Application of Mixed-Integer Programming for Flood Control in the Sacramento Valley: Insights & Limitations

By

DUSTIN JON JONES
B.S. (California State University, Fresno) 1997

THESIS

Submitted in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

in

Engineering

in the

OFFICE OF GRADUATE STUDIES

of the

UNIVERSITY OF CALIFORNIA

DAVIS

Approved by: ____
Chairperson of Thesis Committee

____

____

1999

Table of Contents

List of Figures

List of Tables

Acknowledgments

The U.S. Army Corps of Engineers, Sacramento District, provided observed flow and stage data, outlet rating curves for each of the reservoirs, flow-diversion relationships for several weirs, and economic (flood damage) data for each impact area. David Ford Consulting Engineers provided preliminary incremental flow data, routing criteria, and assisted with the development of stage-damage relationships.

The author wishes to thank the following individuals (in alphabetical order) for their significant contributions to the development of this report: Mike Burnham for assisting in the development of the damage functions and providing overall study direction and support; Richard Hayes and Marilyn Hurst for assisting with system and data development; Dr. Bruce Larock, Dr. Jay Lund, and Jeff Olsen for providing technical assistance; and Dr. David Watkins for assisting with model formulation and execution and providing extensive advice. Without their support, this report would not have been possible.

Abstract

This report presents results from an optimization study of the Sacramento Basin flood control system using the Hydrologic Engineering Center's flood control optimization software, HEC-FCLP. The objective of this study is to determine whether significant benefits might be realized from an integrated operation of the system. To do this, a deterministic mixed-integer program (MIP) is developed and applied to the 1995 and 1997 flood events. A MIP model, rather than a linear programming (LP) model, is used to allow a more accurate representation of non-convex constraint sets.

The objective of the model is to minimize damage throughout the system by deciding what releases should be from each reservoir during each time step of the analysis. For this study a 6-hour time step is used. Penalties are incurred for exceeding certain defined storage and flow levels or for exceeding the change-in-release constraints.

Results of this study show that when incremental inflows to the system are high, Shasta Dam has an appreciable effect only as far downstream as the Bend Bridge gaging station and the Feather/Yuba River system consisting of Oroville Dam and New Bullards Bar Dam has an appreciable effect only as far as the Nicolaus gaging station. The results imply that these subsystems could be optimized separately from the complete system under these conditions.

This study illustrates that MIP is a useful tool for flood control optimization. However, it is also found that solving complex systems using MIP can lead to excessive computation times. Simplifications must be implemented whenever practical to reduce the number of binary variables used by the model.

Chapter 1
INTRODUCTION

1.1 PURPOSE

This document describes a reservoir system optimization model developed for the flood control analysis of the Sacramento River Basin. The purpose of the analysis is to determine whether flood damage can be reduced in the Sacramento Basin by operating flood control projects as an integrated system. This kind of analysis has been able to estimate the value of integrated flood control operations elsewhere (USACE, 1999). By examining the optimal operating sequence for various events, it also may be possible to infer improved operating rules. However, owing to the limited availability of historic data, this will not be attempted in this study. The optimized variables could be plotted in various combinations (e.g., release versus storage, release versus inflow), and regression analysis performed. This technique has suggested improved operating policies in other systems, such as the Missouri River System (USACE, 1994) and the Columbia River System (USACE, 1996).

The potential benefits of various alternatives also can be evaluated using the optimization model. This could be demonstrated by considering a few hypothetical alternatives, such as storage allocation, levee realignment, and reservoir outlet modification. In the case of storage reallocation, the initial storage levels and/or storage penalty functions would be changed, and the minimized flood damages would then be compared (with and without the changes). In the case of levee realignment, model results would be compared with one or more discharge-damage relationships changed. For reservoir outlet modifications, the rating curve in the model would be changed.

1.2 SACRAMENTO RIVER BASIN SYSTEM

The four main rivers that drain the Sacramento Basin are the Sacramento River, Feather River, Yuba River, and the American River. The basin is bounded by the Sierra Nevada on the east, the Coast Range on the west, the Cascade and Trinity Mountains on the north, and the Sacramento-San Joaquin Delta on the south. The basin is approximately 240 miles long and up to 150 miles wide. Figure 1 shows a map of the Sacramento Basin.

1.2.1 Drainage Areas

The Sacramento River Basin drains approximately 25,000 square miles. Table 1 lists the regions that the Sacramento River drains along with their approximate drainage areas. Table 3 lists the drainage areas above each of the principal flood control reservoirs in the system.

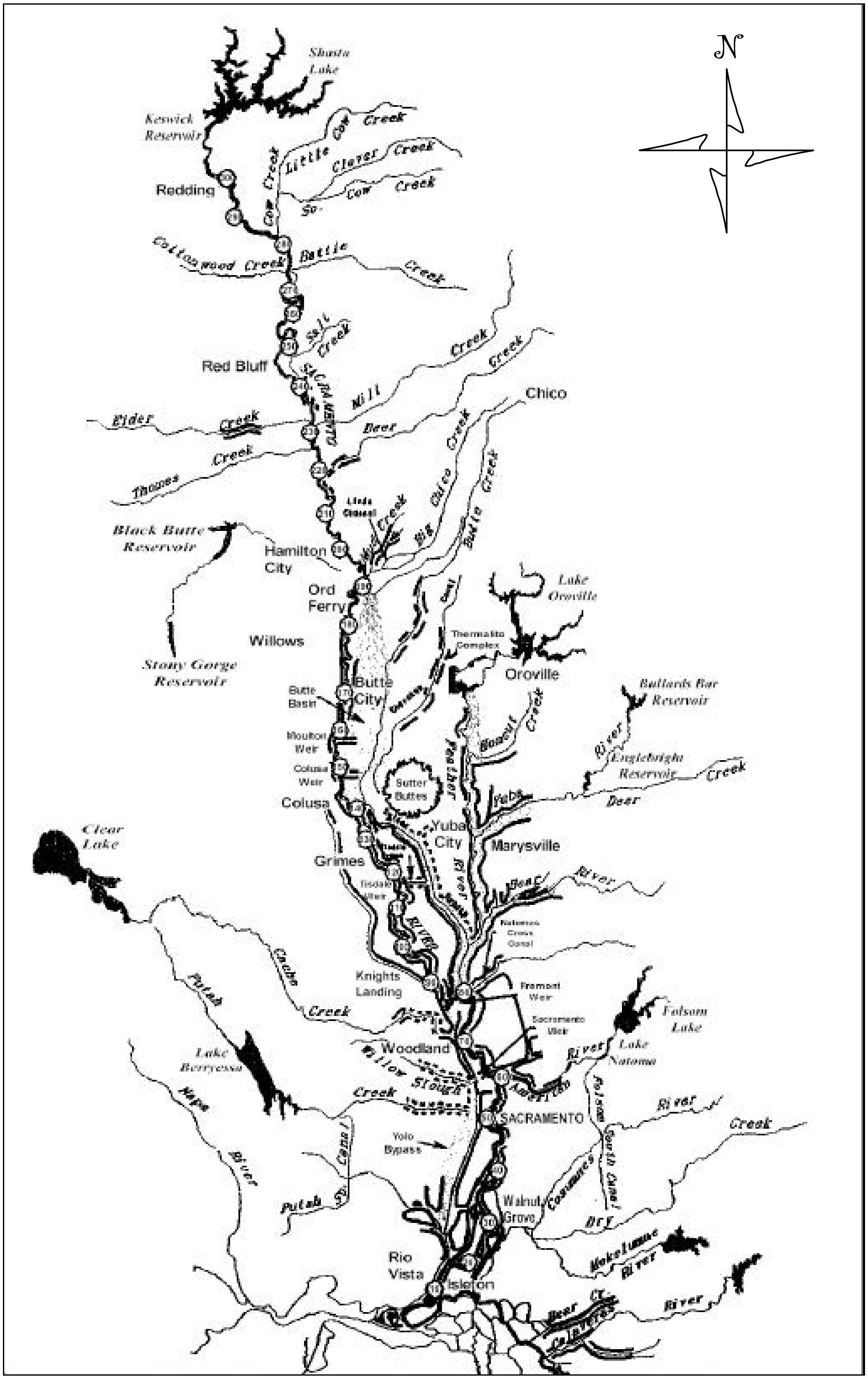

Figure 1. Map of Sacramento Basin.

Table 1. Sacramento Basin drainage areas.

Region	Drainage Area (sq mi)
Sacramento R. above Shasta Dam	6,420
Shasta Dam to Colusa	6,180
Yuba R. abv. confluence w/ Feather R.	1,350
Feather R. abv. Gridley	3,676
American R. at mouth	2,100
Below Colusa	5,300
Sacramento River Basin (total)	**25,000**

USACE 1970, 1972, 1987, 1993

1.2.2 Regional Hydrology

The storm period for the Sacramento Basin is October through April, with maximum flows usually occurring between November and April. Snowfall usually occurs above 5,000 feet elevation. Snowmelt runoff alone does not cause damage in the Sacramento Basin. Damaging floods on the Sacramento River are usually caused by winter rainstorms augmented by snowmelt. The average annual precipitation above five of the flood control reservoirs in the Sacramento Basin is listed in Table 2.

Table 2. Sacramento Basin regional precipitation.

Region Above	Precipitation (inches)
Shasta Dam	68
Black Butte Dam	32
Oroville Dam	44
Folsom Dam	53
New Bullards Bar	70

USACE 1970,1972,1977,1987

1.3 DESCRIPTION OF FLOOD EVENTS

Two flood events will be analyzed in this report, the 1995 and the 1997 events. These events were chosen for their magnitude and the availability data. The system operation for the 1995 event will be modeled between March 8 and 22, 1995. Operation between December 26, 1996 and January 10, 1997 will be modeled for the 1997 event. A description of each event is given below.

1.3.1 1995 Flood Event

The “Northern Sierra” 8-Station Precipitation Index is a wetness index of the north and northeastern mountains of the Sacramento River hydrologic region. Based on this index, 1995 was the second wettest year since the record began in 1922. The index of 85.4 inches was 171% of average and was second only to 1983, which had an index of 88.5 inches (USACE, 1995).

The Sacramento River unimpaired runoff for 1995 was 184% of average, or 33.9 MAF. Unimpaired runoff for 1995 was the second highest since the record began in 1906, exceeded only during the 1983 event (USACE, 1995).

1.3.2 1997 Flood Event

On December 23, 1996, a snowstorm produced heavy snows to low elevations. Over the 3-day period centered on New Year's Day, warm moist winds from the southwest blowing over the Sierra Nevada released more than 30 inches of rain onto the saturated watersheds. The entire northern Sierra received approximately 40 percent of its average annual precipitation in just a few days. The existing snowpack was melted at relatively low elevations. The middle and high elevation snowpack, however, remained. The rain percolated through the pack, and little snow was lost. This contrasts with the public's impression that the melting snow caused the floods. Snowmelt from lower elevations only added about 15 percent to the runoff. The bulk of the runoff was caused by rain, which in a normal year would occur as snow and be held in "cold storage" instead of flowing to the rivers (USACE, 1997).

The resulting New Year's Day Flood of 1997 was probably the largest in the 90-year northern California record. It was notable in the sustained intensity of rainfall, the volume of floodwater, and the areal extent – from the Oregon border to the southern end of the Sierra Nevada. New flood records were set on many of the major Central Valley rivers. This record inflow volume left most flood control projects in northern and central California full or nearly full within the first days in January (USACE, 1997).

1.4 FLOOD CONTROL PROJECTS

Development in the Sacramento River Basin is protected by a system of reservoirs, levees, and bypasses. The five major flood control dams in the Sacramento Basin are Shasta Dam, Black Butte Dam, Oroville Dam, New Bullards Bar Dam, and Folsom Dam. An extensive system of levees protects potential damage locations, including the City of Sacramento. Bypasses are also essential to relieve high flows in the Sacramento River. Chapter 2 of this report lists criteria for operation of these flood control projects.

1.4.1 Flood Reduction Reservoirs

The locations of the five flood control reservoirs are shown in Figure 1. Table 3 lists the modeled reservoirs along with their respective drainage areas, cumulative storage for each operational level, and the maximum amount of storage allocated for flood control. Table 4 lists reservoirs along with their respective standard project flood peak flow and three-day volume.

1.4.2 Bypasses

Two bypasses are used to relieve high flows in the Sacramento River. These are the Sutter Bypass and the Yolo Bypass. Bypass design flows are listed in Chapter 2. Flows enter the Sutter Bypass from Moulton Weir through Butte Basin, from Colusa Weir through Butte Slough, and directly from Tisdale Weir. The Fremont Weir and Sacramento Weir divert flows to the Yolo Bypass. The bypass locations are shown in Figure 1.

