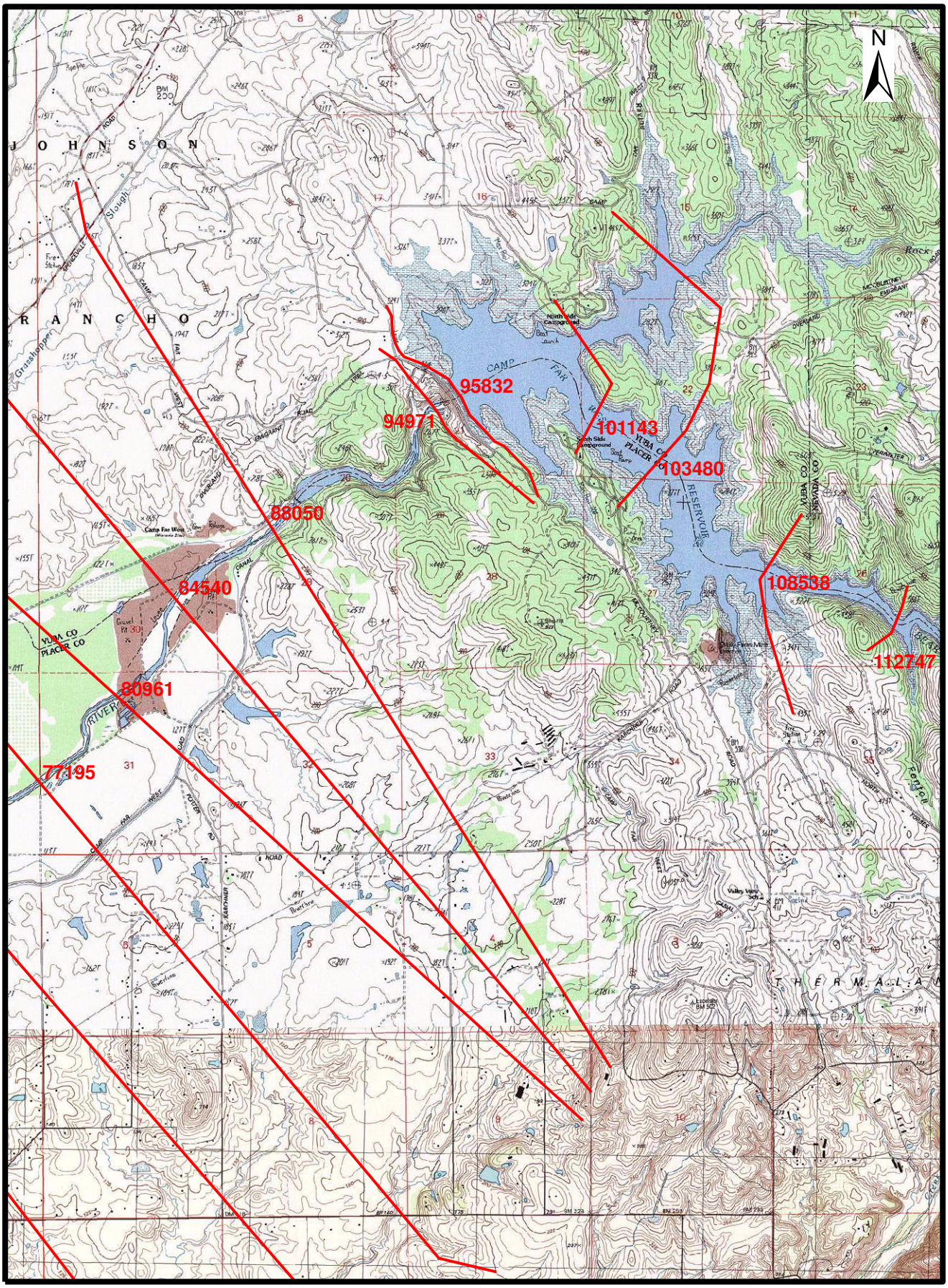
Appendix B. Project Plan Views

North Dike
North Wing Dam
Main Dam
South Wing Dam
Uncontrolled Spillway
Hokan Ln
Camp Far West Rd
Blackford Rd
© 2006 Europa Technologies
Image © 2006 DigitalGlobe
© 2006 Navteq
Google
Pointer 39°03'16.57" N 121°19'21.71" W elev 313 ft
Streaming |||||||||| 100%
Eye alt 8921 ft