1.4.3 Levees

The Sacramento Basin has a network of levees that protect development along various reaches throughout the system. Figure 1 shows the locations of these levees.

Waterways with levees, along with the relative locations of each levee system, are listed in Table 5. Design flows within various reaches also are discussed in Chapter 2.

Table 3. Reservoir descriptions.

Reservoir (Drainage Area)	Operating Levels	Cumulative Capacity (AF)	Flood Control Space (KAF)
Shasta[a] (6,421 sq mi)	Top of Dam	4,850,000	1,300
	Top of Gross Pool	4,552,000	
	Top of Conservation Pool	3,250,900	
	Minimum Operating Pool	587,100	
Black Butte[b] (741 sq mi)	Top of Dam	389,000	137
	Spillway Design Flood Pool	354,000	
	Standard Project Flood Pool	223,000	
	Gross Pool	143,676	
	Inactive Pool	6,640	
Oroville[c] (3,611 sq mi)	Top of Dam	3,870,000	750
	Spillway Design Flood Pool	3,814,000	
	Gross Pool	3,538,000	
	Top of Conservation Pool	2,788,000	
	Minimum Power Pool	852,200	
New Bullards Bar[d] (489 sq mi)	Top of Dam	1,010,000	170
	Spillway Design Flood Pool	998,000	
	Gross Pool	960,000	
	Top of Conservation Pool	790,000	
	Minimum Power Pool	233,600	
Folsom[e] (1,861 sq mi)	Top of Dam	1,300,000	400
	Spillway Design Flood Pool	1,130,000	
	Gross Pool	1,010,000	
	Top of Conservation Pool	610,000*	
	Minimum Power Pool	90,000	

(a) USACE 1977 (b) USACE 1987 (c) USACE 1970 (d) USACE 1972 (e) USACE 1987

* As listed in Folsom Dam and Lake Water Control Manual. For analysis, 486,000 AF will be used to correspond with 1996-1997 operation.

Table 4. Standard project flows and storage.

Reservoir	Standard Proj. Flood Peak Flow (cfs)	Standard Proj. Flood 3-day Volume (AF)
Shasta	345,000	1,574,000*
Black Butte	95,000	254,000
Oroville	440,000	1,520,000
New Bullards Bar	150,000	374,000
Folsom	530,100	1,121,500

* 5-day volume

Table 5. Levees.

Waterway	Reach
Sacramento River	190 mi, Ord Ferry to Collinsville
Feather River	73 mi, Oroville to Sacramento R.
Honcut Creek	4.5 mi, upstream to mouth
Yuba River	7.5 mi, ending at Feather R.
Bear River	3.2 mi, ending at Feather R.
W. Pac. Inter. Canal	Bear River 6 mi to upsteam point
Wadsworth Canal	Sutter Bypass 4.5 mi to upstream point
American River	13 mi, Mahew Drain to Sacramento River
Total length of leveed reaches	**300 mi**

USACE 1993

1.5 REPORT ORGANIZATION

Chapter 2 of this report provides an overview of current operating procedures for flood control in the Sacramento Basin. A discussion of the need for optimization analysis is also presented. Chapter 3 discusses the analysis approach and the model formulation. Chapter 4 covers the model application, including the assumptions made and a discussion of deviations in operation from the rules specified in the regulation manuals. The results are discussed in Chapters 5 and 6. Chapter 7 presents the results of two subsystem analyses, and conclusions are drawn in Chapter 8.

Chapter 2
SYSTEM OPERATION

2.1 CURRENT OPERATING PROCEDURES

This section presents criteria currently used for operating flood control projects in the Sacramento Basin. Design flows at various control points throughout the system are given along with reservoir release parameters. The two bypass operations also are described.

2.1.1 Design Flow

Various locations in the Sacramento Basin, along with their respective design flows, are listed in Table 6. Figure 1 shows these locations in the system. The Sacramento River flood control system was designed under the assumption that a flow of 579,000 cfs at Rio Vista was a rare event and that the upstream flows that contribute to this event also are rare (USACE, 1993).

Table 6. Sacramento River Basin design flows.

Location	Design Flow (cfs)
Sacramento River below	
Bend Bridge	100,000
Vina-Woodson	260,000
Ord Ferry	160,000
Butte City	160,000
Moulton Weir	160,000
Colusa Weir	60,000
Tisdale Weir	30,000
Verona	107,000
Sacramento Bypass	107,000
Sacramento (I street)	110,000
Freeport	110,000
Rio Vista	579,000
Sutter Bypass	
Downstream of Tisdale Bypass	180,000
Downstream of Feather River	380,000
At confluence w/ Sacramento River	380,000
Feather River	
Above Yuba River (at Yuba City)	210,000
At Nicolaus	320,000
Yuba River at Feather River (Marysville)	120,000*
American River at H Street Bridge	115,000
Sacramento-Feather River Confluence (SFRC)	410,000
Yolo Bypass Below	
Fremont Weir	343,000
Woodland	377,000
Sacramento Bypass	480,000
Lisbon	490,000

USACE 1993
* 180,000 cfs when flows in Feather R. are low

2.1.2 Reservoir Operation

Flood control reservoirs in the Sacramento system operate to maintain safe discharges at designated downstream locations. The amount of release is typically constrained by the channel capacities and hydraulic limitations of the outlet works. Rates of increase in release are limited to allow sufficient time for evacuation downstream. Rates of decrease in release are limited to allow groundwater in adjacent banks to drain, thereby reducing bank sloughing.

Shasta Dam. Operation of Shasta Dam requires that flows do not exceed 100,000 cfs at the Bend Bridge gaging station (approximately 50 mi downstream). Bend Bridge is the farthest downstream control point used in the flood control operation of Shasta Dam. Releases from Shasta Dam flow immediately into Keswick Reservoir. A constraint is then placed on the increase and decrease in release from Keswick Dam. However, Keswick Dam is relatively small and has no flood control storage, therefore it is not included in this analysis. Thus, limitations on change in release for Keswick Dam are applied directly to Shasta Dam. Releases from Shasta Dam, therefore, should not be increased by more than 15,000 cfs or decreased by more than 4,000 cfs in any 2-hour period. Runoff forecasts are required 6 to 24 hours in advance for operation of Shasta Dam for flood control (USACE, 1977).

Black Butte Dam. Black Butte Dam is operated to limit flows in Stony Creek below the dam to 15,000 cfs whenever Sacramento River flow at Ord Ferry exceeds 130,000 cfs. Releases from Black Butte Dam shall not be increased by more than 2,000 cfs in any 2-hour period. No amount of release over 1,000 cfs shall be held for more than 18 hours. Releases are decreased according to the following guidelines:

1. *When existing release is between 15,000 cfs and 5,000 cfs, outflow shall be reduced in 1,000 cfs increments with no release sustained for less than 2 hours.*

2. *When existing release is between 5,000 cfs and 50 cfs, outflow shall be reduced in 500 cfs increments, with no release sustained for less than 2 hours.*

The National Weather Service in Sacramento forecasts precipitation amounts for the Stony Creek Basin for the succeeding 24-hour period (USACE, 1987).

Oroville Dam. Oroville Dam is operated to prevent flows on the Feather River from exceeding 150,000 cfs at Oroville, 180,000 cfs above and 300,000 cfs below the mouth of the Yuba River, and 320,000 cfs below the mouth of the Bear River. Oroville Dam releases should not be increased by more than 10,000 cfs or decreased by more than 5,000 cfs during any 2-hour period. The National Weather Service in Sacramento provides 24-hour forecasts twice a day. From January through May, the National Weather Service also publishes water supply forecasts indicating the forecasted volume of runoff for the remainder of the water year (USACE, 1970).

New Bullards Bar. New Bullards Bar reservoir is operated so as not to cause flow in the Yuba River at Marysville to exceed 120,000 cfs (180,000 cfs when flow in the Feather River is low). The dam also is operated to keep flow in the Feather River below the Yuba River confluence from exceeding 300,000 cfs and below the Bear River confluence from exceeding 320,000 cfs. Releases at New Bullards Bar Dam should not be increased or decreased by more than 5,000 cfs in any 1-hour period (USACE, 1972).

Folsom Dam. Folsom Dam is operated so as not to cause flows in the American River to exceed 115,000 cfs. Folsom releases should not be increased more than 15,000 cfs or decreased more than 10,000 cfs in any 2-hour period. The National Weather Service in Sacramento forecasts precipitation amounts for the American River Basin at six-hour intervals for the twenty-four hour period following the forecast (USACE, 1987).

2.1.3 Bypasses

The Sutter Bypass and Yolo Bypass act to relieve excessive flow in the Sacramento River. Flow enters the Sutter Bypass over Moulton Weir, Colusa Weir, and Tisdale Weir. Some flow at Ord Ferry spills into Butte Basin during times of high flow, although there is no weir at that location. Overflow at Ord Ferry into Butte Basin occurs at 195,000 cfs, and overflow into Colusa Basin occurs at 300,000 cfs (USACE, 1977).

Flows are diverted to the Yolo Bypass by the Fremont Weir and Sacramento Weir. When the combined flow of the Sacramento and Feather Rivers and Sutter Bypass exceeds approximately 70,000 cfs, most of the excess spills over the Fremont Weir into the Yolo Bypass. The Sacramento Weir is the only gated weir in the system. When the stage in the Sacramento River at the I Street Bridge exceeds 27.5 feet, gates at the Sacramento Weir are opened and excess flows enter the Yolo Bypass (USACE 1970). Table 7 lists the weirs in the order they are intended to spill, along with the flow at which they begin to spill.

Table 7. Sacramento River diversions.

Weirs (in spill order)	Sacramento R. Flow at Weir Crest (cfs)
Tisdale Weir	18,000
Colusa Weir	30,000
Fremont Weir	62,000
Moulton Weir	60,000
Sacramento Weir	37,000

USACE 1993

2.2 NEED FOR OPTIMIZATION ANALYSIS

Although the operation of the Sacramento System for flood control has thus far been sufficient, it may be possible to make improvements. Post-flood optimization analysis can help to determine the best possible release schedule for a specific flood event, given the inflows to the system. The optimal release schedule is the one that minimizes flood damage throughout the system while satisfying the operational goals and constraints. Hypothetical floods greater than, or different from, those already experienced also can be examined. The formulation of the optimization model is discussed in Chapter 3.

The historic flood damage can be compared with that resulting from optimal operation to provide an estimate of the potential benefits gained from operating flood control facilities in a coordinated matter rather than individually. If the computed and historic damages are equal or nearly equal, then it could be assumed that the current operating procedure is optimal. If the computed damage is significantly less than the historic damage, then there may be some benefit to be realized by coordinated operation (USACE, 1999). The value of additional facilities and flood storage could also be examined. The suggestions for improved operations and facilities that are inferred from the optimization model can then be refined and tested by simulation modeling.

Chapter 3
ANALYSIS APPROACH

3.1 REVIEW OF METHODS

Yeh (1985) and Wurbs (1993) describe numerous methods and models developed for improving reservoir operations. The deterministic flood control model proposed by Karbowski (1993) determines the release schedule as a function of aggregated system storage and inflows. Karbowski assumes in his formulation that reservoir outlet capacity is seldom constraining. However, it was found in the study of the Iowa and Des Moines system that release capacity is an essential constraint, especially when dealing with forced spills (USACE, 1999). Therefore, this method is not applicable to the current study.

Georgakakos et al. (1998) have taken into account the uncertainty of reservoir operation owing to forecasts by using historical atmospheric conditions as input to a Monte Carlo simulation that generates “ensembles” of inflows. Long periods of historical records are necessary to represent the climate variability in the simulation. This information is not always readily available, thereby limiting the use of this method for real-time operation.

Wasimi and Kitanidis (1983) developed a state-space model “for short-term forecasting of river flows” that also is meant to be used for real-time reservoir operation. The optimization problem is solved using linear quadratic Gaussian (LQG) control. It was found in their study that the method was “suitable for operation under moderate flood conditions when capacity constraints are not likely to become binding.”

Unver and Mays (1990) have proposed a nonlinear deterministic model for use in real-time flood control operation. The nonlinear programming model is combined with a simulation model to reduce the problem size. A limitation of this method is that the first partial derivatives of the objective and constraint functions with respect to the controllable variables must be definable. In addition, as noted in the paper, nonlinear programming cannot guarantee a global optimum.

Windsor (1973), along with Ikura and Gross (1984), point out that representing outlet rating curves in a model with nonlinear constraints can cause the feasible set to be non-convex. They suggest dealing with this by introducing binary variables for each forced spill condition. This approach will be used in this study.