Appendix C. Spillway Discharge Rating Curve

Appendix D. Index Maps

Camp Far West Dam IDF Study Map 1

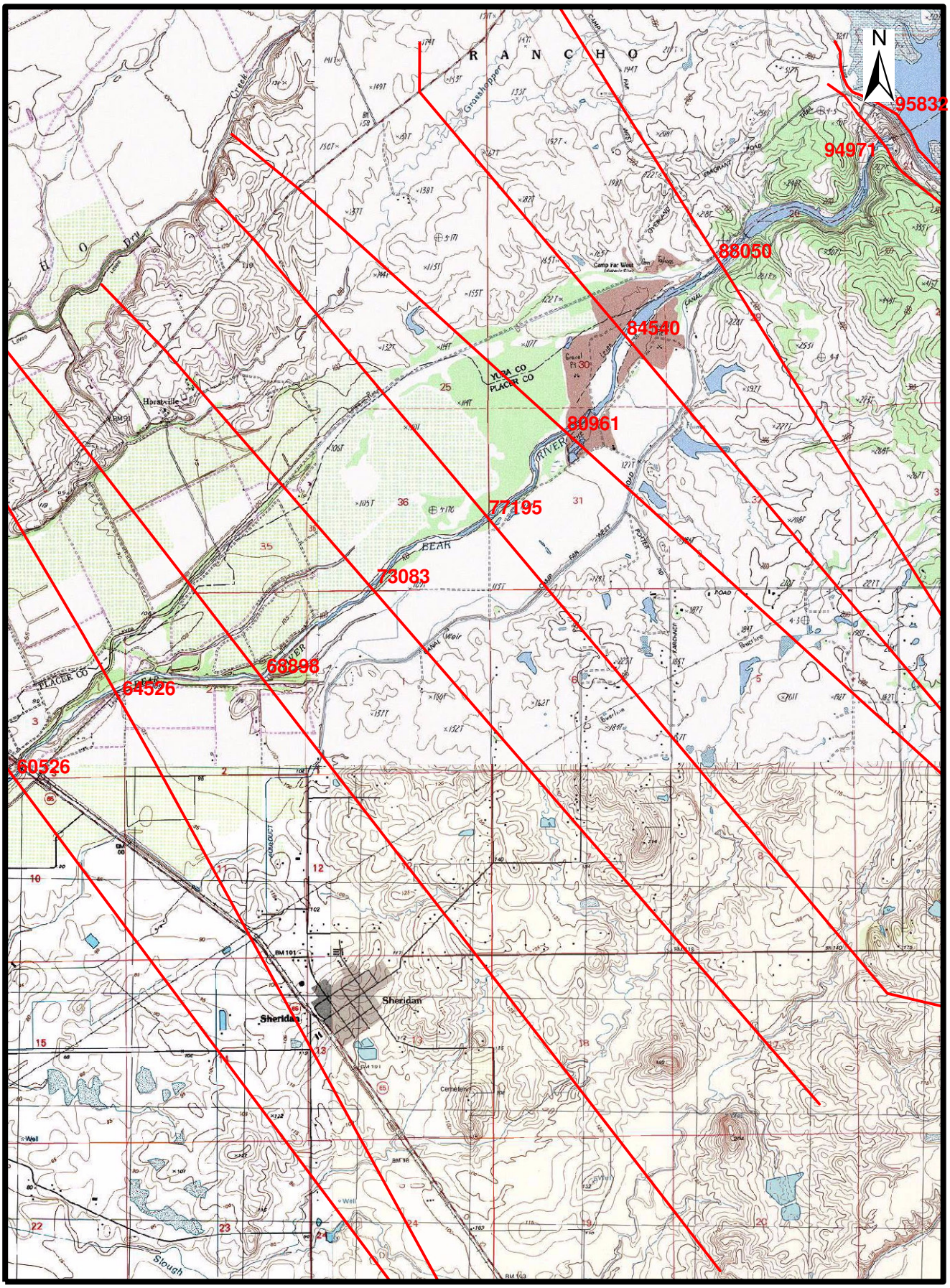


0 2,000 4,000 8,000 Feet

1 inch equals 4,000 feet

Revised by
Mead & Hunt, Inc.
August 2006

Camp Far West Dam IDF Study Map 2

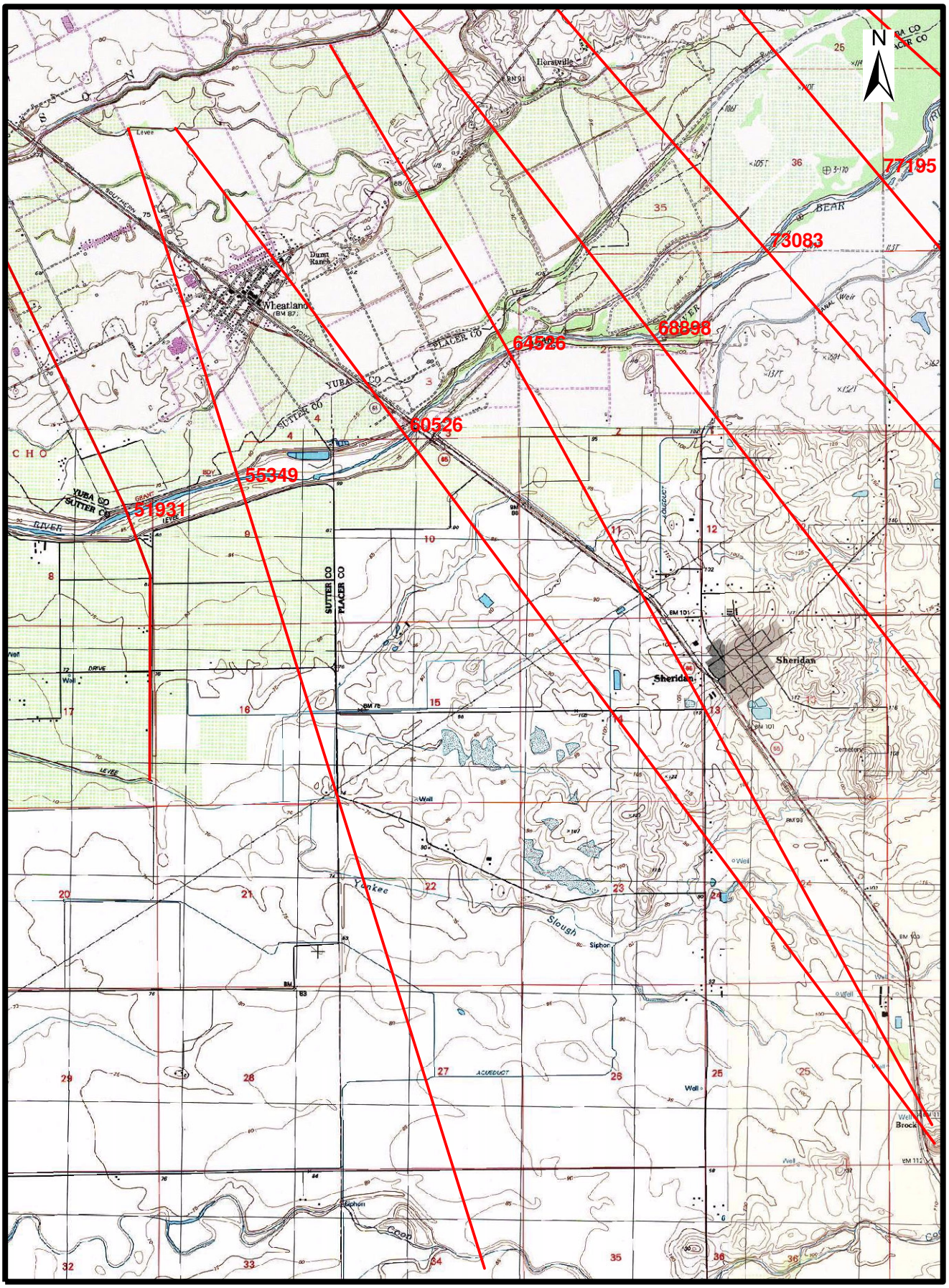


0 2,000 4,000 8,000 Feet

1 inch equals 4,000 feet

Revised by
Mead & Hunt, Inc.
August 2006

Camp Far West Dam IDF Study Map 3

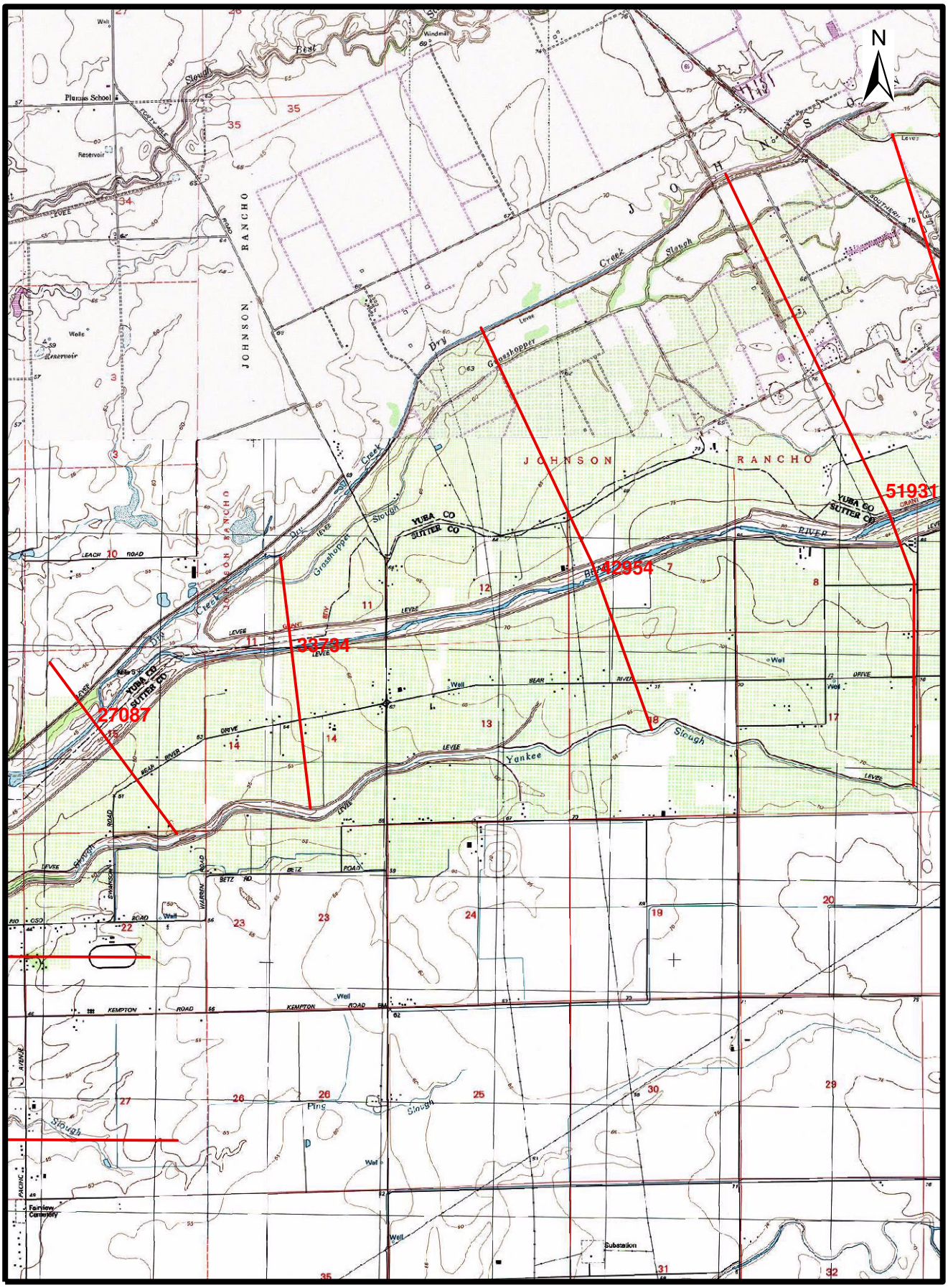


0 2,000 4,000 8,000 Feet

1 inch equals 4,000 feet

Revised by
Mead & Hunt, Inc.
August 2006

Camp Far West Dam IDF Study Map 4



0 2,000 4,000 8,000 Feet

1 inch equals 4,000 feet

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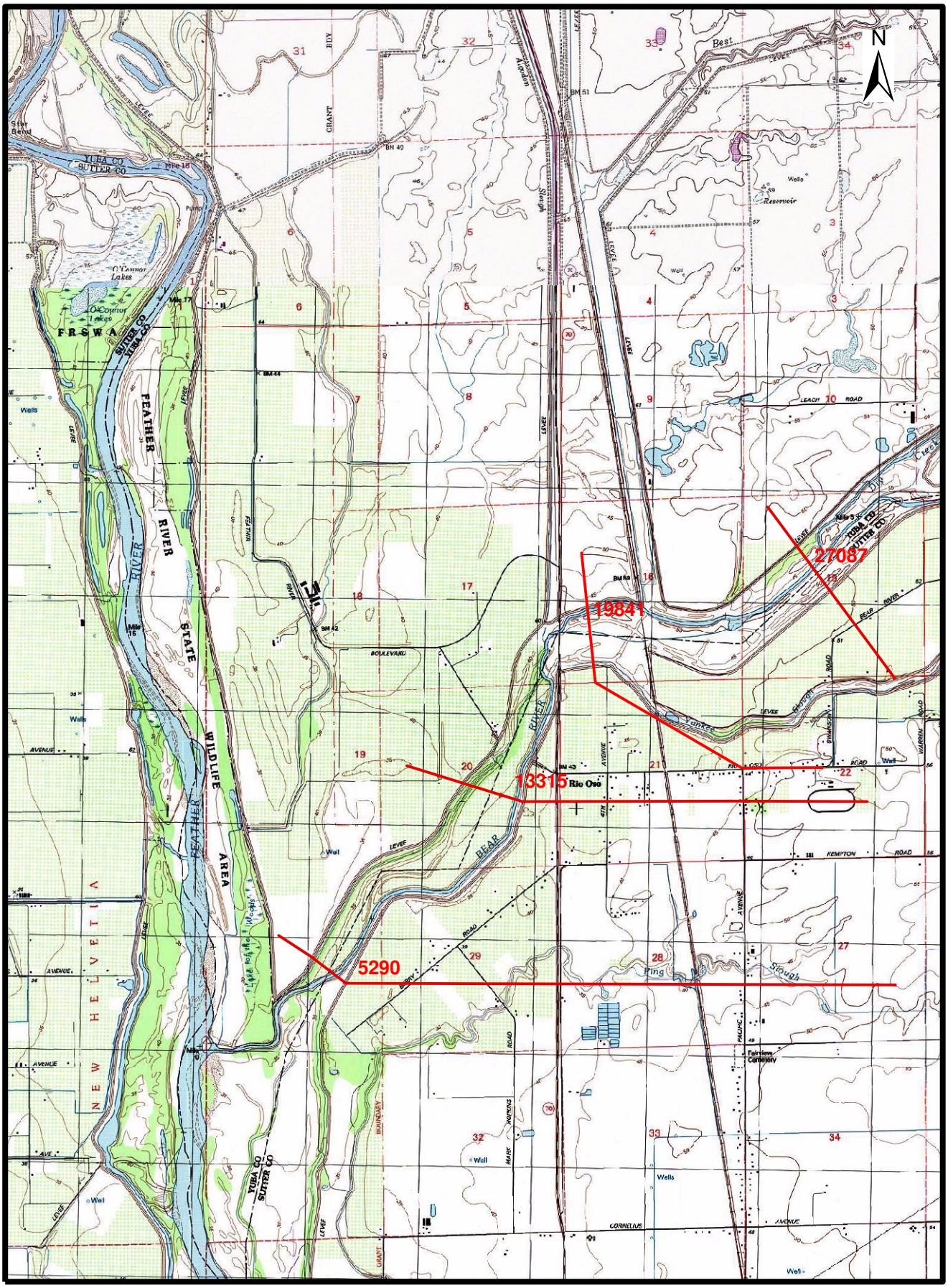
August 2006

Camp Far West Dam IDF Study Map 5



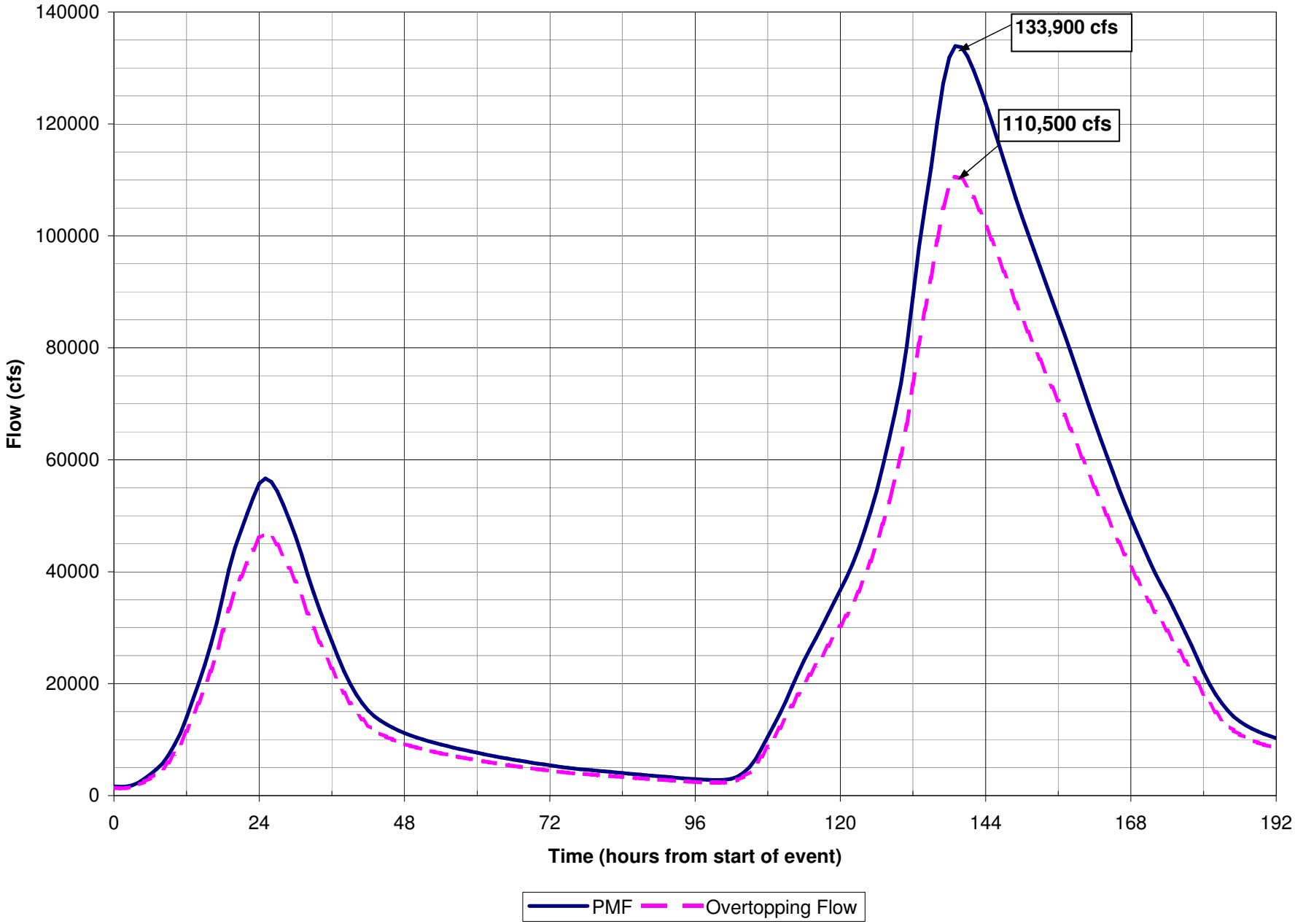
0 2,000 4,000 8,000 Feet

1 inch equals 4,000 feet

Revised by
Mead & Hunt, Inc.
August 2006

Appendix E. PMF and Overtopping Inflow Hydrographs

Appendix E. PMF and Overtopping Flow Hydrographs



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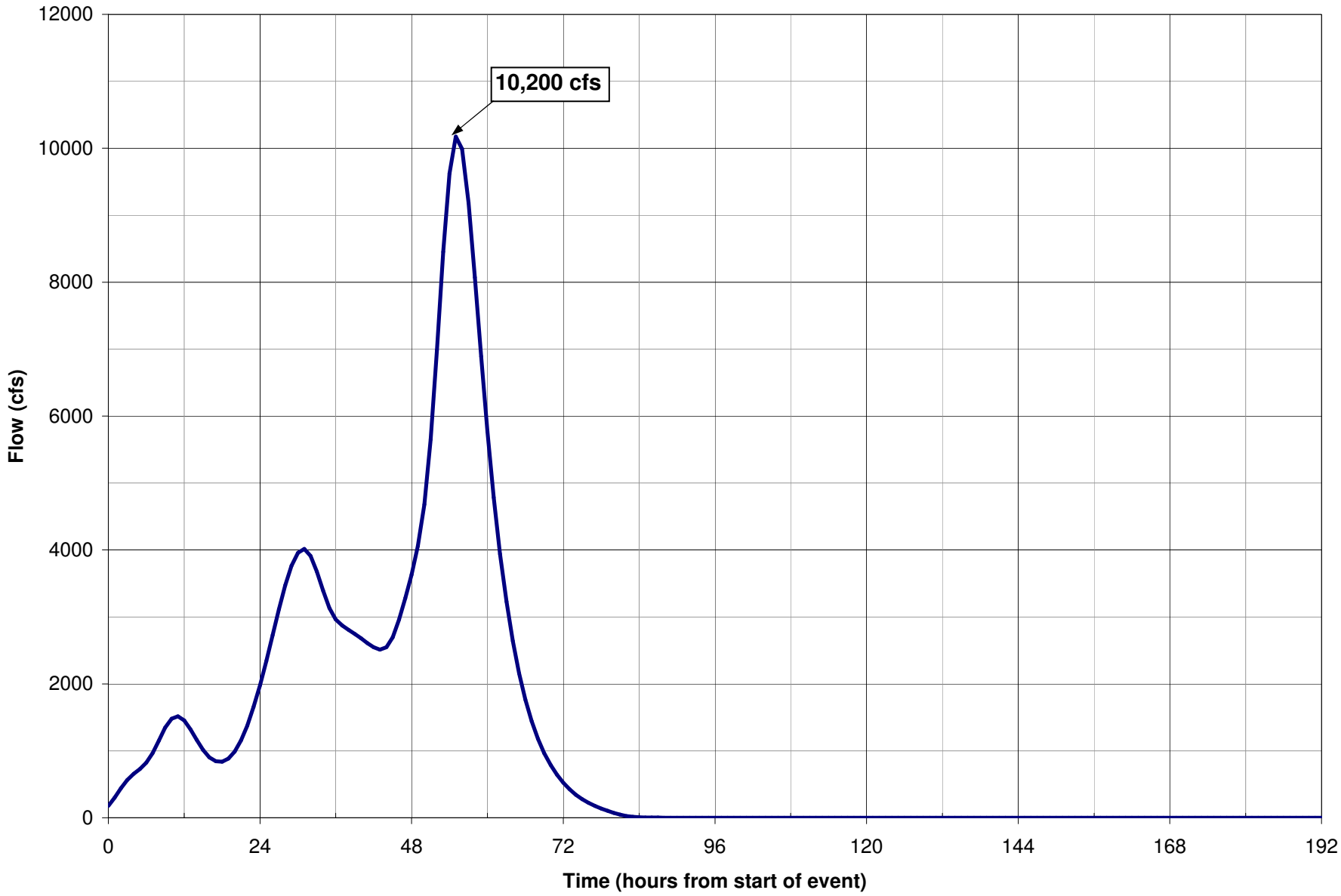
Appendix F. 100-Year Flood Hydrograph for Dry Creek

Appendix F. Dry Creek 100-Year Flood Hydrograph



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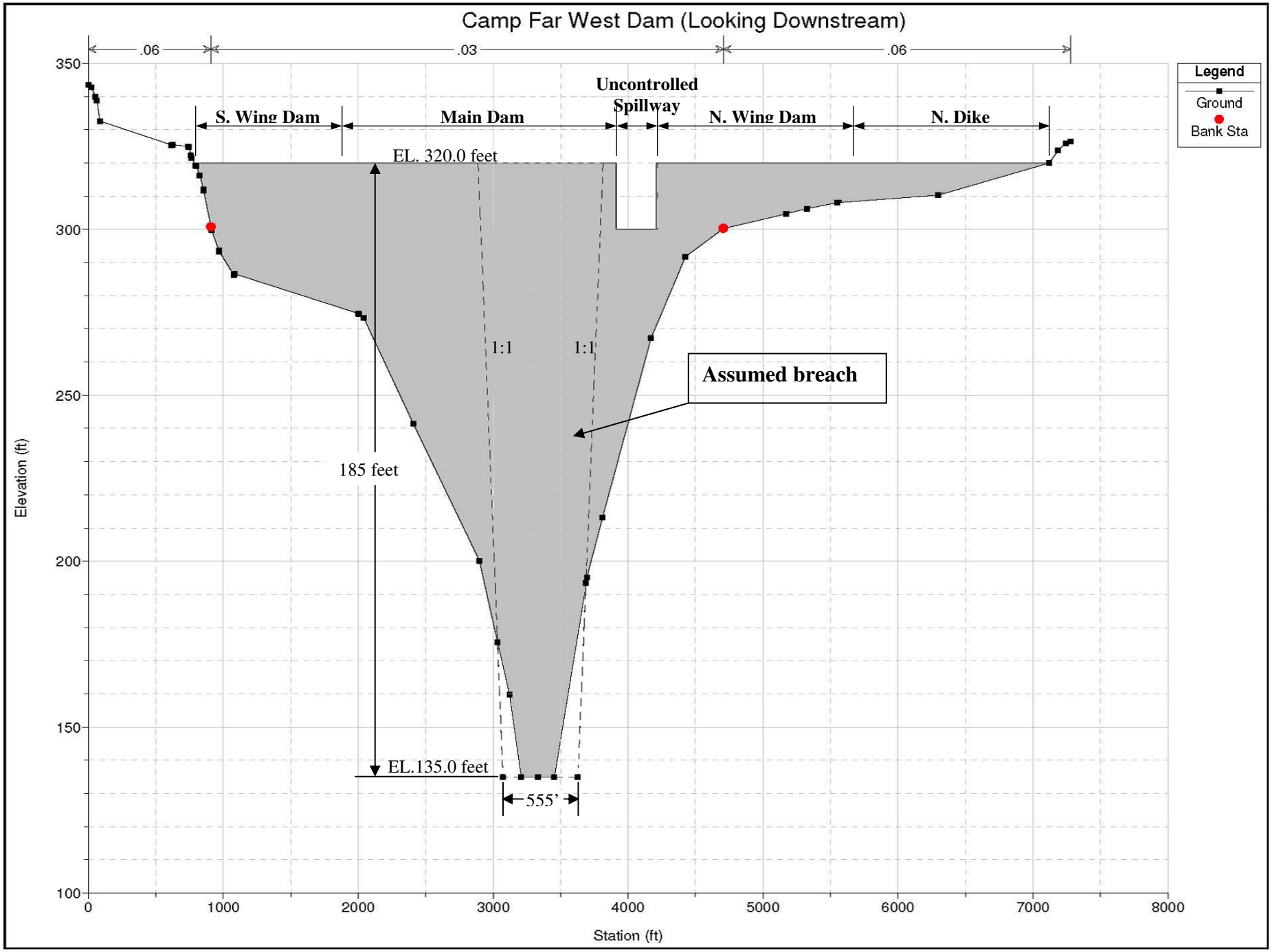
Appendix G. Downstream View of Dam Breach Section


Camp Far West Dam (Looking Downstream)
.06
.03
.06
Uncontrolled Spillway
S. Wing Dam
Main Dam
N. Wing Dam
N. Dike
Legend
Ground
Bank Sta
EL. 320.0 feet
1:1
1:1
Assumed breach
185 feet
EL.135.0 feet
555'
Elevation (ft)
Station (ft)
350
300
250
200
150
100
0
1000
2000
3000
4000
5000
6000
7000
8000

Appendix H. HEC-RAS Output Summary

HEC-RAS Output Summary

Plan "B-F-OTF": Dam Failure at Overtopping Flow
Plan "B-F-OTNF": Dam Non-failure at Overtopping Flow

HEC-RAS River: Bear River Reach: Reach 1 Profile: Max WS

Reach	River Sta	Profile	Plan	Q Total	W.S. Elev	Min Ch El	Diff	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)		(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	112747	Max WS	B-F-OTF	106215.30	319.98	170.00	149.98		320.02	0.000002	1.50	71592.21	772.11	0.03
Reach 1	112747	Max WS	B-F-OTNF	104930.90	320.00	170.00	150.00		320.04	0.000002	1.48	71607.71	772.23	0.03
Reach 1	110642.*	Max WS	B-F-OTF	106191.80	320.00	156.37	163.63		320.01	0.000000	0.69	156584.60	1723.01	0.01
Reach 1	110642.*	Max WS	B-F-OTNF	104922.80	320.02	156.37	163.65		320.03	0.000000	0.68	156619.90	1723.38	0.01
Reach 1	108538	Max WS	B-F-OTF	106067.80	320.01	154.00	166.01		320.01	0.000000	0.44	250876.10	3418.47	0.01
Reach 1	108538	Max WS	B-F-OTNF	104907.50	320.03	154.00	166.03		320.03	0.000000	0.43	250949.90	3418.88	0.01
Reach 1	106009.*	Max WS	B-F-OTF	105888.50	320.01	149.11	170.90		320.01	0.000000	0.21	521094.80	6404.27	0.00
Reach 1	106009.*	Max WS	B-F-OTNF	104880.80	320.03	149.11	170.92		320.03	0.000000	0.20	521240.20	6404.70	0.00
Reach 1	103480	Max WS	B-F-OTF	105501.50	320.01	144.22	175.79		320.01	0.000000	0.17	614840.80	9466.31	0.00
Reach 1	103480	Max WS	B-F-OTNF	104820.90	320.03	144.22	175.81		320.03	0.000000	0.17	615068.30	9466.82	0.00
Reach 1	101143	Max WS	B-F-OTF	105088.40	320.00	140.04	179.96		320.01	0.000000	0.23	468804.90	4663.90	0.00
Reach 1	101143	Max WS	B-F-OTNF	104793.80	320.03	140.04	179.99		320.03	0.000000	0.22	468920.90	4664.26	0.00
Reach 1	98487.5*	Max WS	B-F-OTF	104840.60	320.00	137.45	182.55		320.00	0.000000	0.24	439912.80	5070.49	0.00
Reach 1	98487.5*	Max WS	B-F-OTNF	104767.50	320.03	137.45	182.58		320.03	0.000000	0.24	440045.30	5071.10	0.00
Reach 1	95832	Max WS	B-F-OTF	104521.90	320.00	134.86	185.14	151.25	320.00	0.000000	0.34	329175.00	6335.47	0.01
Reach 1	95832	Max WS	B-F-OTNF	104738.00	320.03	134.86	185.17	151.23	320.03	0.000000	0.34	329350.00	6336.29	0.01
Reach 1	95700			Inl Struct										
Reach 1	94971	Max WS	B-F-OTF	1386085.00	250.70	134.83	115.87		259.01	0.000649	28.59	86110.75	1511.63	0.48
Reach 1	94971	Max WS	B-F-OTNF	104737.10	162.72	134.83	27.89		166.52	0.001711	16.14	7514.42	461.90	0.60
Reach 1	91510.5*	Max WS	B-F-OTF	2067966.00	254.57	126.42	128.15		256.75	0.000238	18.91	336421.00	12137.19	0.30
Reach 1	91510.5*	Max WS	B-F-OTNF	100329.00	148.96	126.42	22.54		153.15	0.002423	16.83	6654.44	454.99	0.69
Reach 1	88050	Max WS	B-F-OTF	2064863.00	161.36	118.00	43.36	189.75	316.04	0.035645	111.43	27204.68	2589.30	3.06
Reach 1	88050	Max WS	B-F-OTNF	95240.02	139.22	118.00	21.22		141.49	0.001296	12.70	8897.27	589.05	0.51
Reach 1	84540	Max WS	B-F-OTF	2022766.00	151.45	108.79	42.66	148.11	155.55	0.003899	35.67	145255.40	8191.93	0.98
Reach 1	84540	Max WS	B-F-OTNF	86849.30	130.59	108.79	21.80		131.66	0.001413	13.47	18644.28	3076.73	0.52
Reach 1	80961	Max WS	B-F-OTF	1986254.00	140.63	102.07	38.56		143.15	0.002856	29.37	173498.80	9357.11	0.84
Reach 1	80961	Max WS	B-F-OTNF	89438.91	122.96	102.07	20.89		123.69	0.001204	12.48	28116.81	7359.46	0.49
Reach 1	77195	Max WS	B-F-OTF	1882253.00	131.49	94.00	37.49		133.21	0.002279	25.75	205203.60	13348.54	0.75
Reach 1	77195	Max WS	B-F-OTNF	98560.12	114.57	94.00	20.57		115.84	0.001797	15.04	22041.00	6339.96	0.60
Reach 1	73083	Max WS	B-F-OTF	1805467.00	124.31	84.98	39.33		125.54	0.001502	21.04	223142.80	13489.05	0.61
Reach 1	73083	Max WS	B-F-OTNF	49142.16	110.04	84.98	25.06		110.05	0.000034	2.30	66819.01	9172.24	0.08

Note: Column "Q Total" indicates the flow at the peak stage, and does not indicate the peak flow at each station.