3.2 OVERVIEW OF LINEAR PROGRAMMING

A linear program is composed of an objective function, model constraints, and decision variables. The objective function is a measure of performance that can be either minimized or maximized, depending on the desired outcome. The general form of the objective function is

$$\text{Maximize } Z = c_1x_1+c_2x_2+\ldots+c_nx_n \qquad \text{(A)}$$

In the above equation x_i represents a level of activity (the decision variables), and c_i represents the amount of increase or decrease in the objective function (cost or benefit) corresponding to a unit change in x_i (Hillier and Lieberman, 1990).

The constraints of the linear program take the form

$$\begin{aligned} a_{11}x_1 + a_{12}x_2 + \cdots + a_{1n}x_n &\le b_1 \\ a_{21}x_1 + a_{22}x_2 + \cdots + a_{2n}x_n &\le b_2 \\ &\vdots \\ a_{m1}x_1 + a_{m2}x_2 + \cdots + a_{mn}x_n &\le b_m \end{aligned} \tag{B}$$

and $\quad x_1 \ge 0, \quad x_2 \ge 0, \quad \ldots, \quad x_n \ge 0$

In the case of reservoir operation, the decision variables are the reservoir releases, storage levels, flow levels, and diversion amounts in each time step. The constraints represent physical limitations of the system such as flow continuity, maximum storage available, and reservoir outlet capacities. The costs or benefits in the objective function could represent either damage caused by flooding or benefits realized from water delivery.

The linear program is then solved by determining the value for each decision variable that results in either a maximum or minimum value of the objective function. These values can be determined by any number of systematic methods, including the simplex method (Hillier and Lieberman, 1990).

3.3 PROBLEM DESCRIPTION

To use a nonlinear cost function in a linear program, it must be approximated by a piecewise linear function. Figure 2 presents an example of a nonlinear cost function along with the piecewise linear approximation. In linear programming models where the costs are to be minimized, the piecewise linear functions used to penalize undesirable storage and flow levels must be convex, as shown in Figure 2. That is, the penalty coefficients, or slopes (c_i in equation A above), must be monotonically increasing. If not, the model may unrealistically place water in higher flow/storage zones before the lower zones are filled.

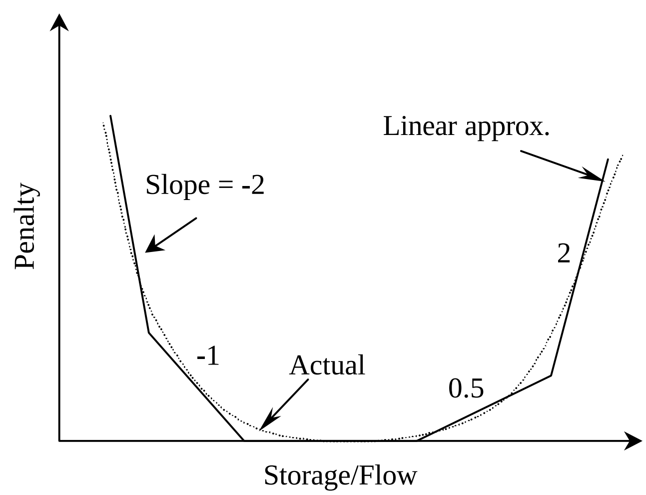

Figure 2. Convex piecewise linear storage/flow penalty function.

However, many cases arise in which damage-flow relationships are not strictly convex. The damage-flow relation for a leveed reach is an example of such a case. Figure 3 shows a sample non-convex damage function. This function represents the case in which no damage occurs until flow surpasses the channel capacity or top of levee. However, once the first flow zone is filled, the model may put water in Zone 3 before Zone 2 is filled. Since damage is calculated as a function of the slope in each zone, the model will calculate less damage by placing water in Zone 1, then Zone 3, and finally Zone 2. This would not violate any constraints of the linear program and would therefore be a feasible solution, although it would be physically impossible.

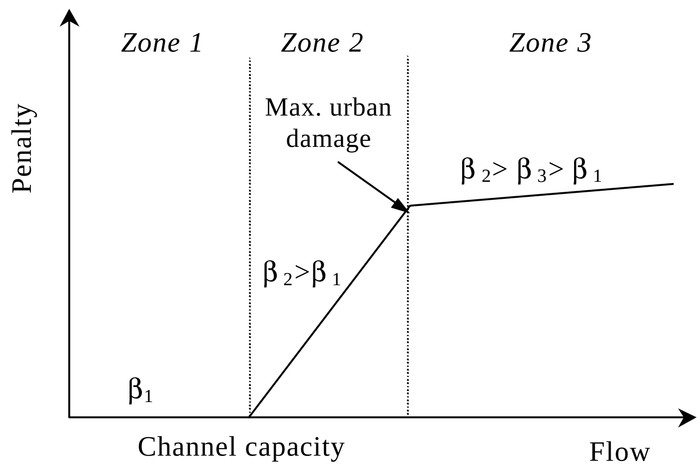

Figure 3. A non-convex penalty function.

Linear programming also requires all constraints to be linear, or for piecewise linear constraints to form convex feasible regions. This is true for diversion functions (assuming diversions reduce flood damage) and outlet rating curves. So long as the coefficients (slopes) are monotonically decreasing, the feasible region for releases and diversions will be convex, and linear programming may be used to find an optimal (and physically realistic) solution. The model will have no incentive to place water in the higher zones before the lower zones are filled. An example of a concave outlet capacity constraint is shown in Figure 4. The storage zones are represented by X_i and the slopes are represented by β_i.

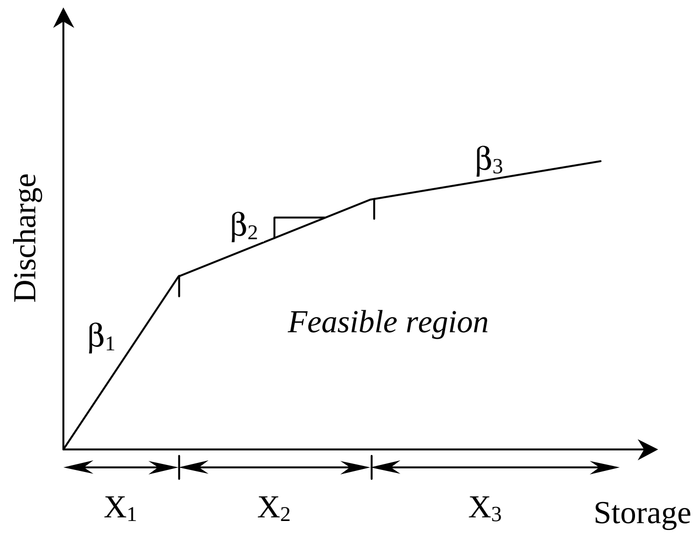

Figure 4. A concave constraint forming a convex feasible region.

Unfortunately, flow-diversion functions and reservoir outlet rating curves are typically non-convex. For reservoirs, this is due to outlet works in multiple tiers, including gated and uncontrolled spillways. Figure 5 shows a piecewise linear approximation of a typical outlet curve. The slope of each line segment is represented as β_i. The storage zones are denoted as S_i. As seen in this figure, the concave function consisting of β_1 and β_2 forms a convex feasible region (the region under the curve). The feasible region formed by β_2 and β_3, however, forms a non-convex feasible region.

With diversions, the non-convex feasible region is formed because flow over the weir does not occur until some designated flow in the channel has been reached. An example piecewise linear diversion function is shown in Figure 6. Here α_i represent the slopes of the function and f_i represent the flow zones.

Unrealistic solutions may result when functions like these are used as constraints in a linear program. The problem with the outlet rating curve, Figure 5, is that the program could potentially place water in the third storage zone, to take advantage of the higher release rates, without filling the second storage zone. Similarly for the diversion function, Figure 6, the program could place water in the second flow zone before the first is filled.

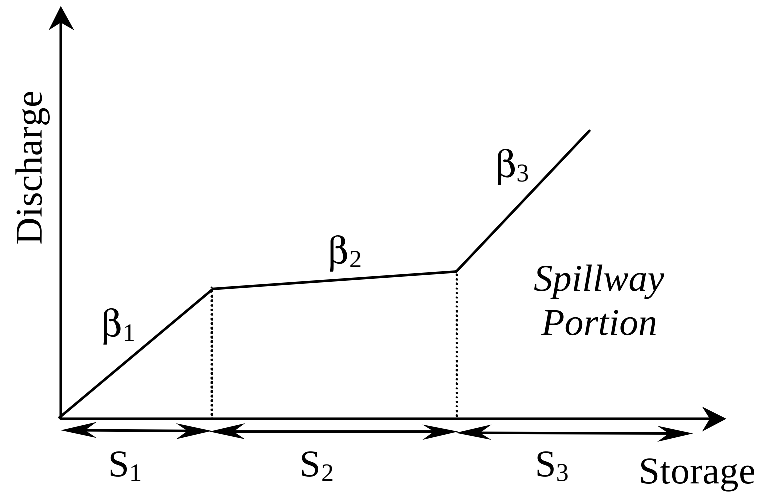

Figure 5. A piecewise linear non-convex outlet rating curve.

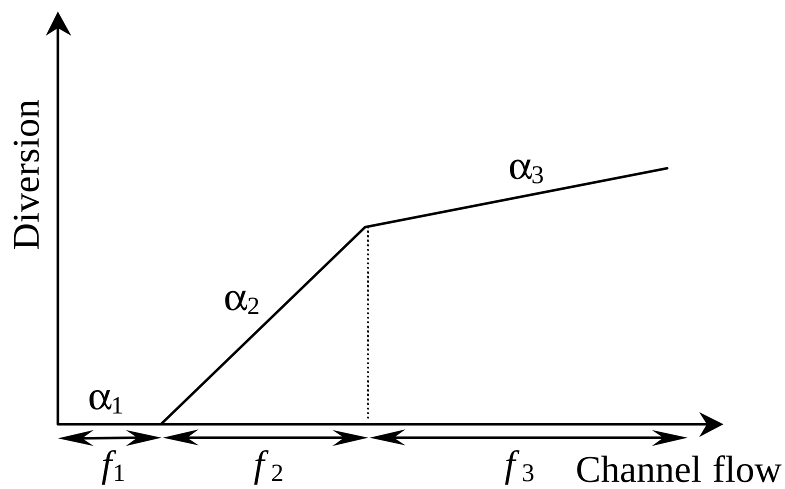

Figure 6. A piecewise linear non-convex diversion function.

3.4 MIXED-INTEGER PROGRAMMING APPROACH

A mixed-integer programming (MIP) approach, similar to that proposed by Windsor (1973), will be used to overcome the limitations mentioned above. It will be used to represent diversion functions as well as forced spill conditions. A disadvantage of the MIP approach is that it may require excessively long solution times and large amounts of computer memory.

3.4.1 Model Formulation

A MIP model is similar to a linear programming model except that some variables are constrained to take on integer values. In many cases, including this one, binary (0,1) variables are all that are needed. Using binary variables allows a consideration of disjunctive constraint sets of practically any shape. The mixed-integer program consists of a linear objective function (to be minimized) and a set of linear constraints. Some nonlinear terms (e.g. penalty functions) and constraints (e.g. reservoir outlet capacities) are approximated with piecewise linear functions.

3.4.2 General Objective Function and Constraints

Following the formulation by Watkins et al. (1999), a non-convex objective function similar to that shown in Figure 7 can be modeled as follows:

$$\text{Min} \quad \sum_{\ell=1}^{N} c_\ell f_\ell \tag{1}$$

subject to

$$\sum_{\ell=1}^{2} f_\ell \geq Y \sum_{\ell=1}^{2} f_\ell^{\max} \tag{2}$$

$$f_3 \leq Y\left(f_3^{\max}\right) \tag{3}$$

$$0 \leq f_\ell \leq f_\ell^{\max} \qquad \ell = 1, \ldots, N \tag{4}$$

$$Y \in \{0,1\} \tag{5}$$

in which N is the number of flow zones, f_ℓ is the flow in zone ℓ, $f_\ell^{\max}$ is the capacity of zone ℓ, and c_ℓ is the unit cost of flow in zone ℓ. Here Y is a binary variable indicating whether the flow is in zones 1 or 2 or in zone 3. If $Y = 1$, then Eq. (2) requires that flow zones 1 and 2 be filled, and Eq. (3) allows flow in zone 3. If $Y = 0$, then Eq. (2) is redundant, but Eq. (3) prevents flow in zone 3. This assures that flow zones fill in the correct order.

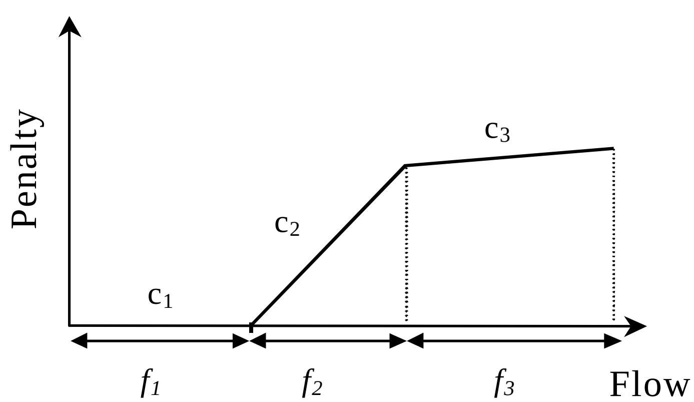

Figure 7. Piecewise linear approximation of a non-convex penalty function.