HEC-RAS Output Summary

Plan "B-F-OTF": Dam Failure at Overtopping Flow
Plan "B-F-OTNF": Dam Non-failure at Overtopping Flow

HEC-RAS River: Bear River Reach: Reach 1 Profile: Max WS (Continued)

Reach	River Sta	Profile	Plan	Q Total	W.S. Elev	Min Ch El	Diff	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)		(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	68898	Max WS	B-F-OTF	1746937.00	113.76	83.38	30.38		115.60	0.003246	25.19	177625.00	14362.57	0.84
Reach 1	68898	Max WS	B-F-OTNF	94404.29	109.00	83.38	25.62		109.76	0.000968	12.13	23814.21	3571.21	0.45
Reach 1	64526	Max WS	B-F-OTF	1555607.00	104.46	79.56	24.90		105.20	0.001435	14.70	266456.70	27874.67	0.55
Reach 1	64526	Max WS	B-F-OTNF	99956.49	107.94	79.56	28.38		109.65	0.000762	11.81	12182.92	732.84	0.41
Reach 1	60526	Max WS	B-F-OTF	1474819.00	96.88	73.44	23.44		97.72	0.002021	16.95	217878.60	22920.90	0.64
Reach 1	60526	Max WS	B-F-OTNF	294290.80	103.71	73.44	30.27	104.19	141.67	0.019344	62.85	8987.92	693.93	2.07
Reach 1	55349	Max WS	B-F-OTF	1120865.00	91.45	66.06	25.39		91.61	0.000443	8.01	352040.50	31124.86	0.29
Reach 1	55349	Max WS	B-F-OTNF	234467.50	98.20	66.06	32.14	99.00	108.17	0.006473	36.34	12252.61	693.92	1.17
Reach 1	51931	Max WS	B-F-OTF	1109046.00	89.36	63.48	25.88		89.72	0.000656	10.27	247332.70	18727.19	0.37
Reach 1	51931	Max WS	B-F-OTNF	140515.60	93.52	63.48	30.04		98.50	0.002595	22.74	11231.67	770.08	0.76
Reach 1	42954	Max WS	B-F-OTF	878709.30	83.32	54.47	28.85		83.72	0.000704	10.86	182385.80	12490.15	0.38
Reach 1	42954	Max WS	B-F-OTNF	106419.00	84.19	54.47	29.72		84.20	0.000009	1.23	193296.80	12490.15	0.04
Reach 1	33734	Max WS	B-F-OTF	834244.00	75.87	47.61	28.26		76.57	0.000889	12.82	132314.30	7236.37	0.44
Reach 1	33734	Max WS	B-F-OTNF	142202.30	73.37	47.61	25.76		78.01	0.002908	21.68	10522.74	627.56	0.79
Reach 1	27087	Max WS	B-F-OTF	549255.50	70.12	43.99	26.13		70.56	0.000427	8.39	116303.00	6186.77	0.31
Reach 1	27087	Max WS	B-F-OTNF	118840.30	61.87	43.99	17.88		62.53	0.000676	7.90	21316.13	1612.19	0.36
Reach 1	19841	Max WS	B-F-OTF	499439.80	68.67	35.00	33.67		68.78	0.000092	4.39	205673.70	8756.55	0.15
Reach 1	19841	Max WS	B-F-OTNF	92952.23	57.17	35.00	22.17		57.46	0.000318	5.73	25384.80	1804.28	0.25
Reach 1	13315	Max WS	B-F-OTF	495155.80	68.20	28.00	40.20		68.29	0.000068	4.29	233536.40	9943.58	0.13
Reach 1	13315	Max WS	B-F-OTNF	106007.80	55.10	28.00	27.10		55.36	0.000222	5.63	30906.90	1843.69	0.22
Reach 1	5290	Max WS	B-F-OTF	1688.25	52.78	16.00	36.78	18.68	52.78	0.000000	0.08	25760.01	1184.97	0.00
Reach 1	5290	Max WS	B-F-OTNF	1683.01	52.78	16.00	36.78	18.67	52.78	0.000000	0.08	25760.01	1184.97	0.00

Note: Column "Q Total" indicates the flow at the peak stage, and does not indicate the peak flow at each station.

HEC-RAS Output Summary

Plan "B-F-PMFF": Dam Failure at PMF
Plan "B-F-PMFNF": Dam Non-failure at PMF

HEC-RAS River: Bear River Reach: Reach 1 Profile: Max WS

Reach	River Sta	Profile	Plan	Q Total	W.S. Elev	Min Ch El	Diff	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)		(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	112747	Max WS	B-F-PMFF	133874.70	320.87	170.00	150.87		320.93	0.000003	1.87	72279.93	777.37	0.03
Reach 1	112747	Max WS	B-F-PMFNF	133244.20	320.90	170.00	150.90		320.96	0.000003	1.86	72303.80	777.55	0.03
Reach 1	110642.*	Max WS	B-F-PMFF	133852.90	320.90	156.37	164.53		320.91	0.000001	0.86	158140.90	1739.39	0.01
Reach 1	110642.*	Max WS	B-F-PMFNF	133532.80	320.94	156.37	164.57		320.95	0.000001	0.85	158195.90	1739.96	0.01
Reach 1	108538	Max WS	B-F-PMFF	133684.60	320.91	154.00	166.91		320.91	0.000000	0.55	253962.50	3435.81	0.01
Reach 1	108538	Max WS	B-F-PMFNF	133525.80	320.94	154.00	166.94		320.95	0.000000	0.55	254076.30	3436.45	0.01
Reach 1	106009.*	Max WS	B-F-PMFF	133454.30	320.91	149.11	171.80		320.91	0.000000	0.26	526879.60	6429.79	0.00
Reach 1	106009.*	Max WS	B-F-PMFNF	133492.60	320.94	149.11	171.83		320.94	0.000000	0.26	527100.90	6430.39	0.00
Reach 1	103480	Max WS	B-F-PMFF	133072.80	320.91	144.22	176.69		320.91	0.000000	0.21	623382.30	9485.20	0.00
Reach 1	103480	Max WS	B-F-PMFNF	133424.00	320.94	144.22	176.72		320.94	0.000000	0.21	623721.00	9485.94	0.00
Reach 1	101143	Max WS	B-F-PMFF	132556.00	320.91	140.04	180.87		320.91	0.000000	0.28	473011.90	4677.09	0.00
Reach 1	101143	Max WS	B-F-PMFNF	133418.80	320.94	140.04	180.90		320.94	0.000000	0.28	473184.60	4677.81	0.00
Reach 1	98487.5*	Max WS	B-F-PMFF	132199.90	320.90	137.45	183.45		320.91	0.000000	0.30	444488.40	5090.36	0.01
Reach 1	98487.5*	Max WS	B-F-PMFNF	133397.90	320.94	137.45	183.49		320.94	0.000000	0.31	444683.40	5091.19	0.01
Reach 1	95832	Max WS	B-F-PMFF	131843.80	320.90	134.86	186.04	153.72	320.90	0.000000	0.43	334887.80	6362.41	0.01
Reach 1	95832	Max WS	B-F-PMFNF	133386.00	320.94	134.86	186.08	153.84	320.94	0.000000	0.43	335141.80	6363.60	0.01
Reach 1	95700			Inl Struct										
Reach 1	94971	Max WS	B-F-PMFF	1441613.00	251.60	134.83	116.77		260.33	0.000677	29.35	87475.64	1524.81	0.49
Reach 1	94971	Max WS	B-F-PMFNF	129531.30	166.22	134.83	31.39		170.25	0.001534	16.82	9162.62	479.82	0.58
Reach 1	91510.5*	Max WS	B-F-PMFF	2120991.00	255.53	126.42	129.11		257.63	0.000232	18.77	348087.00	12266.21	0.30
Reach 1	91510.5*	Max WS	B-F-PMFNF	111419.40	151.57	126.42	25.15		155.37	0.001859	16.12	7876.76	480.65	0.62
Reach 1	88050	Max WS	B-F-PMFF	2091609.00	161.67	118.00	43.67	190.06	315.69	0.035331	111.50	28009.28	2625.33	3.05
Reach 1	88050	Max WS	B-F-PMFNF	114017.80	141.53	118.00	23.53		144.00	0.001230	13.36	10279.69	610.87	0.51
Reach 1	84540	Max WS	B-F-PMFF	2071905.00	151.73	108.79	42.94	148.39	155.88	0.003905	35.86	147566.00	8206.59	0.98
Reach 1	84540	Max WS	B-F-PMFNF	110173.00	131.52	108.79	22.73		132.72	0.001609	14.80	21607.78	3296.54	0.56
Reach 1	80961	Max WS	B-F-PMFF	2039671.00	140.84	102.07	38.77		143.44	0.002920	29.81	175485.50	9404.14	0.85
Reach 1	80961	Max WS	B-F-PMFNF	96389.56	123.92	102.07	21.85		124.34	0.000771	10.31	35255.59	7541.55	0.40
Reach 1	77195	Max WS	B-F-PMFF	1937098.00	131.76	94.00	37.76		133.51	0.002287	25.93	208842.60	13380.32	0.76
Reach 1	77195	Max WS	B-F-PMFNF	121403.60	115.39	94.00	21.39		116.57	0.001787	15.42	27476.18	6919.79	0.61
Reach 1	73083	Max WS	B-F-PMFF	1842166.00	124.57	84.98	39.59		125.81	0.001499	21.11	226729.10	13721.09	0.61
Reach 1	73083	Max WS	B-F-PMFNF	34416.85	110.82	84.98	25.84		110.82	0.000014	1.53	74230.38	9824.82	0.06
Reach 1	68898	Max WS	B-F-PMFF	1795608.00	113.95	83.38	30.57		115.84	0.003277	25.43	180379.10	14417.17	0.85
Reach 1	68898	Max WS	B-F-PMFNF	-135883.00	109.28	83.38	25.90		109.31	0.000068	-3.25	116460.00	12763.47	0.12

Note: Column "Q Total" indicates the flow at the peak stage, and does not indicate the peak flow at each station.

HEC-RAS Output Summary

Plan "B-F-PMFF": Dam Failure at PMF
Plan "B-F-PMFNF": Dam Non-failure at PMF

HEC-RAS River: Bear River Reach: Reach 1 Profile: Max WS (Continued)

Reach	River Sta	Profile	Plan	Q Total	W.S. Elev	Min Ch El	Diff	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)		(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	64526	Max WS	B-F-PMFF	1621792.00	104.66	79.56	25.10		105.44	0.001481	15.02	272203.20	28272.16	0.56
Reach 1	64526	Max WS	B-F-PMFNF	111755.10	108.39	79.56	28.83		110.41	0.000886	12.88	12510.13	732.84	0.45
Reach 1	60526	Max WS	B-F-PMFF	1531968.00	97.09	73.44	23.65		97.96	0.002049	17.18	222705.80	23044.93	0.64
Reach 1	60526	Max WS	B-F-PMFNF	538760.70	104.03	73.44	30.59	104.19	224.22	0.060916	112.35	9209.01	693.93	3.67
Reach 1	55349	Max WS	B-F-PMFF	1177450.00	91.73	66.06	25.67		91.90	0.000451	8.14	360756.80	31124.86	0.30
Reach 1	55349	Max WS	B-F-PMFNF	372823.30	98.35	66.06	32.29	99.00	123.06	0.015957	57.25	12357.54	693.92	1.85
Reach 1	51931	Max WS	B-F-PMFF	1021073.00	89.72	63.48	26.24		90.01	0.000511	9.16	254070.60	18727.19	0.33
Reach 1	51931	Max WS	B-F-PMFNF	363428.10	93.66	63.48	30.18	93.91	126.27	0.016937	58.28	11335.53	770.08	1.93
Reach 1	42954	Max WS	B-F-PMFF	907789.30	83.76	54.47	29.29		84.16	0.000682	10.82	187856.10	12490.15	0.38
Reach 1	42954	Max WS	B-F-PMFNF	374971.30	86.68	54.47	32.21		86.73	0.000065	3.59	224346.60	12490.15	0.12
Reach 1	33734	Max WS	B-F-PMFF	870762.80	76.34	47.61	28.73		77.06	0.000892	12.99	135696.00	7236.37	0.44
Reach 1	33734	Max WS	B-F-PMFNF	677966.60	74.45	47.61	26.84		75.51	0.001373	15.35	89532.03	5193.20	0.55
Reach 1	27087	Max WS	B-F-PMFF	556922.10	70.90	43.99	26.91		71.32	0.000384	8.13	121175.70	6186.77	0.29
Reach 1	27087	Max WS	B-F-PMFNF	553373.80	67.65	43.99	23.66		68.26	0.000692	9.91	101057.10	6186.77	0.38
Reach 1	19841	Max WS	B-F-PMFF	514252.60	69.58	35.00	34.58		69.69	0.000086	4.33	213646.10	8756.55	0.14
Reach 1	19841	Max WS	B-F-PMFNF	468864.60	62.91	35.00	27.91		63.08	0.000207	5.64	155247.60	8756.55	0.21
Reach 1	13315	Max WS	B-F-PMFF	511800.80	69.14	28.00	41.14		69.23	0.000064	4.24	242887.00	9943.58	0.13
Reach 1	13315	Max WS	B-F-PMFNF	339182.20	61.38	28.00	33.38		61.76	0.000297	7.74	77780.36	3524.80	0.26
Reach 1	5290	Max WS	B-F-PMFF	1687.63	52.78	16.00	36.78	18.68	52.78	0.000000	0.08	25760.01	1184.97	0.00
Reach 1	5290	Max WS	B-F-PMFNF	1682.21	52.78	16.00	36.78	18.67	52.78	0.000000	0.08	25760.01	1184.97	0.00

Note: Column "Q Total" indicates the flow at the peak stage, and does not indicate the peak flow at each station.

HEC-RAS Output Summary - Spillway Modification Alternative 1: Raising the Dam

Dam Non-failure at PMF

HEC-RAS Plan: HighDamPMFN River: Bear River Reach: Reach 1 Profile: Max WS

Reach	River Sta	Profile	Q Total	W.S. Elev	Min Ch El	Diff	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)		(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	112747	Max WS	127292.50	322.76	170.00	152.76		322.81	0.000003	1.75	73762.05	789.01	0.03
Reach 1	110642.*	Max WS	127657.50	322.79	156.37	166.42		322.80	0.000001	0.80	161461.10	1773.82	0.01
Reach 1	108538	Max WS	127691.10	322.80	154.00	168.80		322.80	0.000000	0.51	260493.90	3471.77	0.01
Reach 1	106009.*	Max WS	127590.10	322.80	149.11	173.69		322.80	0.000000	0.24	539076.30	6462.01	0.00
Reach 1	103480	Max WS	127600.90	322.80	144.22	178.58		322.80	0.000000	0.20	641378.80	9524.49	0.00
Reach 1	101143	Max WS	127564.00	322.80	140.04	182.76		322.80	0.000000	0.27	481905.10	4705.60	0.00
Reach 1	98487.5*	Max WS	127532.60	322.80	137.45	185.35		322.80	0.000000	0.29	454179.30	5131.51	0.00
Reach 1	95832	Max WS	127536.80	322.80	134.86	187.94	153.31	322.80	0.000000	0.40	347013.10	6411.83	0.01
Reach 1	95700		Inl Struct										
Reach 1	94971	Max WS	108322.20	166.05	134.83	31.22		168.92	0.001098	14.17	9083.90	478.98	0.49
Reach 1	91510.5*	Max WS	115429.30	152.29	126.42	25.87		156.05	0.001768	16.08	8225.61	487.72	0.61
Reach 1	88050	Max WS	66666.14	141.78	118.00	23.78		142.60	0.000403	7.71	10431.17	613.21	0.29
Reach 1	84540	Max WS	83538.32	131.56	108.79	22.77		132.24	0.000912	11.16	21735.53	3305.85	0.42
Reach 1	80961	Max WS	98022.33	123.82	102.07	21.75		124.29	0.000844	10.75	34516.20	7517.77	0.42
Reach 1	77195	Max WS	103528.70	115.44	94.00	21.44		116.27	0.001268	13.01	27816.76	6954.51	0.51
Reach 1	73083	Max WS	30350.18	110.84	84.98	25.86		110.84	0.000011	1.35	74415.08	9826.24	0.05
Reach 1	68898	Max WS	-135492.00	109.28	83.38	25.90		109.31	0.000068	-3.24	116443.40	12762.63	0.12
Reach 1	64526	Max WS	114549.60	108.36	79.56	28.80		110.48	0.000935	13.23	12486.17	732.84	0.46
Reach 1	60526	Max WS	527740.60	104.02	73.44	30.58	104.19	219.58	0.058580	110.15	9201.03	693.93	3.60
Reach 1	55349	Max WS	539012.70	98.34	66.06	32.28	99.01	150.02	0.033377	82.79	12354.61	693.92	2.67
Reach 1	51931	Max WS	538653.90	93.79	63.48	30.31	93.91	164.00	0.036336	85.63	11435.85	770.08	2.83
Reach 1	42954	Max WS	628589.80	88.12	54.47	33.65		88.23	0.000142	5.49	242290.50	12490.15	0.18
Reach 1	33734	Max WS	697517.40	74.66	47.61	27.05		75.23	0.000787	11.69	123547.70	7236.37	0.41
Reach 1	27087	Max WS	607446.30	67.73	43.99	23.74		68.46	0.000821	10.82	101567.90	6186.77	0.42
Reach 1	19841	Max WS	615741.00	64.89	35.00	29.89		65.12	0.000253	6.59	172512.80	8756.55	0.24
Reach 1	13315	Max WS	377629.40	62.20	28.00	34.20		62.64	0.000326	8.28	80692.37	3524.80	0.27
Reach 1	5290	Max WS	1682.21	52.78	16.00	36.78	18.67	52.78	0.000000	0.08	25760.01	1184.97	0.00

Note: Column "Q Total" indicates the flow at the peak stage, and does not indicate the peak flow at each station.

HEC-RAS Output Summary - Spillway Modification Alternative 2: Lowering N. Dike

Plan "LowNDikeOTF": N. Dike Failure at Overtopping Flow
Plan "LowNDikeOTNF": N. Dike Non-failure at Overtopping Flow

HEC-RAS River: Bear River Reach: Reach 1 Profile: Max WS

Reach	River Sta	Profile	Plan	Q Total	W.S. Elev	Min Ch El	Diff	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)		(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	112747	Max WS	LowNDikeOTF	70297.52	315.95	170.00	145.95		315.97	0.000001	1.03	68527.67	748.25	0.02
Reach 1	112747	Max WS	LowNDikeOTNF	69187.34	315.97	170.00	145.97		315.98	0.000001	1.02	68539.05	748.34	0.02
Reach 1	110642.*	Max WS	LowNDikeOTF	70286.99	315.96	156.37	159.59		315.97	0.000000	0.47	149747.50	1664.87	0.01
Reach 1	110642.*	Max WS	LowNDikeOTNF	69190.32	315.98	156.37	159.61		315.98	0.000000	0.47	149772.30	1665.06	0.01
Reach 1	108538	Max WS	LowNDikeOTF	70274.89	315.96	154.00	161.96		315.97	0.000000	0.30	237551.00	3143.78	0.01
Reach 1	108538	Max WS	LowNDikeOTNF	69262.26	315.98	154.00	161.98		315.98	0.000000	0.30	237597.90	3144.37	0.01
Reach 1	106009.*	Max WS	LowNDikeOTF	70246.56	315.97	149.11	166.86		315.97	0.000000	0.14	495516.20	6254.36	0.00
Reach 1	106009.*	Max WS	LowNDikeOTNF	69445.34	315.98	149.11	166.87		315.98	0.000000	0.14	495610.10	6254.81	0.00
Reach 1	103480	Max WS	LowNDikeOTF	70199.88	315.97	144.22	171.75		315.97	0.000000	0.12	576762.90	9379.78	0.00
Reach 1	103480	Max WS	LowNDikeOTNF	69378.01	315.98	144.22	171.76		315.98	0.000000	0.12	576904.70	9380.15	0.00
Reach 1	101143	Max WS	LowNDikeOTF	70166.54	315.97	140.04	175.93		315.97	0.000000	0.16	450094.70	4588.10	0.00
Reach 1	101143	Max WS	LowNDikeOTNF	69444.44	315.98	140.04	175.94		315.98	0.000000	0.15	450165.10	4588.49	0.00
Reach 1	98487.5*	Max WS	LowNDikeOTF	70142.07	315.96	137.45	178.51		315.97	0.000000	0.17	419685.70	4931.47	0.00
Reach 1	98487.5*	Max WS	LowNDikeOTNF	69413.51	315.98	137.45	178.53		315.98	0.000000	0.17	419763.10	4932.10	0.00
Reach 1	95832	Max WS	LowNDikeOTF	69998.87	315.96	134.86	181.10	147.67	315.96	0.000000	0.24	304362.00	5955.00	0.00
Reach 1	95832	Max WS	LowNDikeOTNF	69377.27	315.98	134.86	181.12	147.61	315.98	0.000000	0.24	304458.50	5956.48	0.00
Reach 1	95700			Inl Struct										
Reach 1	94971	Max WS	LowNDikeOTF	72208.88	156.38	134.83	21.55		160.42	0.002677	16.23	4748.88	406.25	0.71
Reach 1	94971	Max WS	LowNDikeOTNF	69377.38	156.05	134.83	21.22		159.95	0.002659	15.96	4613.74	400.65	0.70
Reach 1	91510.5*	Max WS	LowNDikeOTF	72203.36	143.50	126.42	17.08		148.33	0.004372	17.83	4306.34	402.36	0.88
Reach 1	91510.5*	Max WS	LowNDikeOTNF	69377.45	143.22	126.42	16.80		147.90	0.004357	17.54	4194.80	399.53	0.87
Reach 1	88050	Max WS	LowNDikeOTF	72020.52	134.58	118.00	16.58		137.09	0.002020	13.16	6268.25	540.87	0.61
Reach 1	88050	Max WS	LowNDikeOTNF	69375.43	134.35	118.00	16.35		136.76	0.001988	12.91	6139.93	537.47	0.61
Reach 1	84540	Max WS	LowNDikeOTF	72032.56	129.72	108.79	20.93		130.79	0.001377	12.92	16046.43	2882.81	0.51
Reach 1	84540	Max WS	LowNDikeOTNF	69341.04	129.61	108.79	20.82		130.64	0.001336	12.67	15728.18	2856.58	0.50
Reach 1	80961	Max WS	LowNDikeOTF	72279.23	121.20	102.07	19.13		123.77	0.003062	18.71	15405.21	7088.84	0.77
Reach 1	80961	Max WS	LowNDikeOTNF	69429.54	121.09	102.07	19.02		123.78	0.003124	18.82	14590.27	7071.14	0.78
Reach 1	77195	Max WS	LowNDikeOTF	67490.73	112.75	94.00	18.75		113.80	0.001480	12.78	14349.61	2560.37	0.54
Reach 1	77195	Max WS	LowNDikeOTNF	67490.73	112.75	94.00	18.75		113.80	0.001480	12.78	14349.61	2560.37	0.54
Reach 1	73083	Max WS	LowNDikeOTF	66973.20	109.89	84.98	24.91		109.92	0.000066	3.18	65478.61	8744.16	0.12
Reach 1	73083	Max WS	LowNDikeOTNF	66973.20	109.89	84.98	24.91		109.92	0.000066	3.18	65478.61	8744.16	0.12

Note: Column "Q Total" indicates the flow at the peak stage, and does not indicate the peak flow at each station.

HEC-RAS Output Summary - Spillway Modification Alternative 2: Lowering N. Dike

Plan "LowNDikeOTF": N. Dike Failure at Overtopping Flow
Plan "LowNDikeOTNF": N. Dike Non-failure at Overtopping Flow

HEC-RAS River: Bear River Reach: Reach 1 Profile: Max WS (Continued)

Reach	River Sta	Profile	Plan	Q Total	W.S. Elev	Min Ch El	Diff	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)		(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	68898	Max WS	LowNDikeOTF	68792.13	108.58	83.38	25.20		109.07	0.000617	9.57	22309.30	3571.21	0.36
Reach 1	68898	Max WS	LowNDikeOTNF	68095.80	108.54	83.38	25.16		109.03	0.000615	9.54	22170.75	3571.21	0.36
Reach 1	64526	Max WS	LowNDikeOTF	68626.55	106.87	79.56	27.31		107.79	0.000430	8.63	11397.17	732.84	0.31
Reach 1	64526	Max WS	LowNDikeOTNF	67929.20	106.75	79.56	27.19		107.67	0.000430	8.60	11310.76	732.84	0.31
Reach 1	60526	Max WS	LowNDikeOTF	68625.47	101.08	73.44	27.64		104.41	0.001782	17.89	7175.41	650.04	0.62
Reach 1	60526	Max WS	LowNDikeOTNF	67928.16	100.98	73.44	27.54		104.29	0.001775	17.81	7111.25	642.27	0.62
Reach 1	55349	Max WS	LowNDikeOTF	66258.08	95.33	66.06	29.27		96.55	0.000869	12.45	10264.66	693.92	0.42
Reach 1	55349	Max WS	LowNDikeOTNF	66280.12	95.34	66.06	29.28		96.56	0.000869	12.45	10269.07	693.92	0.42
Reach 1	51931	Max WS	LowNDikeOTF	66113.10	92.62	63.48	29.14		93.90	0.000680	11.39	10537.46	770.08	0.39
Reach 1	51931	Max WS	LowNDikeOTNF	66093.53	92.63	63.48	29.15		93.90	0.000679	11.38	10545.21	770.08	0.38
Reach 1	42954	Max WS	LowNDikeOTF	67466.75	83.97	54.47	29.50		86.30	0.001269	14.84	7732.25	553.69	0.51
Reach 1	42954	Max WS	LowNDikeOTNF	67420.04	83.97	54.47	29.50		86.29	0.001267	14.83	7732.21	553.69	0.51
Reach 1	33734	Max WS	LowNDikeOTF	66860.65	67.03	47.61	19.42		69.96	0.002539	16.43	6542.19	627.56	0.70
Reach 1	33734	Max WS	LowNDikeOTNF	66816.08	67.02	47.61	19.41		69.95	0.002539	16.42	6539.66	627.56	0.70
Reach 1	27087	Max WS	LowNDikeOTF	66832.27	58.00	43.99	14.01		58.42	0.000632	6.28	15069.27	1612.19	0.33
Reach 1	27087	Max WS	LowNDikeOTNF	66801.50	58.00	43.99	14.01		58.42	0.000631	6.28	15066.20	1612.19	0.33
Reach 1	19841	Max WS	LowNDikeOTF	66776.29	54.81	35.00	19.81		55.02	0.000297	5.00	21115.33	1804.28	0.23
Reach 1	19841	Max WS	LowNDikeOTNF	66757.31	54.80	35.00	19.80		55.02	0.000297	5.00	21113.53	1804.28	0.23
Reach 1	13315	Max WS	LowNDikeOTF	66736.43	53.51	28.00	25.51		53.64	0.000121	3.94	27988.15	1843.69	0.16
Reach 1	13315	Max WS	LowNDikeOTNF	66721.34	53.51	28.00	25.51		53.64	0.000121	3.94	27987.44	1843.69	0.16
Reach 1	5290	Max WS	LowNDikeOTF	1677.24	52.78	16.00	36.78	18.67	52.78	0.000000	0.08	25760.01	1184.97	0.00
Reach 1	5290	Max WS	LowNDikeOTNF	1677.24	52.78	16.00	36.78	18.67	52.78	0.000000	0.08	25760.01	1184.97	0.00

Note: Column "Q Total" indicates the flow at the peak stage, and does not indicate the peak flow at each station.