3.4.3 Reservoir Outlet Rating Curves

Binary variables may also be used to represent non-convex constraint sets. The idea in doing so is to consider disjunctive constraint sets—only one of a pair (or more generally, *k* of *m*) must hold—each of which is convex. Binary variables are used to indicate which convex set is "active." Consider the following constraints (for a single time period), along with Figure 5:

$$R \le \sum_{\ell=1}^{3} b_{\ell} S_{\ell} \tag{6}$$

$$\sum_{\ell=1}^{2} S_{\ell} \ge Y \sum_{\ell=1}^{2} S_{\ell}^{\max} \tag{7}$$

$$S_3 \le Y\left(S_3^{\max}\right) \tag{8}$$

$$0 \le S_{\ell} \le S_{\ell}^{\max} \qquad \ell = 1,\ldots,N \tag{9}$$

$$Y \in \{0,1\} \tag{10}$$

Here R is the release from the reservoir, S_ℓ is the storage in zone ℓ, and $S_\ell^{\max}$ is the storage capacity of zone ℓ. If $Y = 0$, then the region formed by S_1 and S_2 is active, and the storage in zone 3 is limited by Eq. (8) to be zero. If $Y = 1$, then the region formed by S_3 is active. In this case, Eq. (7) requires storage zones 1 and 2 to be filled.

3.4.4 Flow Over a Weir

Flow over a weir can be constrained in essentially the same way as discharge from a reservoir. In the case of uncontrolled flow, an equality constraint is used rather than an inequality. Consider the following constraints (for a single time period), along with Figure 6:

$$D = \sum_{\ell=1}^{3} a_{\ell} f_{\ell} \tag{11}$$

$$f_1 \ge Y\left(f_1^{\max}\right) \tag{12}$$

$$\sum_{\ell=2}^{3} f_{\ell} \le Y \sum_{\ell=2}^{3} f_{\ell}^{\max} \tag{13}$$

$$0 \le f_{\ell} \le f_{\ell}^{\max} \qquad \ell = 1,\ldots,N \tag{14}$$

$$Y \in \{0,1\} \tag{15}$$

Here D is the flow over the weir, f_ℓ is flow in zone ℓ of the main channel, and $f_\ell^{\max}$ is the flow capacity of zone ℓ in the main channel. If $Y = 0$, then by Eq. (13) there is no flow in zones 2 or 3 of the main channel. If $Y = 1$, then Eq. (12) requires flow zone 1 to be at capacity. So long as the "main channel" is defined such that diverting flow to a bypass always leads to a reduction in flood damages, these constraints will ensure that the flow zones fill in the proper order.

3.4.5 Reservoir Continuity and Capacity Constraints

A continuity constraint for each reservoir in each time period is needed. The general form of this constraint for reservoir i, time period j, is

$$\frac{1}{\Delta t}\left[S_{i,j} - S_{i,j-1}\right] + R_{i,j} - \sum_{k,k\in\Omega} \sum_{t=1}^{j} g_{t,k}\, f_{t,k} = I_{i,j} \qquad (16)$$

where $S_{i,j-1}$ and $S_{i,j}$ = storage at the beginning and end of period j, respectively; $R_{i,j}$ = total release in period j; Ω = set of all control points upstream of i from which flow is routed to i; $f_{t,k}$ = average flow at control point k in period t; $g_{t,k}$ = linear coefficient to route period t flow from control point k to control point i for period j; $I_{i,j}$ = unregulated inflow to the reservoir during period j. Linear routing coefficients may be input directly or the model can compute them from given Muskingum coefficients.

3.4.6 Storage Zones

To model desired operating policies, including storage-balancing schemes among reservoirs, the total storage capacity of each reservoir in the system may be divided into storage zones. Then the total storage at any time j is the sum of storage in these zones:

$$S_{i,j} = \sum_{\ell=1}^{NLF} S'_{i,j,\ell} \qquad (17)$$

Here ℓ = index of the storage zone and NLF = number of zones. Substituting this relation into the continuity equation yields

$$\frac{1}{\Delta t}\left[\sum_{\ell=1}^{NLF} S'_{i,j,\ell} - \sum_{\ell=1}^{NLF} S'_{i,j-1,\ell}\right] + R_{i,j} - \sum_{k,k\in\Omega} \sum_{t=1}^{j} g_{t,k}\, f_{t,k} = I_{i,j} \qquad (18)$$

where the storage in each zone ℓ is constrained as

$$S'_{i,j,\ell} \le SMAX_{i,\ell} \qquad (19)$$

3.4.7 Control Point Continuity Constraints

A continuity constraint is included for each control point for each time period. A control point is any point other than a reservoir where water enters or leaves the system or where information about flow is desired. This constraint takes the following general form for each control point i in period j:

$$f_{i,j} - \sum_{k,k\in\Omega} \sum_{t=1}^{j} g_{t,k}\, f_{t,k} = I_{i,j} \qquad (20)$$

Here $f_{i,j}$ = the average control-point flow during period j; $I_{i,j}$ = local inflow during period j; $g_{t,k}$ = linear routing coefficients from point k to point i.

3.4.8 Discharge Zones

To model system operating priorities, the discharge at each control point may be divided into discharge zones. The control point continuity equation then takes the form

$$\sum_{\ell=1}^{NF} f_{i,j,\ell} - \sum_{k,k\in\Omega} \sum_{t=1}^{j} g_{t,k}\, f_{t,k} = I_{i,j} \tag{21}$$

where ℓ = index of discharge zone and NF = number of discharge zones.

3.4.9 Penalty for too much or too little Storage

Penalties in this category quantify the desire to avoid storage outside an acceptable range. This might include a desire to retain flood storage capacity for a possible future flood or, ultimately, a desire to avoid storage levels that might threaten the dam's structural integrity. The penalty is specified for each reservoir as a piecewise linear function of the volume of water stored in the reservoir during the period. The total penalty for storage, *SP*, is defined as

$$SP_i = \sum_{j=1}^{T} \sum_{\ell=1}^{NLF} c_{i,l}^{S}\, S_{i,j,\ell} \tag{22}$$

where $c_{i,l}^{S}$ is the slope of the storage penalty function in zone ℓ of reservoir ***i***.

3.4.10 Penalty for changing Release too rapidly

Penalties in this category quantify the negative impact of varying releases too quickly from one period to the next. Such rapid variations may be unacceptable if they would cause bank sloughing downstream or if they would allow insufficient time for evacuation. To impose this penalty, the LP model, through a set of auxiliary constraints, segregates the release for each period into the previous period's release plus or minus a change in release. If the absolute value of this change in release exceeds a specified maximum, a penalty is imposed; otherwise there is no penalty.

The auxiliary constraints relate the release for each period to the release in the previous period by the equation

$$R_{i,j} = R_{i,j-1} + R_{i,j}^{+} - R_{i,j}^{-} \tag{23}$$

where $R_{i,j}^{+}$ = the total increase in release from period *j*-1 to period *j*; and $R_{i,j}^{-}$ = the total decrease in release from period *j*-1 to period *j*. If $R_{i,j} \geq R_{i,j-1}$, then $R_{i,j}^{+}$ is positive and $R_{i,j}^{-}$ is zero. If $R_{i,j} \leq R_{i,j-1}$, then $R_{i,j}^{-}$ is positive and $R_{i,j}^{+}$ is zero. If $R_{i,j} = R_{i,j-1}$, then both $R_{i,j}^{+}$ and $R_{i,j}^{-}$ are zero.

To define allowable increases and decreases, $R_{i,j}^{+}$ and $R_{i,j}^{-}$ are partitioned into a portion that is acceptable and a portion that is excessive using the following relationships:

$$R_{i,j}^{+} = Ra_{i,j}^{+} + Re_{i,j}^{+}$$

$$R_{i,j}^{-} = Ra_{i,j}^{-} + Re_{i,j}^{-}$$

Here $Ra^+_{i,j}$, $Re^+_{i,j}$ are the acceptable and excessive release increase, respectively; and $Ra^-_{i,j}$, $Re^-_{i,j}$ are the acceptable and excessive release decrease, respectively. Thus, the current release can be defined as

$$R_{i,j} = R_{i,j-1} + \left[Ra^+_{i,j} + Re^+_{i,j}\right] - \left[Ra^-_{i,j} + Re^-_{i,j}\right] \qquad (24)$$

Thus $Ra^+_{i,j}$ and $Ra^-_{i,j}$ are constrained not to exceed the desired limits, and a penalty, *RP*, is imposed on $Re^+_{i,j}$ and $Re^-_{i,j}$ at reservoir *i* as

$$RP_i = \sum_{j=1}^{T} c_i^{r+} Re^+_{i,j} + \sum_{j=1}^{T} c_i^{r-} Re^-_{i,j} \qquad (25)$$

where c_i^{r+} is the penalty per cfs for an excessive increase in release rate and c_i^{r-} is the penalty per cfs for an excessive decrease in release rate.

3.4.11 Penalty for too much or too little Flow at Control Points (in each time step)

Penalties in this category quantify the desire to avoid downstream flows outside an acceptable range. The penalties are specified as piecewise linear functions of downstream flow, which is the sum of local runoff and routed reservoir releases. The total penalty for flow, *QP*, at location *i* is

$$QP_i = \sum_{j=1}^{T} \sum_{\ell=1}^{NF} c^f_{i,l}\, f_{i,j,\ell} \qquad (26)$$

where $c^f_{i,l}$ is the slope of the penalty function in flow zone l at control point *i*.

3.4.12 Peak Flow Penalty

Peak flow penalties, $\overline{QP}_i$, are assigned to the single largest flow, $\overline{f}$, in each flow zone l at control point *i* in the form

$$\overline{QP}_i = \sum_{l=1}^{NF} c^{\bar{f}}_{i,l}\, \overline{f}_{i,l} \qquad (27)$$

$$\sum_{l=1}^{NF} \overline{f}_{i,l} \geq \sum_{l=1}^{NF} f_{i,j,l} \qquad \forall i, j \qquad (28)$$

where $c^{\bar{f}}_{i,l}$ is the slope of the peak flow penalty function in flow zone l at control point *i*.

3.4.13 Flood Control Objective Function

The total penalty, TP, is defined as a function of releases, storage levels, and flows throughout the system for the entire period of analysis. The complete objective function is

$$\min TP = \left[\sum_{i,i\in\Psi}\left(QP_i + \overline{QP}_i\right) + \sum_{i,i\in\Phi}\left(RP_i + SP_i\right)\right] \qquad (29)$$

where Ψ = set of all damage centers and Φ = set of all reservoirs. The operating schedule that minimizes the value of this function is considered the optimal schedule.

3.4.14 Computational Considerations

Although MIP models allow much greater flexibility than do LP models, solving large MIP problems may require orders of magnitude more computation time. Using standard branch-and-bound codes, in which an LP relaxation is solved at each node (see Figure 8), computational expense may increase at a rate as high as 2^n, where n is the number of binary variables. Although the proposed formulation attempts to minimize the number of binary variables used (just one per reservoir or weir per time period), large systems analyzed over numerous time periods may require an excessive number of binary variables. Solving practical flood control optimization problems using MIP models requires care in model formulation and logical preprocessing to fix as many binary variables as possible. Some methods used to reduce the amount of computations are discussed in Chapter 4.

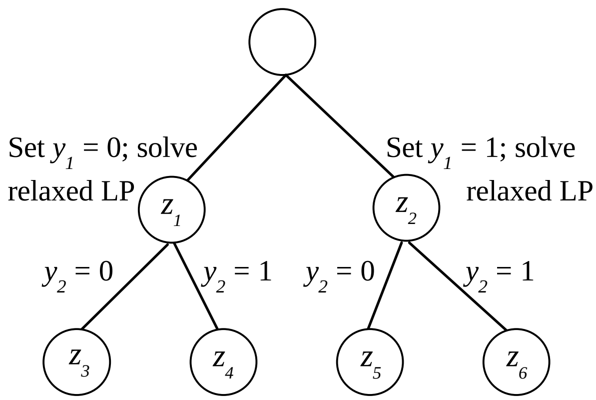

Figure 8. Branch-and-bound method for mixed-integer programming.

Chapter 4
MODEL APPLICATION

The computer program used in this study, HEC-FCLP (Flood Control Linear Program), is a generalized program that formulates the multi-reservoir flood control problem as a linear or mixed-integer linear programming model. The program input is similar to that of HEC-5, a general-purpose reservoir simulation program (USACE, 1998), with the addition of penalty function data. System inflow data and initial conditions are read from an HEC Data Storage System file (HEC-DSS) (USACE, 1995). Time series results are written to a text file and to HEC-DSS. HEC-FCLP runs on PC-DOS computers and requires the IBM Optimization Subroutine Library (OSL, 1995).

4.1 SACRAMENTO BASIN MODEL

The Sacramento system model considers a network of nodes and links. These are shown in Figure 9. A node in the network can represent a reservoir, a junction where two or more flows converge, a weir location, a gage location, and/or a potential damage location. If a node represents a damage location, then there will be a penalty function related to the flow leaving that node. This penalty function represents the relation between the flow at the node (or along the downstream reach) and the corresponding damage caused by that flow. The links of the network represent the means by which water is conveyed between nodes. Links represent rivers, stream channels, diversions, or bypasses. Arrows in the schematic represent locations of incremental inflows to the system. Five flood control reservoirs and six diversion locations are used in the Sacramento system. The model input used in this study, which includes all values used to define constraints and penalty functions in the system, is given in Appendix A along with a description of the program input.

4.1.1 Outlet Rating Curves

Reservoir outlet rating curves were approximated in the model using piecewise linear functions. Points were chosen on the curve to correspond to the storage at reservoir operating levels (e.g., top of conservation, top of flood pool). In a few instances it was necessary to add additional points, called match points, to define the rating curve better.

4.1.2 Diversion Functions

For the Sacramento Basin model, diversion functions at Ord Ferry, Moulton Weir, and Colusa Weir were provided by the U.S. Army Corps of Engineers Sacramento District. Diversion functions were derived for Tisdale Weir, Fremont Weir, and the Sacramento Weir using historic spill and flow data.

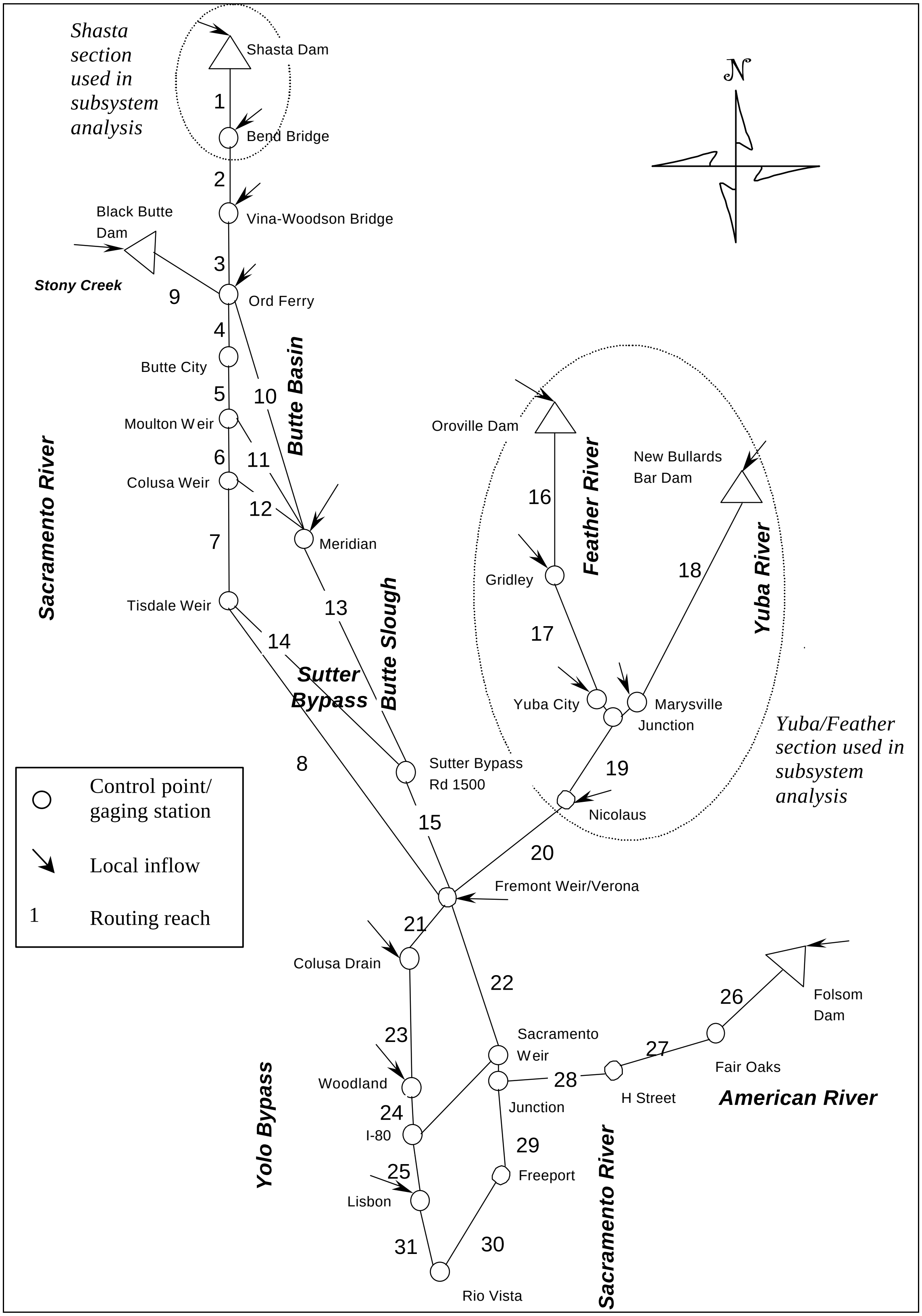

Figure 9. Schematic of Sacramento Basin.

4.2 PENALTY FUNCTIONS

The objective function of the optimization model comprises five different types of penalty functions. These include three types of flow-based penalty functions, storage-based penalty functions, and change-in-release penalty functions.

4.2.1 Peak Flow Penalties

Peak flow penalties are assigned to the maximum flow at a control point during an event. For primarily urban impact areas, the damage caused by the peak flow is generally representative of the total damage incurred.

4.2.2 Duration Flow Penalties

Duration flow penalties are applied to excessively high flow levels in each time step and are used primarily to encourage the model to reduce flow rates as soon as practical following the peak flow. These penalties are cumulative. Duration-based flow penalties may also be used to represent agricultural damage whenever crops are sensitive to the duration of inundation.

4.2.3 Minimum Flow Penalties

Minimum flow penalties are assigned to some control points immediately downstream of reservoirs. These are used to encourage the model to maintain minimum release rates from the dams. Minimum-flow penalties could also be used to represent environmental objectives.

4.2.4 Storage Penalties

Storage penalties are used to represent the aversion of reservoir operators to deviating from the top of conservation storage when future inflow volumes are uncertain. Like the flow duration penalties, these are summed over all time periods. For computational purposes the storage penalties are assigned to the same reservoir operating levels (zones) that are used to define the outlet rating curves.

4.2.5 Change-in-release Penalties

Change-in-release penalties are assigned to the reservoirs to prevent the model from increasing or decreasing the amount of release by more than a specified amount in each time step. In general, the penalties are made sufficiently large to prevent the change-in-release limits from being violated.

4.2.6 Damage Functions

Potential damage estimates for the Sacramento basin were developed by the District based on the Federal Emergency Management Agency's (FEMA) 0.01 and 0.002 annual exceedence probability mapping and census tract data. Levee heights in the Sacramento system were assumed to be one foot above the 0.01 annual exceedence probability event and one foot below the 0.002 event. In areas without levees, the 0.01 and 0.002 flow magnitudes were derived from a Flood Impact Analysis (HEC-FIA) model of the system.

The penalty functions relating flow to damage (in dollars) were approximated with convex piecewise linear functions. The damage in each impact area was evaluated at the control point nearest the center of the impact area. If there were no control points immediately adjacent to a particular impact area, then the damage was assessed at the nearest upstream control point. The control points used in the Sacramento study, the impact areas that they represent, and the sum of potential damages for each event frequency are listed in Table 8. If an impact area could incur damage from more than one source, such as a bypass or river flow, then the damage potential for that impact area was assigned to both sources (i.e., two or more control points). However, owing to the limited spatial extent of the reservoir models, not all impact areas were assigned to control points. Figure 10 shows the location of the impact areas used in the analysis.

Figure 11 depicts an example damage function for a leveed area. The damage functions were developed so that little damage would be calculated until flow reached 98% of the top-of-levee flow. In some cases a small "persuasion" penalty was placed on flow in the zone just below the 98% flow point to discourage the model from operating above the nominal channel capacity or near the top-of-levee stage. For computational purposes, the penalty functions were assumed to be convex.

4.3 HYDROLOGIC DATA

Incremental flows and routing criteria were derived from observed flow and stage data. Table 9 lists the Muskingum routing coefficients used in the model. The number of sub-reaches (steps) in each routing reach were adjusted to be compatible with the 6-hour time step used in the optimization model. Appendix B describes the methods used to develop incremental inflow data at each of the model control points and routing information between control points.

Table 8. Sacramento Basin damage representation.

Control Point	Impact Areas Represented (from Figure 10)	Total .01 Exceedence Probability Damage (In $ millions)	Total .002 Exceedence Probability Damage (In $ millions)
Bend Bridge	SAC 1	$699	$1,361
Vina-Woodson	SAC 2	$93	$97
Ord Ferry	SAC 4A	$0	$25
Butte City	SAC 3+SAC 4B	$11	$26
Moulton Weir	SAC 4C	$33	$33
Colusa Weir	SAC 5A+SAC 5B+SAC 6	$123	$158
Butte Slough near Meridian	SAC 6+SAC 7+SAC 8A+ SAC 9	$0	$265
Tisdale Weir	SAC 5C+SAC 5D+SAC 9	$16	$80
Sutter Bypass Rd 1500	--	--	--
Gridley	SAC 10B	$0	$87
Yuba City	SAC 8A+SAC 8B+SAC 11A+ SAC 11B	$0	$2,380
Marysville	SAC 10A	$393	$393
Nicolaus	SAC 13A+SAC 13B	$0	$80
Fremont Weir/Verona	SAC 14+SAC 15+SAC 16+ SAC 19+SAC 20+SAC 22	$15	$2,377
Colusa Drain	--	--	--
Woodland	SAC 17A+SAC 17B+SAC 18+ SAC 20	$6	$13
I-80	--	--	--
Lisbon	SAC 21B+SAC 21D+SAC 29+ SAC 29B+ SAC 30+SAC 31	$5	$55
Fair Oaks	SAC 24	$197	$520
H St	SAC 23+SAC 25	$17,943	$19,525
Sacramento Weir	SAC 21B+SAC 21C+SAC 25+ SAC 31A	$13,735	$15,065
Freeport	SAC 26+SAC 27+SAC 28+ SAC 32+SAC 33+SAC 34+ SAC 35+ SAC 36+ SAC 37+ SAC 38	$118	$229
Rio Vista	SAC 40 + SAC 41	$5	$5