HEC-RAS Output Summary - Spillway Modification Alternative 4-1: Lower Ogee-shaped Spillway
Dam Non-failure at PMF

HEC-RAS Plan: DeepOgeePMFN River: Bear River Reach: Reach 1 Profile: Max WS

Reach	River Sta	Profile	Q Total	W.S. Elev	Min Ch El	Diff	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)		(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	112747	Max WS	127407.90	319.92	170.00	149.92		319.97	0.000003	1.80	71541.55	771.72	0.03
Reach 1	110642.*	Max WS	127707.30	319.95	156.37	163.58		319.96	0.000001	0.82	156489.00	1722.00	0.01
Reach 1	108538	Max WS	127657.10	319.95	154.00	165.95		319.96	0.000000	0.53	250696.30	3417.46	0.01
Reach 1	106009.*	Max WS	127643.60	319.96	149.11	170.85		319.96	0.000000	0.25	520770.80	6403.32	0.00
Reach 1	103480	Max WS	127577.00	319.96	144.22	175.74		319.96	0.000000	0.21	614374.30	9465.26	0.00
Reach 1	101143	Max WS	127547.40	319.96	140.04	179.92		319.96	0.000000	0.27	468578.20	4663.21	0.00
Reach 1	98487.5*	Max WS	127540.40	319.96	137.45	182.51		319.96	0.000000	0.30	439672.70	5069.36	0.01
Reach 1	95832	Max WS	127523.80	319.95	134.86	185.09	153.31	319.96	0.000000	0.42	328880.60	6334.07	0.01
Reach 1	95700		Inl Struct										
Reach 1	94971	Max WS	124837.20	166.01	134.83	31.18		169.84	0.001467	16.36	9065.75	478.79	0.57
Reach 1	91510.5*	Max WS	109719.50	152.30	126.42	25.88		155.69	0.001597	15.28	8227.27	487.75	0.58
Reach 1	88050	Max WS	111810.40	141.45	118.00	23.45		143.85	0.001198	13.15	10233.60	610.15	0.50
Reach 1	84540	Max WS	103689.40	131.42	108.79	22.63		132.52	0.001481	14.15	21260.04	3271.04	0.54
Reach 1	80961	Max WS	92158.49	123.79	102.07	21.72		124.21	0.000762	10.20	34238.99	7508.83	0.40
Reach 1	77195	Max WS	113065.00	115.31	94.00	21.31		116.38	0.001617	14.63	26898.77	6860.52	0.58
Reach 1	73083	Max WS	64527.97	110.56	84.98	25.58		110.58	0.000050	2.82	71700.45	9614.47	0.10
Reach 1	68898	Max WS	-117644.00	109.21	83.38	25.83		109.23	0.000053	-2.84	115589.80	12719.14	0.10
Reach 1	64526	Max WS	92386.36	108.30	79.56	28.74		109.70	0.000614	10.70	12446.45	732.84	0.37
Reach 1	60526	Max WS	458216.10	103.95	73.44	30.51	104.19	192.15	0.044768	96.14	9152.38	693.93	3.15
Reach 1	55349	Max WS	189120.10	98.31	66.06	32.25		104.70	0.004135	29.12	12327.88	693.92	0.94
Reach 1	51931	Max WS	371294.40	93.79	63.48	30.31	93.91	127.12	0.017241	59.00	11441.57	770.08	1.95
Reach 1	42954	Max WS	592006.00	87.71	54.47	33.24		87.81	0.000135	5.30	237191.80	12490.15	0.17
Reach 1	33734	Max WS	738679.80	74.90	47.61	27.29		75.52	0.000843	12.17	125318.90	7236.37	0.43
Reach 1	27087	Max WS	800410.80	68.70	43.99	24.71		69.82	0.001184	13.39	107543.40	6186.77	0.50
Reach 1	19841	Max WS	683330.30	64.68	35.00	29.68		64.97	0.000322	7.40	170679.10	8756.55	0.27
Reach 1	13315	Max WS	376942.90	62.20	28.00	34.20		62.64	0.000325	8.26	80683.20	3524.80	0.27
Reach 1	5290	Max WS	1683.14	52.78	16.00	36.78	18.67	52.78	0.000000	0.08	25760.01	1184.97	0.00

Note: Column "Q Total" indicates the flow at the peak stage, and does not indicate the peak flow at each station.

HEC-RAS Output Summary - Spillway Modification Alternative 4-2: Lower Spillway with Flat Crest

Dam Non-failure at PMF

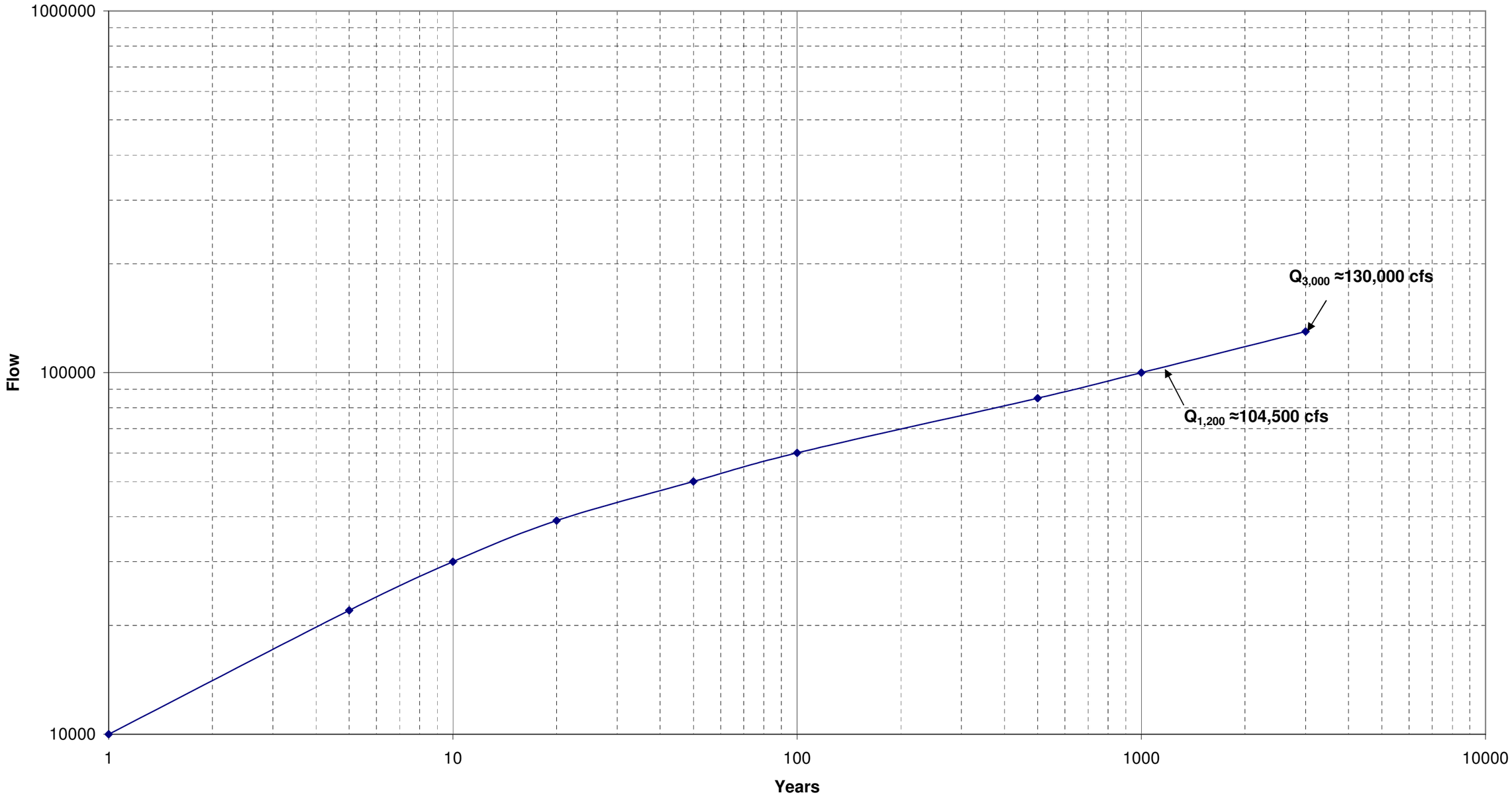
HEC-RAS Plan: DeepFlatPMFN River: Bear River Reach: Reach 1 Profile: Max WS

Reach	River Sta	Profile	Q Total	W.S. Elev	Min Ch El	Diff	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)		(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	112747	Max WS	125471.40	319.98	170.00	149.98		320.02	0.000003	1.77	71585.33	772.06	0.03
Reach 1	110642.*	Max WS	125815.60	320.00	156.37	163.63		320.02	0.000001	0.81	156585.50	1723.02	0.01
Reach 1	108538	Max WS	125799.00	320.01	154.00	166.01		320.01	0.000000	0.52	250886.90	3418.53	0.01
Reach 1	106009.*	Max WS	125750.00	320.01	149.11	170.90		320.01	0.000000	0.24	521127.50	6404.37	0.00
Reach 1	103480	Max WS	125700.30	320.01	144.22	175.79		320.01	0.000000	0.21	614901.40	9466.44	0.00
Reach 1	101143	Max WS	125679.10	320.01	140.04	179.97		320.01	0.000000	0.27	468838.00	4664.00	0.00
Reach 1	98487.5*	Max WS	125702.70	320.01	137.45	182.56		320.01	0.000000	0.29	439954.50	5070.68	0.01
Reach 1	95832	Max WS	125678.40	320.01	134.86	185.15	153.15	320.01	0.000000	0.41	329233.30	6335.74	0.01
Reach 1	95700		Inl Struct										
Reach 1	94971	Max WS	121204.70	166.31	134.83	31.48		169.81	0.001326	15.67	9206.06	480.28	0.54
Reach 1	91510.5*	Max WS	122463.50	151.62	126.42	25.20		156.18	0.002226	17.67	7901.82	481.16	0.68
Reach 1	88050	Max WS	103819.50	141.68	118.00	23.68		143.70	0.000994	12.06	10374.37	612.34	0.46
Reach 1	84540	Max WS	98432.76	131.49	108.79	22.70		132.47	0.001297	13.28	21521.50	3290.23	0.50
Reach 1	80961	Max WS	96940.39	123.74	102.07	21.67		124.22	0.000867	10.87	33877.32	7497.15	0.42
Reach 1	77195	Max WS	111720.30	115.28	94.00	21.28		116.35	0.001599	14.54	26728.15	6842.90	0.57
Reach 1	73083	Max WS	63920.29	110.57	84.98	25.59		110.60	0.000049	2.79	71873.66	9624.66	0.10
Reach 1	68898	Max WS	-149962.00	109.26	83.38	25.88		109.30	0.000084	-3.60	116235.40	12752.04	0.13
Reach 1	64526	Max WS	86566.48	108.37	79.56	28.81		109.58	0.000534	9.99	12492.10	732.84	0.35
Reach 1	60526	Max WS	479116.90	104.01	73.44	30.57	104.19	199.40	0.048363	100.06	9195.13	693.93	3.27
Reach 1	55349	Max WS	286844.10	98.31	66.06	32.25	99.00	113.01	0.009506	44.15	12331.09	693.92	1.42
Reach 1	51931	Max WS	358742.30	93.66	63.48	30.18	93.91	125.44	0.016502	57.53	11335.72	770.08	1.91
Reach 1	42954	Max WS	319796.80	88.69	54.47	34.22		88.72	0.000033	2.69	249431.80	12490.15	0.09
Reach 1	33734	Max WS	637072.20	74.50	47.61	26.89		75.43	0.001202	14.38	89778.55	5193.20	0.51
Reach 1	27087	Max WS	539869.60	67.47	43.99	23.48		68.07	0.000683	9.79	99932.17	6186.77	0.38
Reach 1	19841	Max WS	491468.90	64.17	35.00	29.17		64.33	0.000182	5.48	166221.60	8756.55	0.20
Reach 1	13315	Max WS	363330.70	62.09	28.00	34.09		62.50	0.000307	8.01	80290.05	3524.80	0.27
Reach 1	5290	Max WS	1680.15	52.78	16.00	36.78	18.66	52.78	0.000000	0.08	25760.01	1184.97	0.00

Note: Column "Q Total" indicates the flow at the peak stage, and does not indicate the peak flow at each station.