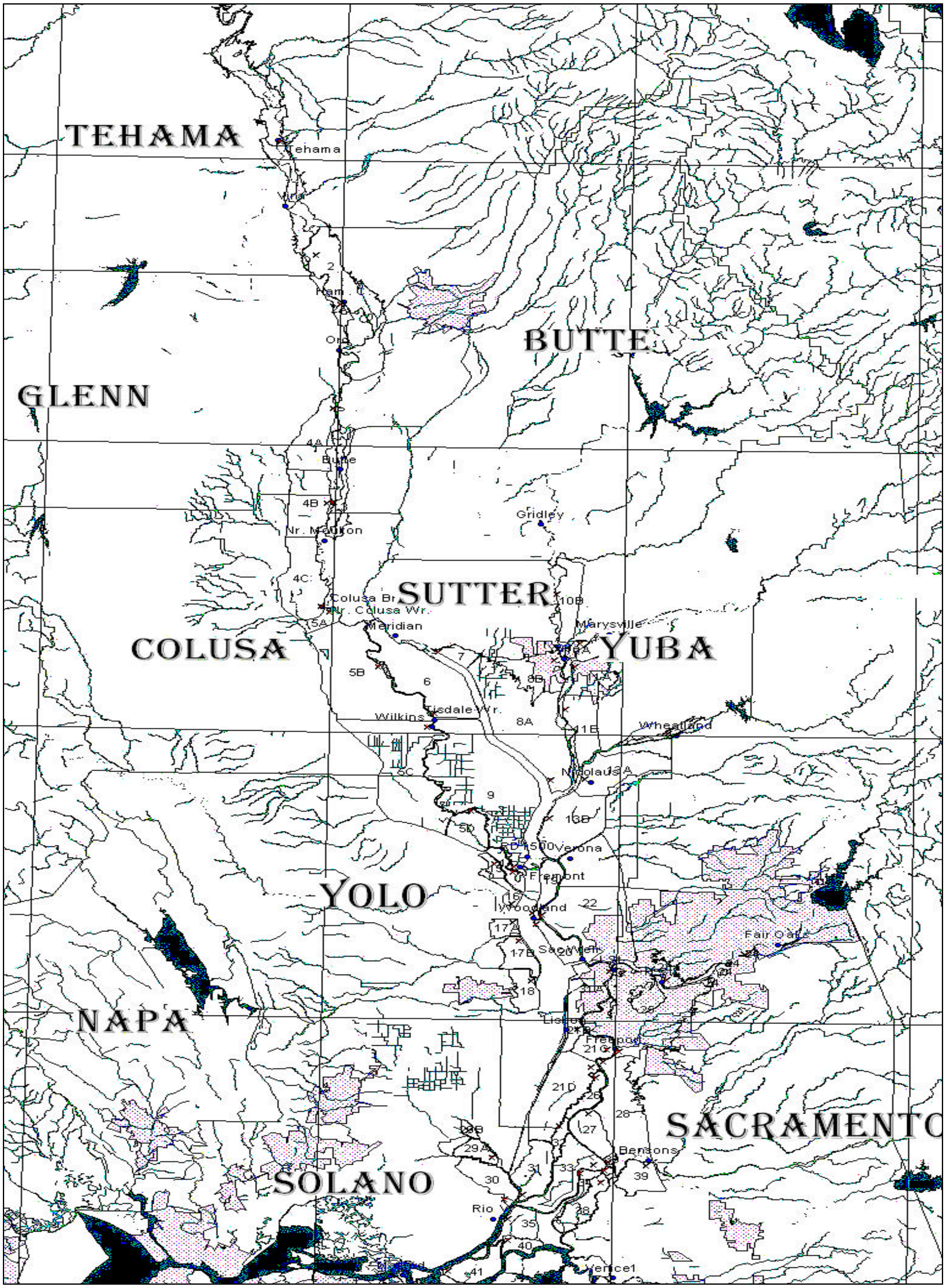

Figure 10. Map of Sacramento system impact areas.

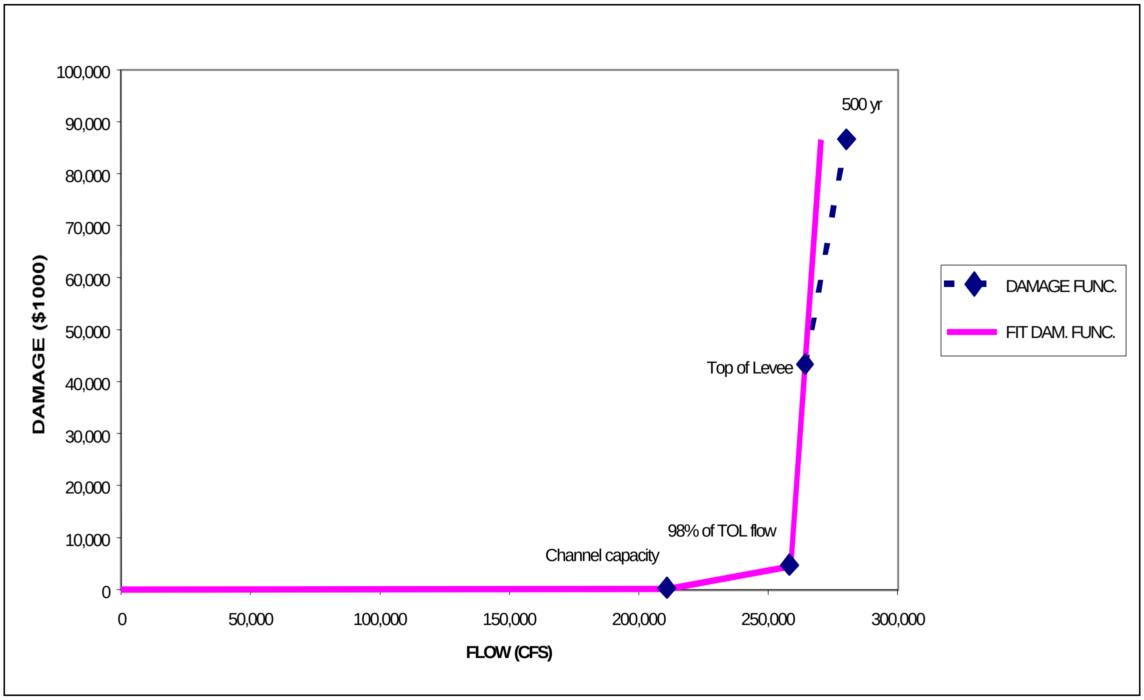

Figure 11. Example damage function.

Table 9. Sacramento model Muskingum routing coefficients (1 hour time step).

Reach #	Control Points	X	K (hrs)	Steps
1	Shasta Reservoir to Bend Bridge	0.1	3	4
2	Bend Bridge to Vina-Woodson Bridge	0.2	2.5	4
3	Vina-Woodson Bridge to Ord Ferry	0.15	2	4
4	Ord Ferry to Butte City	0.2	2	4
5	Butte City to Moulton Weir	0.2	2	4
6	Moulton Weir to Colusa Weir	0.2	1	1
7	Colusa Weir to Tisdale Weir	0.25	2	4
8	Tisdale Weir to Fremont Weir/Verona	0.38	1	8
9	Black Butte Reservoir to Ord Ferry	0.2	2	5
10	Ord Ferry to Butte Slough nr Meridian	0.1	4	10
11	Moulton Weir to Butte Slough near Meridian	0.1	4	5
12	Colusa Weir to Butte Slough near Meridian	0.1	4	4
13	Butte Slough near Meridian to Sutter Bypass Rd 1500	0.2	2	8
14	Tisdale Weir to Sutter Bypass Rd 1500	0.2	2	6
15	Sutter Bypass Rd 1500 to Fremont Weir/Verona	0.2	2	2
16	Oroville Reservoir to Gridley	0.2	2	4
17	Gridley to Yuba City/Junction	0.17	2	4
18	New Bullards Bar Dam to Marysville/Junction	0.15	2	4
19	Yuba City/Junction to Nicolaus	0.35	1	10
20	Nicolaus to Fremont Weir/Verona	0.2	2	2
21	Fremont Weir/Verona to Colusa Drain	0.2	2	3
22	Fremont Weir/Verona to Sacramento Weir	0.2	2	4
23	Colusa Drain to Woodland	0.2	2	1
24	Woodland to I-80	0.2	1	1
25	I-80 to Lisbon	0.2	2	3
26	Folsom Dam to Fair Oaks	0.4	1	2
27	Fair Oaks to H Street	0.2	2	2
28	H Street to Sacramento Weir/Junction	0.2	2	1
29	Sacramento Weir/Junction to Freeport	0.2	2.5	2
30	Freeport to Rio Vista	0.2	2	4
31	Lisbon to Rio Vista	0.2	2	8

4.4 MODEL CALIBRATION

Not all of the penalty coefficients in the flood control optimization model are based on economic data (i.e. storage and flow duration penalties). Therefore, a procedure is needed to "calibrate" the model, i.e., determine the magnitudes of the non-economic penalties. Two general approaches are available. The first approach, the one taken in this study, involves the trial-and-error adjustment of storage, change-of-release, and flow persuasion penalty coefficients to obtain results that are reasonably similar to observed values for one or more historic flood events. The second approach involves a more rigorous calculation of the magnitudes of non-economic penalties that are needed to ensure that particular operational priorities are always met.

The Sacramento Basin model was calibrated by adjusting the storage and minimum-flow penalties until the model operation matched the historic operation reasonably well. To reflect current operating procedures, the model was calibrated using the 1997 event. Figure 12 shows the final storage penalty function for Folsom Dam, and Figure 13 and Figure 14 show calibration results for the reservoir. The storage penalty functions for the remaining reservoirs are listed in the model input in Appendix A. Following the trial-and-error procedure, the magnitudes of the non-economic penalties can be considered to be representative of the preferences and risk aversion of the operators. A limitation of this approach is that, with only a small amount of observed data available for calibration, the model may give unrealistic results for flood events of greater or lesser magnitude than the event for which it is calibrated.

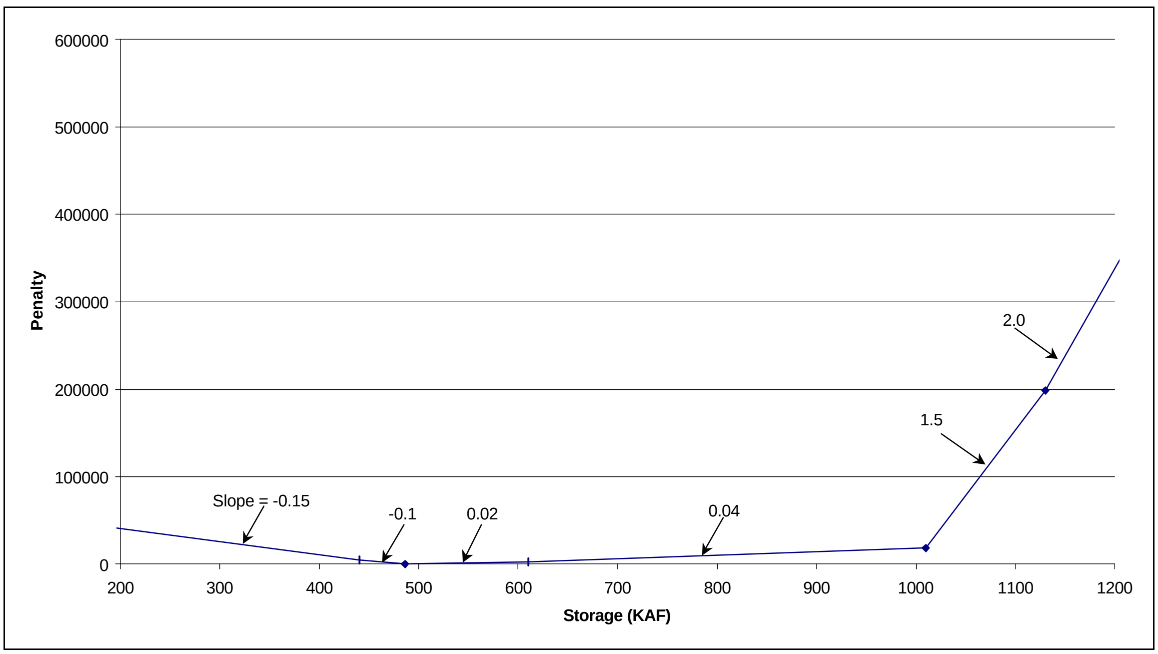

Figure 12. Storage penalty for Folsom Dam.

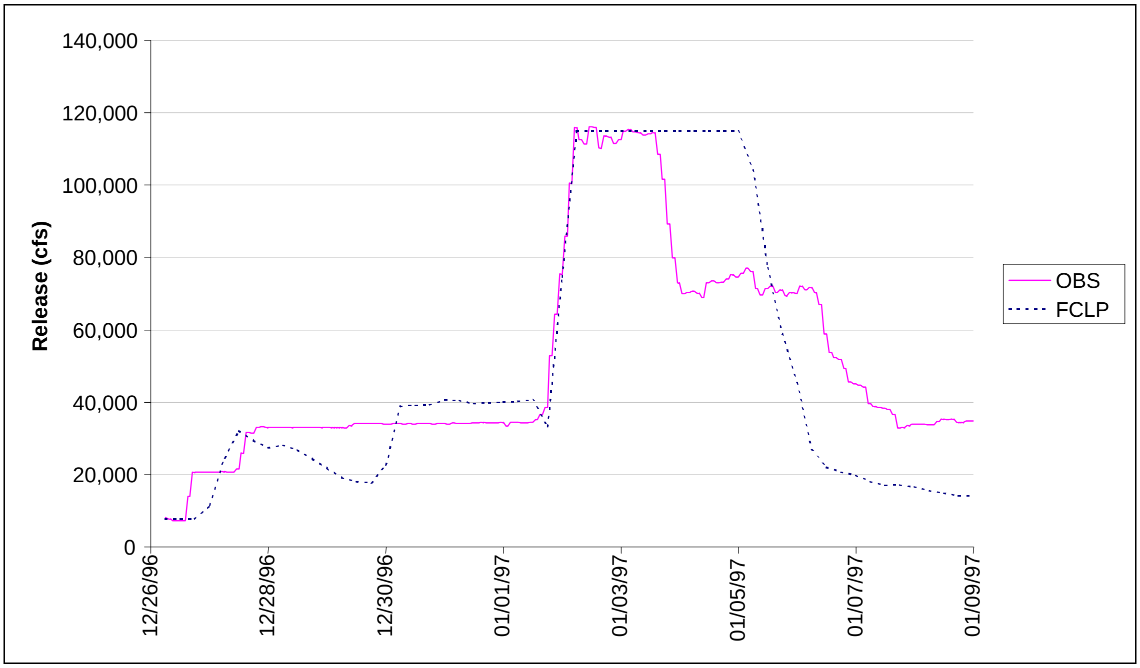

Figure 13. Model and observed releases from Folsom Dam (1997 event).

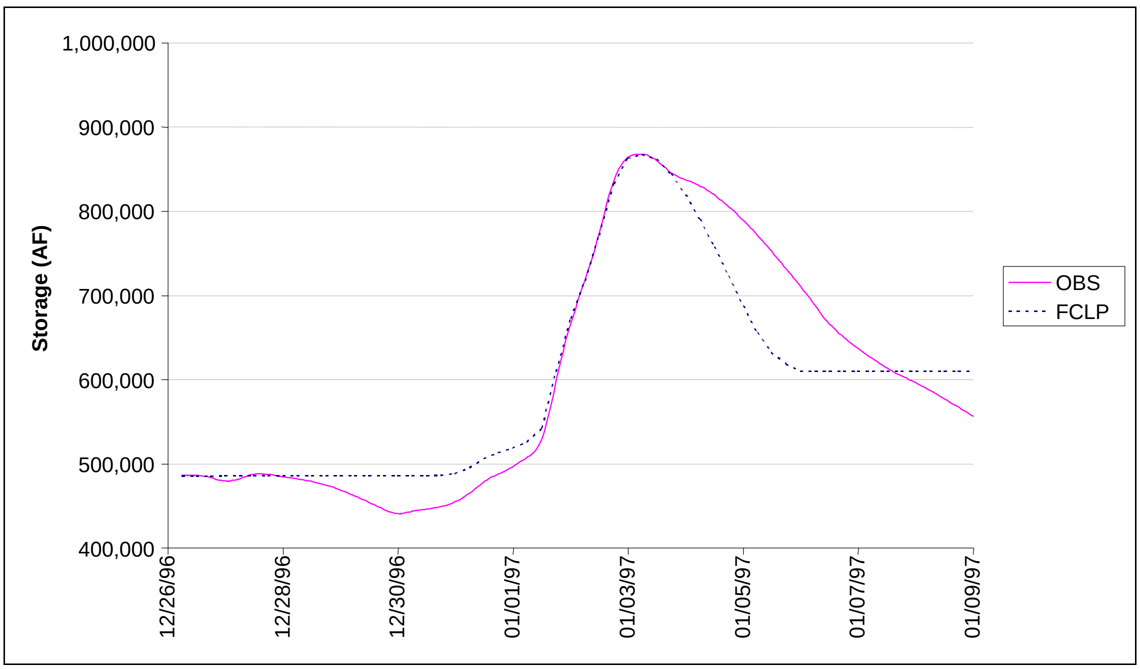

Figure 14. Model and observed storage levels in Folsom Reservoir (1997 event).

The alternative approach to model calibration involves a more rigorous calculation of the magnitudes of penalties that are needed to ensure that particular operational priorities are always met. For example, consider the following flood operation priorities for Folsom Dam:

1. Keep reservoir storage below the top of the spillway design pool (1,130 KAF).
2. Keep flows below the following damaging levels (whichever is limiting): 115,000 cfs at Fair Oaks; 197,000 cfs at H Street; and 260,900 cfs at the Sacramento Weir.
3. Keep reservoir storage below the top of the flood gross pool (610 KAF).
4. Keep all downstream flows within channel design capacities.
5. Keep reservoir storage at the top of the conservation pool (486 KAF).

For these priorities always to be met by the optimization model, unless other constraints become binding (such as reservoir outflow capacity), the storage penalty coefficients for Folsom Dam and the persuasion penalty coefficients for flows downstream must meet certain numerical criteria (Israel and Lund, 1999).

One criterion pertains to situations where reducing the downstream flow below a certain level has higher a priority than reducing reservoir storage below a certain level, as in Priorities 4 and 5 above. In this case, the unit penalty on the downstream flow, P_4, and the corresponding storage penalty, P_5, must have values such that

$$P_4 > T \times P_5 \tag{30}$$

where T is the number of time periods in the analysis. The reason that P_5 is multiplied by T is that a unit of water held in storage may incur a penalty in each time step, whereas the same unit of water may incur a particular flow penalty only once when it is released. The factor T need not equal the total number of time periods but only the maximum number of consecutive time periods in which water can be stored in the corresponding reservoir zone. This factor may also be reduced if a fraction of the reservoir release is diverted before it reaches the downstream reach.

Another criterion pertains to situations in which reducing reservoir storage has higher priority than reducing downstream flows, as in Priorities 3 and 4 above. In this case the storage penalty P_3 must be greater than the sum of corresponding flow penalties downstream. For J locations downstream with Priority Level 4, the storage penalty coefficient must satisfy

$$P_3 > \sum_{j=1}^{J} P_4^j \tag{31}$$

P_3 could be reduced if a fraction of the release is diverted before reaching one or more of the downstream locations.

For Priorities 2 and 3 in the example, criterion (30) would again hold. If the flow penalty coefficients P_2^j were to be defined by actual economic data, then these values would provide a starting point for the calibration procedure. For Priority 1, criterion (31) would be applied next. The highest priority in the optimization model, to prevent overtopping of the dam, is enforced as constraint.

A conceptual limitation of this approach is that the model no longer has freedom to choose what the operational priorities should be because the existing priorities are already built into the penalty functions. A practical limitation is that application of criteria (30) and (31), particularly to large reservoir systems, could lead to a wide range in the penalty coefficient values, which may cause numerical instability in the linear programming solution procedure.

Ideally, for an optimization model in which minimization of flood damage is the sole objective, there would be little or no need for non-economic penalties. The need for non-economic penalties arises primarily from the fact that the deterministic approach does not explicitly model the uncertainty in inflows, nor does it accurately model duration effects such as the saturation of levees. Of course, non-economic penalties are also needed for environmental goals, such as minimum in-stream flows.

4.5 SOLUTION STRATEGY

Modeling the Sacramento Basin system with 64 six-hour time steps leads to a mixed-integer linear programming problem with approximately 10,000 continuous variables, 600 integer (0-1) variables, and 6,000 constraints. The integer variables are used to model non-convex relations between flow and damage, between storage and release capacity, and between channel flow and weir flow. The continuous problem (with the integer requirements relaxed) can be solved in just a few minutes using a 400 MHz Pentium II processor with 128 MB of RAM. However, with the integer requirements, the complexity of the problem increases at a rate up to 2^n, where n is the number of 0-1 variables. Therefore, it is imperative that the number of 0-1 variables be kept to a minimum. To do this, convex functions are used for all flow-damage relationships, and piecewise linear functions leading to convex feasible regions are used for storage-release capacity and flow-diversion relations whenever it is determined that model results will not be affected by these approximations.

The FCLP program also reduces the number of binary variables considered. First, the program uses simulation results to fix nearly 2/3 of the binary variables (approximately 400). OSL's MIP preprocessing routine then fixes nearly 1/6 more binary variables (approximately 100), leaving only about 100 binary variables in the branch-and-bound tree. The MIP model is solved in approximately 30 minutes using a 400 MHz Pentium II processor with 128 MB of RAM.

4.5.1 Outlet Rating Curves

Integer (0-1) variables are required to model forced spills from reservoirs. It was possible to eliminate some of the 0-1 variables by assuming a reservoir would not spill in optimal operation if it had not spilled in historic operation. Generally, the rating curve for a reservoir that does not spill can be modeled as a simple concave function, which leads to a convex feasible region for releases. Based on 1995 and 1997 operations, the maximum observed storage levels were used to determine if a reservoir had spilled during historic operation. Based on this information, it was determined that the rating curves for both Black Butte Dam and Oroville Dam could be simplified. Figure 15 shows the outlet rating curve for Black Butte Dam, the limits of historic operation, and two piecewise linear approximations. The more accurate concave-convex approximation can be used if computer computation time is not a concern and/or if more severe flood events are to be analyzed.

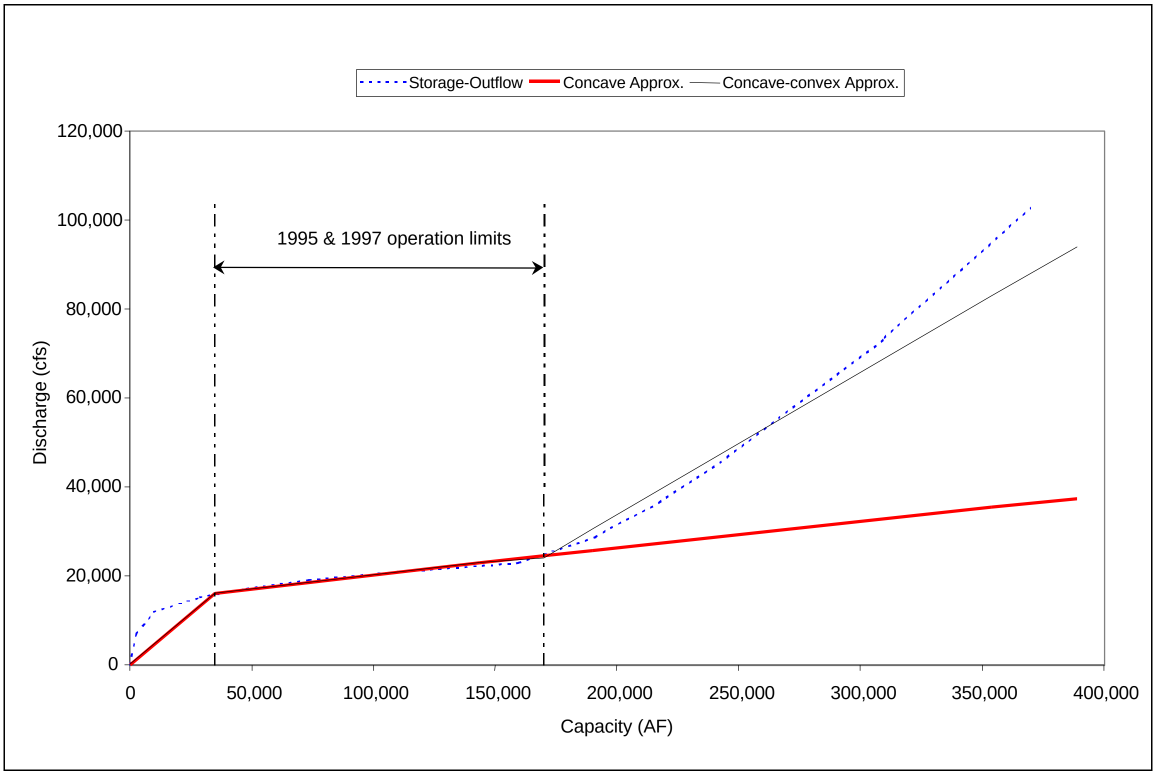

Figure 15. Black Butte Dam outlet rating curve.

4.5.2 Diversion Functions

Diversions in the Sacramento system were modeled by using piecewise linear approximations to the actual spill relationships. The diversions were fit with linear approximations based on the maximum historical spills during the 1995 and 1997 events. Figure 16 shows an example diversion function and a piecewise linear approximation. Assuming the purpose of the diversion is to reduce flood damage, a 0-1 variable is needed for the convex function to assure that the flow zones fill in the proper order, as described earlier.