Appendix I. Flood-Frequency Curve at Bear River near Wheatland

Extrapolation of Flood Frequency Curve
Bear River near Wheatland, CA

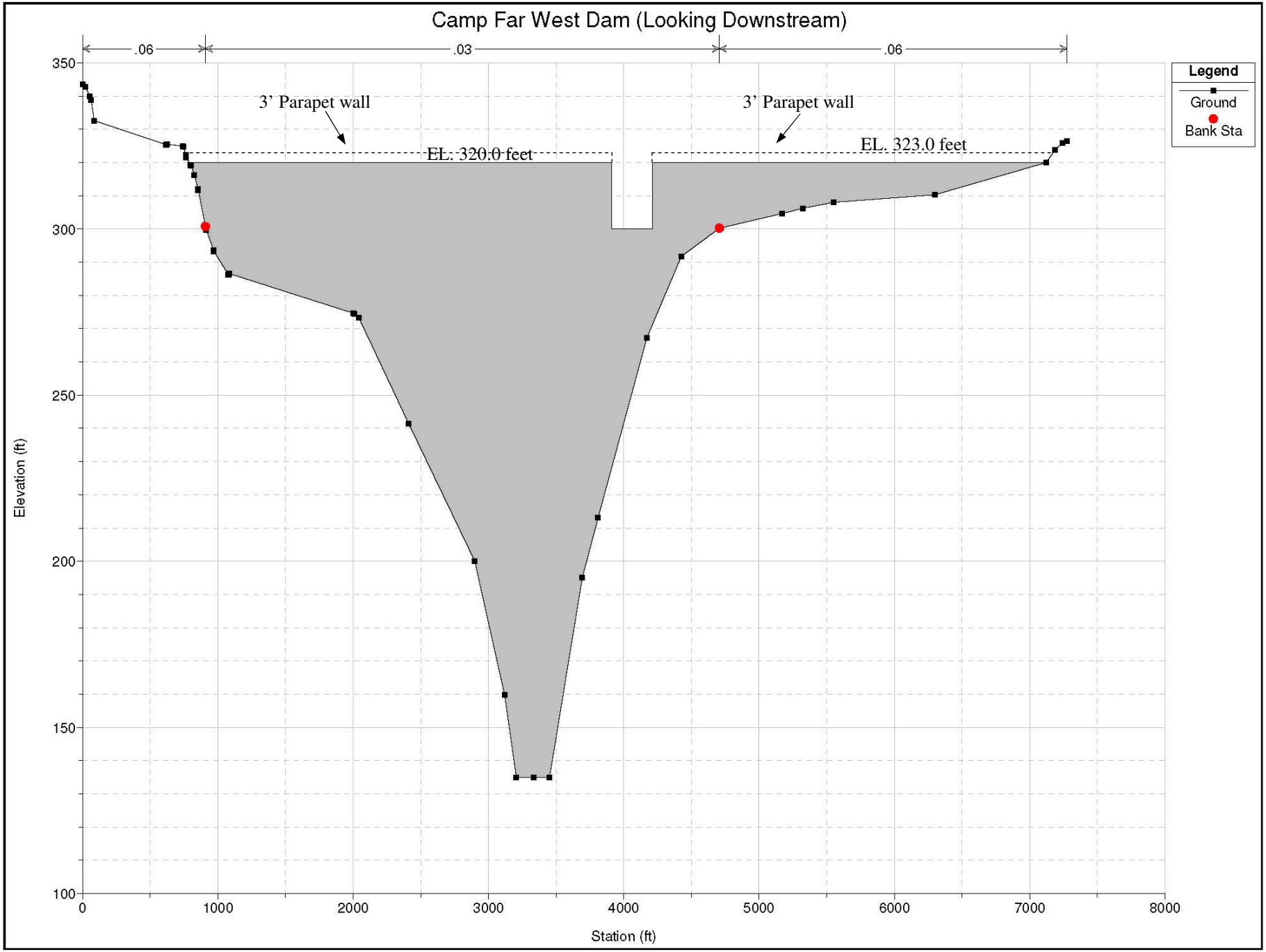


Note: This is an approximate extrapolation of the original flood frequency curve for the Bear River near Wheatland, CA, which was developed by the Corps of Engineers, Sacramento, CA.

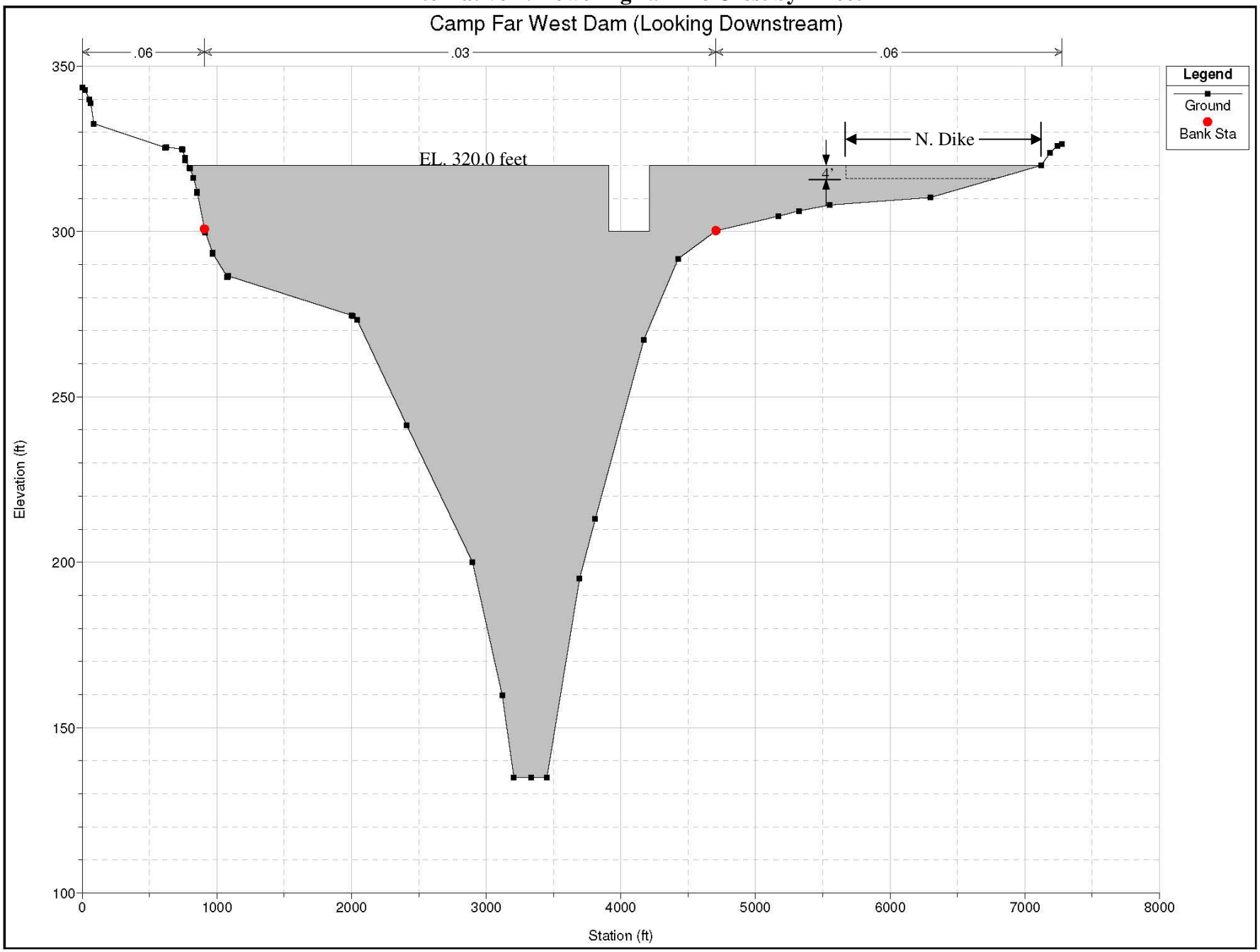
Revised by Mead Hunt, Inc.
August 2006

Appendix J. Sketches of Spillway Modification Alternatives and Cost Estimates

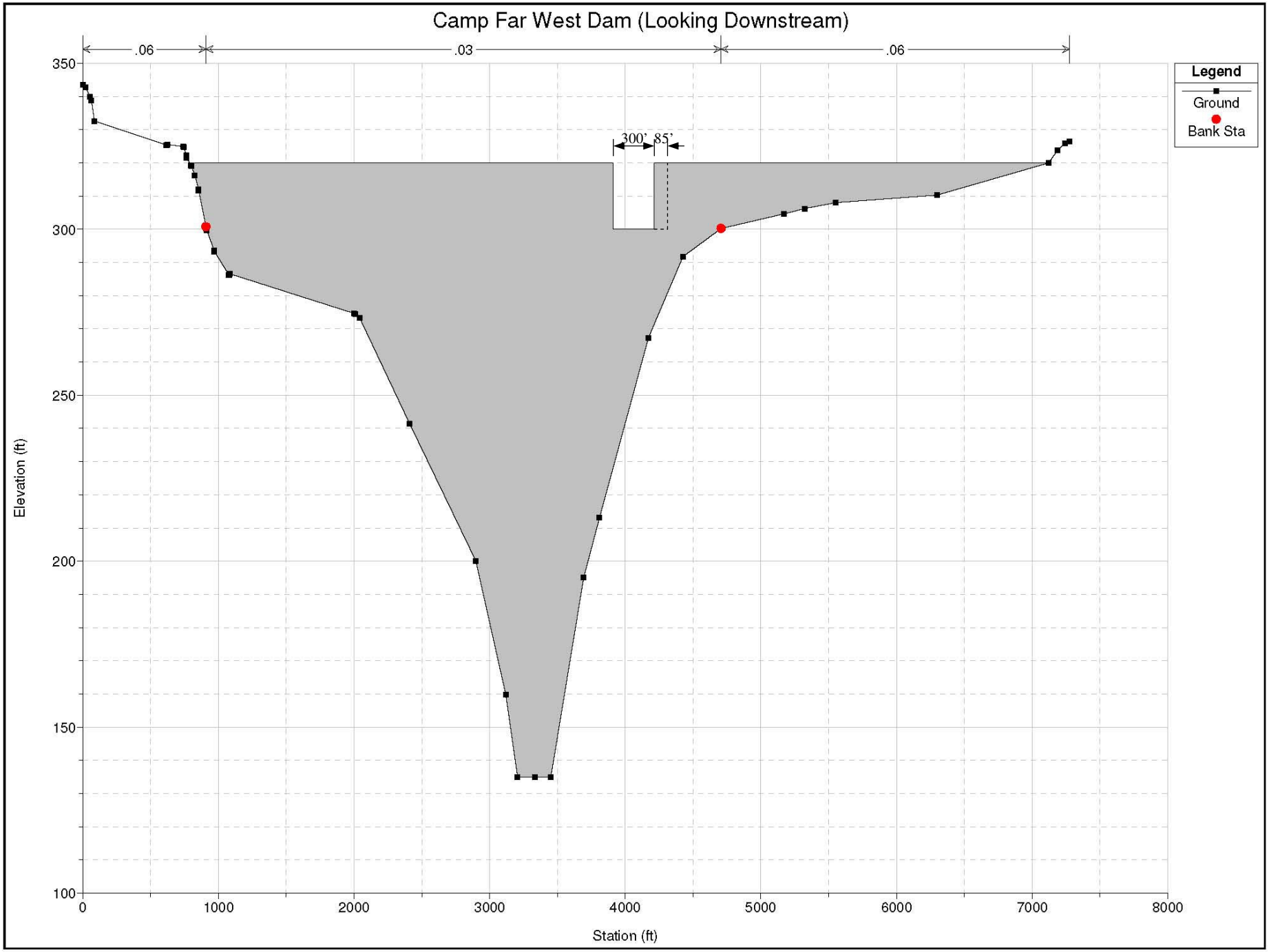
Alternative 1: Raising the Dam with a 3-Foot Parapet Wall



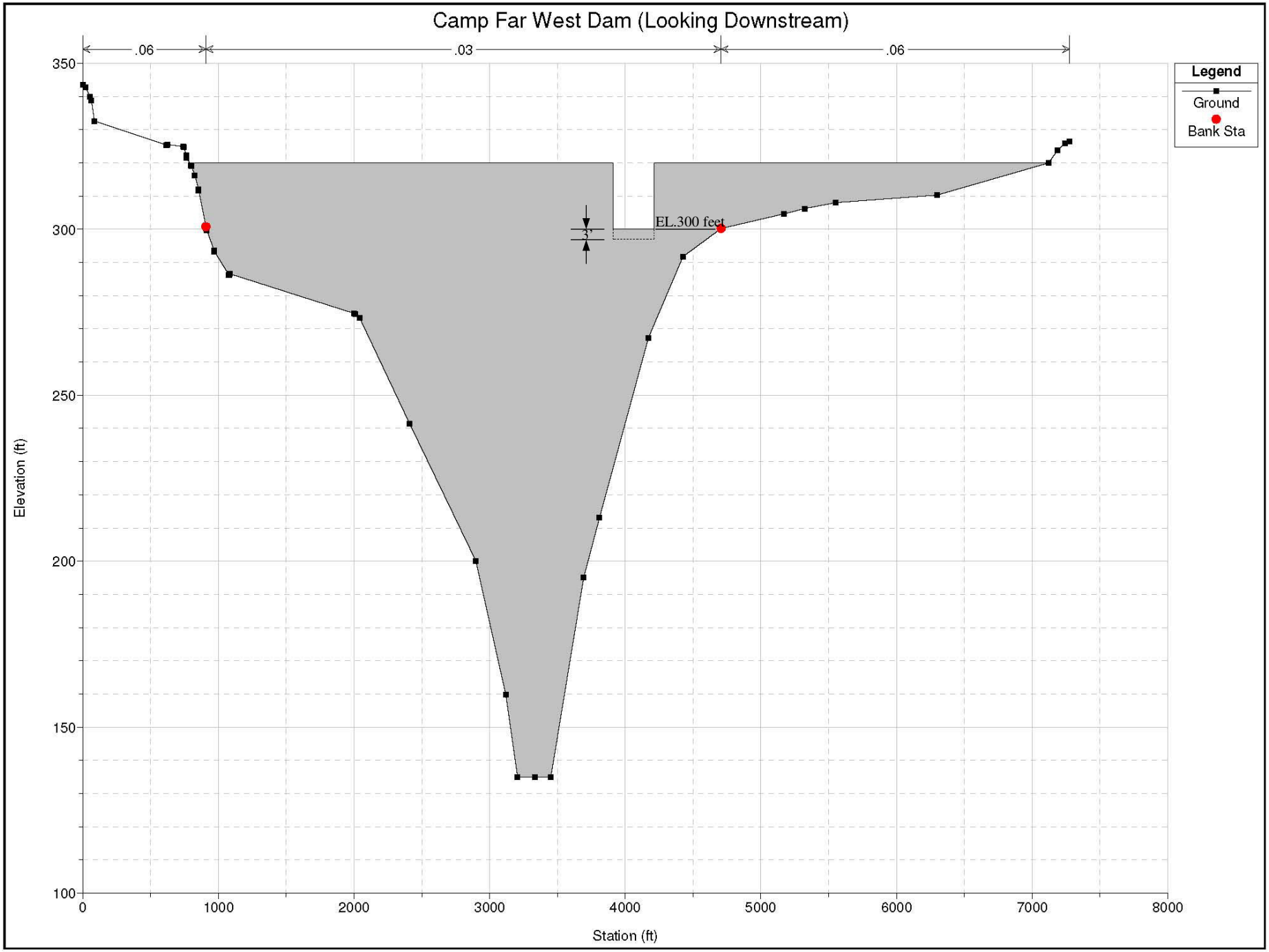
Alternative 2: Lowering N. Dike Crest by 4 Feet

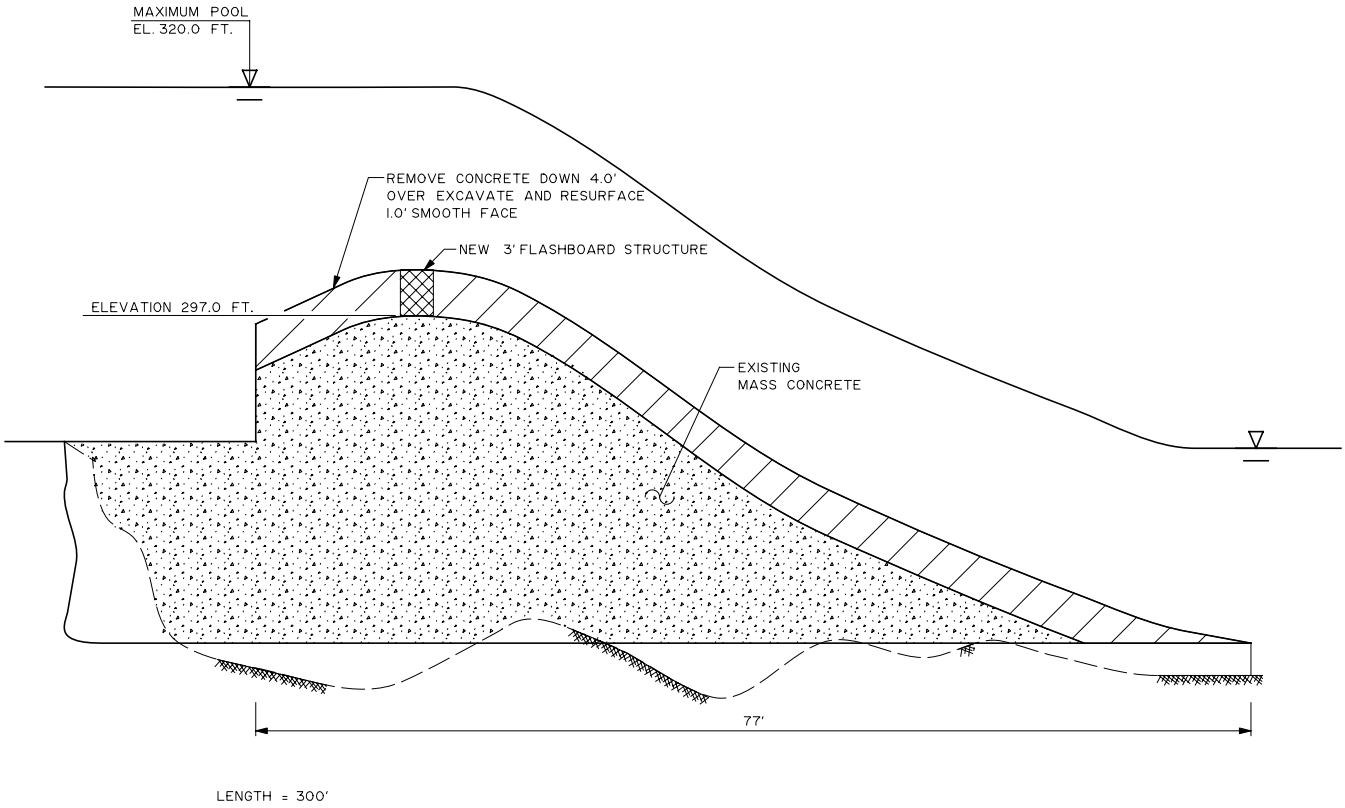


Alternative 3: Parallel Auxiliary Spillway



Alternative 4-1: Lower Ogee-shaped Spillway



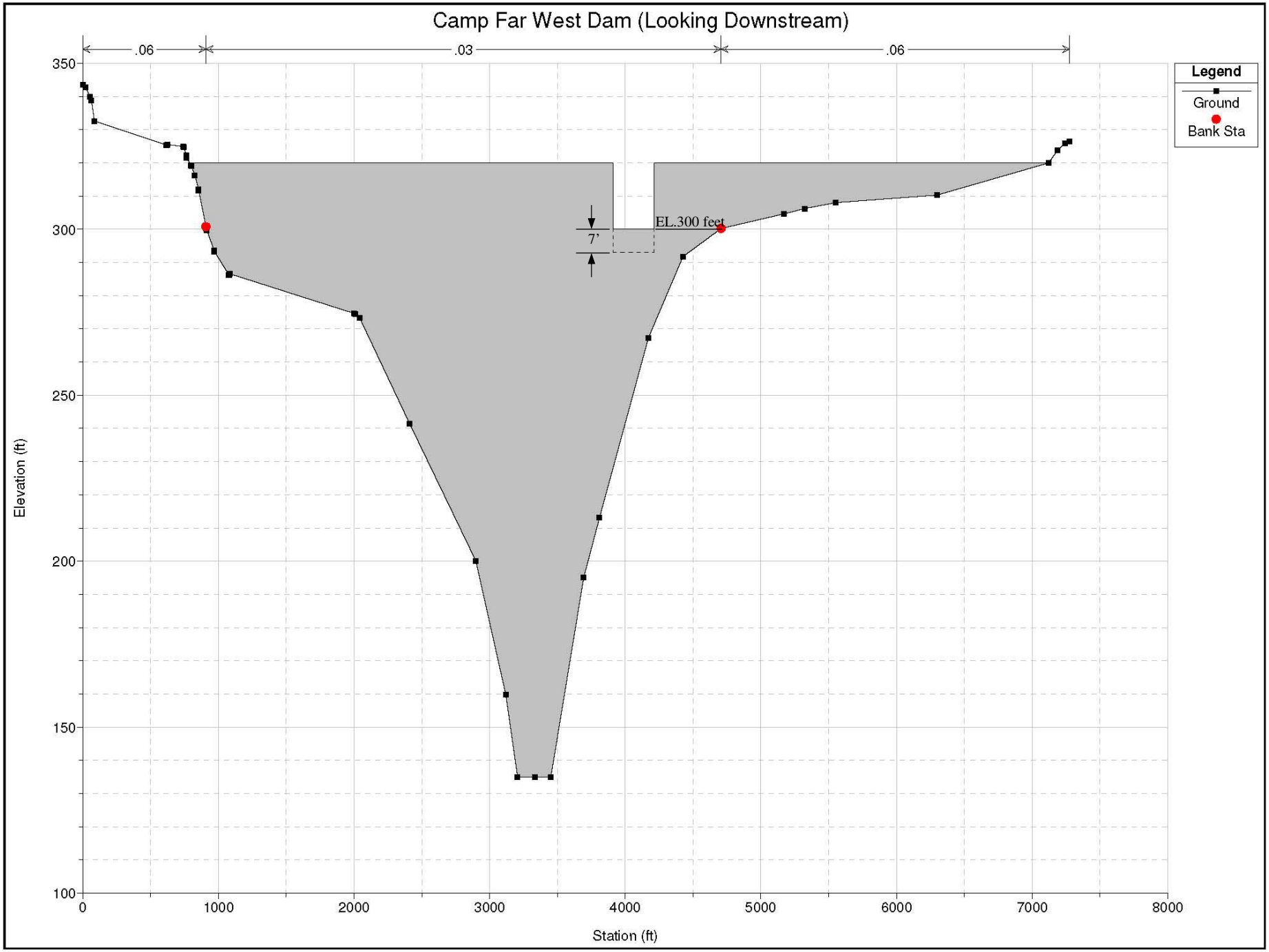


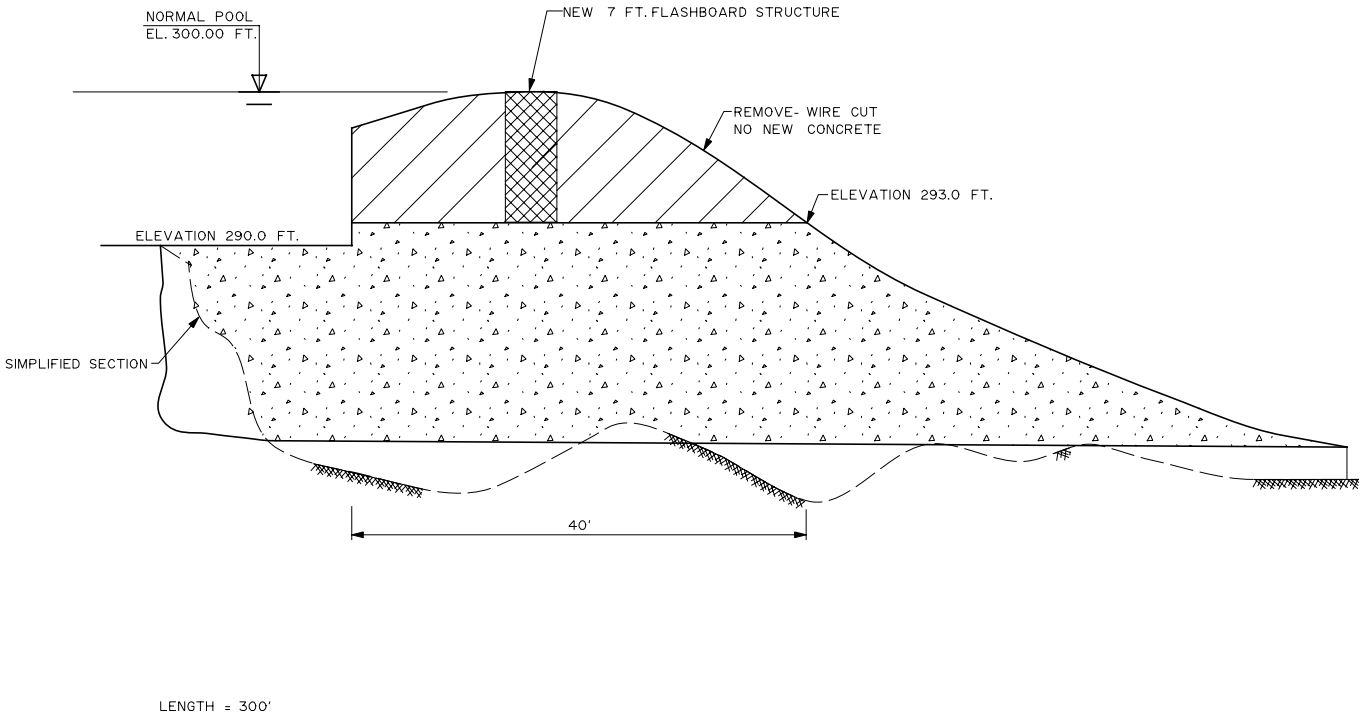
CAMP FAR WEST DAM
ALTERNATIVE 4-I NOT TO SCALE

MEAD & HUNT
ENGINEERS
ARCHITECTS
SCIENTISTS
PLANNERS
3327 Longview Drive North Highlands, California 95660
Phone: (916) 971-3961 Fax: (916) 971-0578

FILE NAME: X:\1941-00\0800\TECH\CadReport_typ.dgn PLOT DATE: 8/25/2006 PLOT BY: Jeff Bill

Alternative 4-2: Lower Uncontrolled Spillway with Flat Crest





LENGTH = 300'

CAMP FAR WEST DAM
ALTERNATIVE 4-2

NOT TO SCALE

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ENGINEERS
ARCHITECTS
SCIENTISTS
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FILE NAME: X:\1941-00\06001\TECH\CadReport\t_typ.dgn PLOT DATE: 8/25/2006 PLOT BY: Jeff Bill

Camp Far West Dam Spillway Modification Alternatives
Cost Estimates and Assumptions

*All costs include 15% E.&C., and 10% Contingency

Alternative 1 – Raise Dam using Parapet Crest Wall

- Assumes a secant-type augured-in-place reinforced concrete parapet wall 12 feet deep over the entire crest of the embankment

Estimated Cost – $4.1 Million

Alternative 2 – Lower Critical Section of Dam

- Assumes the entire length of the north low dike cut down by 4 feet
- Assumes a new floodwall downstream of the north dike will be required to protect existing development

Estimated Cost – $2.7 Million

Alternative 3 – Construct Auxiliary Spillway

- Assumes an 85-foot wide spillway of identical geometry to the existing spillway will be constructed contiguous with or adjacent to the existing spillway.

Estimated Cost – $3.2 Million

Alternative 4-1 – Lower Existing Ogee Spillway 3 Feet

- Assumes the existing spillway ogee geometry will be preserved
- Assumes spillway concrete will be partially removed and resurfaced
- Assumes a new 3-foot high flashboard structure will be constructed to maintain current headwater

Estimated Cost – $1.2 Million

Alternative 4-2 – Lower Existing Spillway 7 Feet to Flat Crest

- Assumes the existing spillway will be cut to a flat crest approximately 40 feet long, 7 feet below the existing elevation
- Assumes no resurfacing will be required
- Assumes a new 7-foot high flashboard structure will be constructed to maintain current headwater

Estimated Cost – $1.4 Million