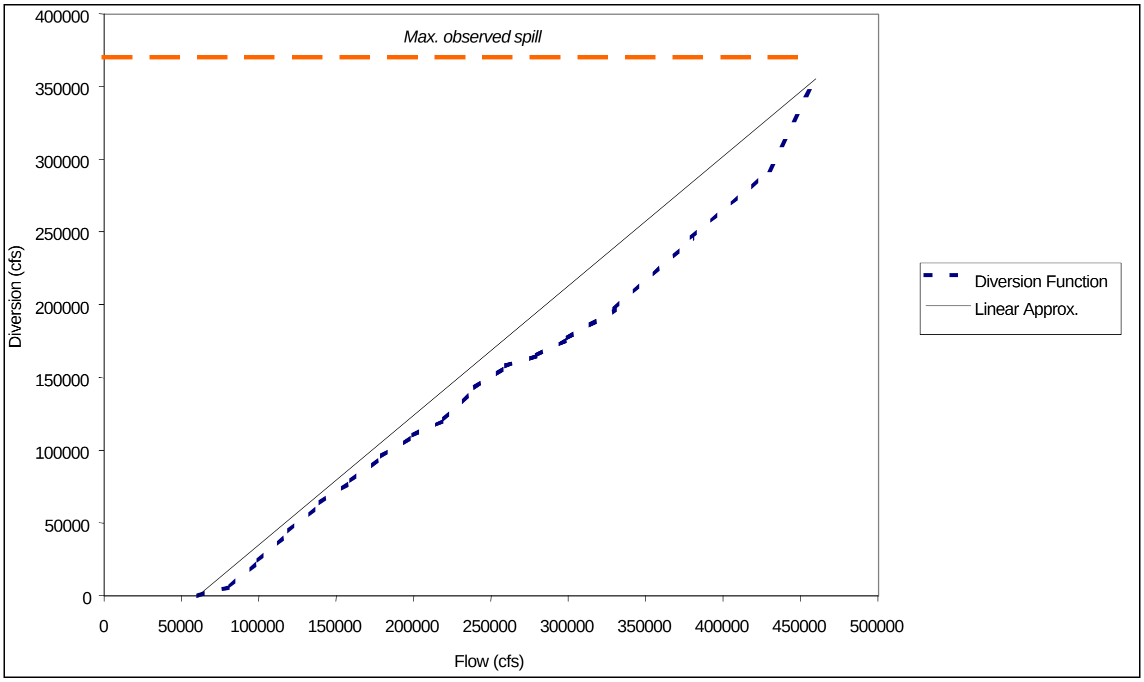

Figure 16. Fremont Weir diversion function.

Chapter 5

1997 MODEL RESULTS AND DISCUSSION

Figures 17 to 52 present the model results and observed data for the 1997 event. Figure 18 shows that the model makes higher releases from Shasta Dam at the beginning of the event to increase the amount of storage available for the upcoming inflows. This may not have been done in actual operation due to the limited foresight of the operators. The model begins reducing releases on December 29 and does not significantly increase releases until January 3. This corresponds with the period of highest precipitation, as described in Chapter 1. Once the inflow begins to subside, the model begins making higher releases to bring the reservoir storage down to the top of the conservation level at 3,250,900 AF. The model did not exceed the gross pool storage of 4,552,000 AF and was able to keep the peak discharge near the channel capacity of 100,000 cfs at Bend Bridge (Figure 17 and Figure 19). Current flood control procedures state that Bend Bridge is the farthest downstream control point for which Shasta Dam operates (USACE 1977). It appears, however, that the model also reduces the peak flow at Vina-Woodson approximately 40 miles downstream of Bend Bridge (Figure 20).

Operation of Black Butte Dam is shown in Figure 21 and Figure 22. Model releases from Black Butte Dam are highest during the period of highest inflow. The model is able to keep the reservoir storage from exceeding the gross pool storage of 143,676 AF. A comparison of Figure 25 and Figure 26 shows there is a discrepancy between the observed flow leaving Ord Ferry and that entering Butte City. It was assumed during model development that the incremental inflow to control points within leveed reaches would be negligible. It appears, however, that the incremental inflow to Butte City does have a significant effect on flow levels within this reach and should not be neglected in future models.

The operation of Oroville Dam is shown in Figure 34 and Figure 35. The model makes higher releases from Oroville Dam after the inflow begins to subside. In historical operation, the highest releases correspond to the period of highest inflow. As mentioned before, this is due in part to the limited foresight of the operators. The model is able to reduce the peak flow at Gridley, as shown in Figure 36. Although the model peak flow at Yuba City is slightly higher than the historic flow, neither exceeds the listed channel capacity of 210,000 cfs (Figure 37). Figure 41 shows that the coordinated operation of Oroville Dam and New Bullards Bar Dam produces a reduction in peak flow at the Nicolaus control point. Flows at Fremont Weir/Verona, however, appear to be little affected by the reservoir operations, as shown in Figure 42 and Figure 43.

The model operation of Folsom Dam, shown in Figure 46 and Figure 47, is very similar to historic operation. This is due in part to the limited capacity of the outlet works. The model does not exceed the channel capacity of 115,000 cfs, although the historic operation is slightly higher. At the Sacramento Weir, shown in Figure 49, the computed spill is significantly higher than the observed spill. This is due to the model assumption that the weir gates were fully open throughout the entire flood event. In practice, the operators may not open the weir gates until a specified flow level has been reached.

Although there is no historic data available for the flow at Rio Vista due to the tidal influence there, Figure 52 shows that the flow could be kept below the design capacity of 579,000 cfs. A summary of channel capacities, along with 1997 model and historic peak flows at various locations throughout the system, is listed in Table 10.

Table 10. Summary of 1997 results.

Control Point	Channel Capacity (cfs)	Observed Peak (cfs)	Model Peak (cfs)
Bend Bridge	100,000	121,070	100,951
Vina-Woodson	260,000	154,000	140,986
Ord Ferry	160,000	118,332	121,478
Butte City	160,000	146,520	120,795
Moulton Weir	160,000	119,699	96,518
Colusa Weir	60,000	58,204	44,016
Tisdale Weir	30,000	40,882	27,928
Yuba City	210,000	165,721	172,764
Marysville	120,000*	143,880	145,000
Nicolaus	320,000	319,133	299,418
Fair Oaks	115,000	116,650	115,000
Sacramento (I st.)	110,000	107,520	96,664
Freeport	110,000	114,900	96,597
Woodland	377,000	396,550	408,158
Lisbon	490,000	460,394	406,957
Rio Vista	579,000	N/A	496,979

* 180,000 cfs when flow on Feather R. is low

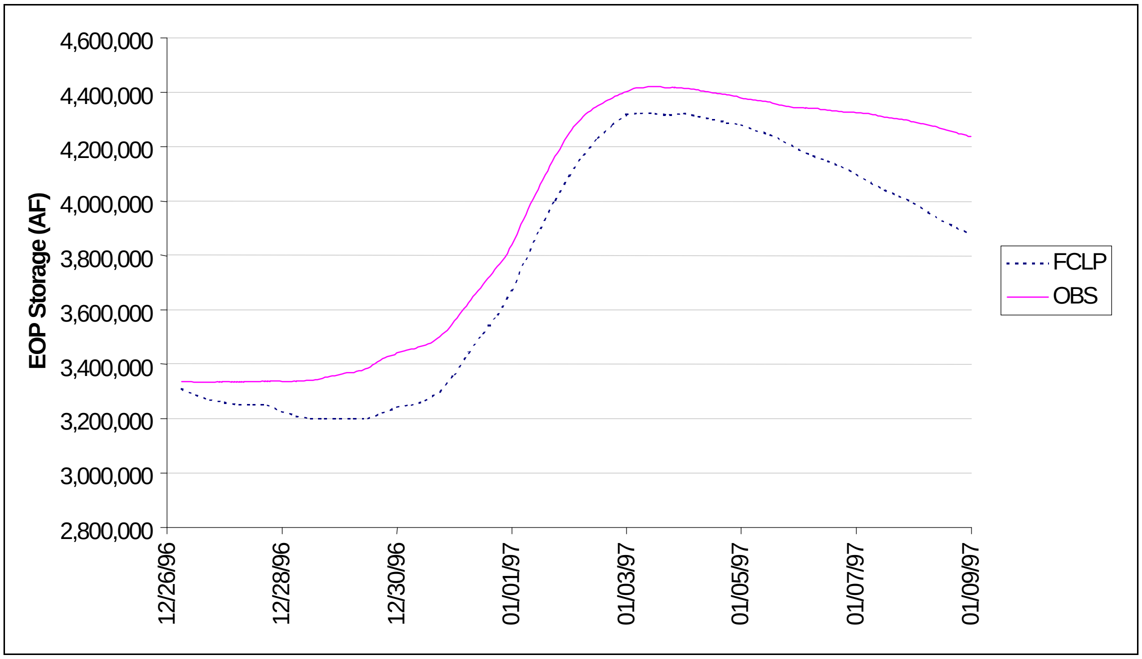

Figure 17. Shasta Dam EOP storage.

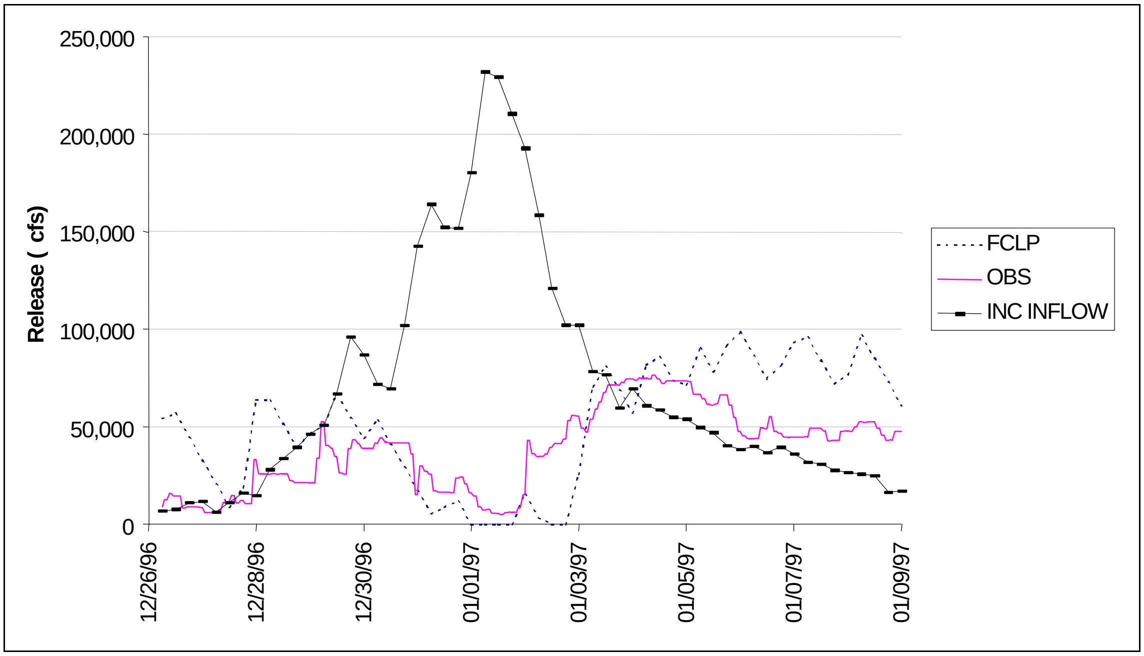

Figure 18. Shasta Dam release.

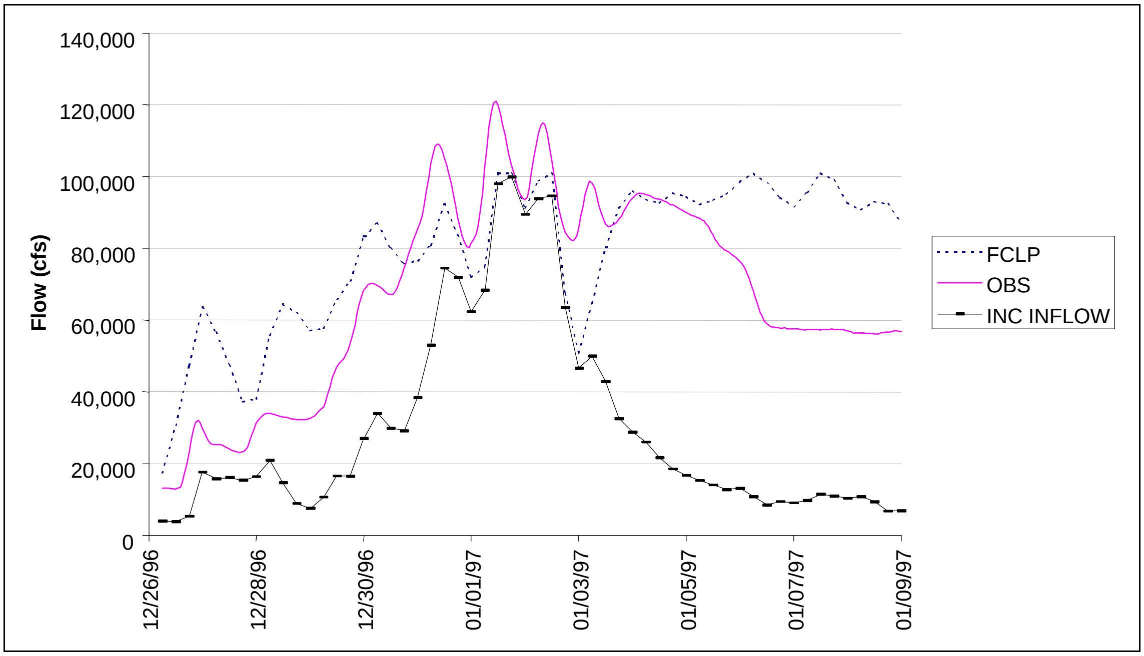

Figure 19. Flow at Bend Bridge.

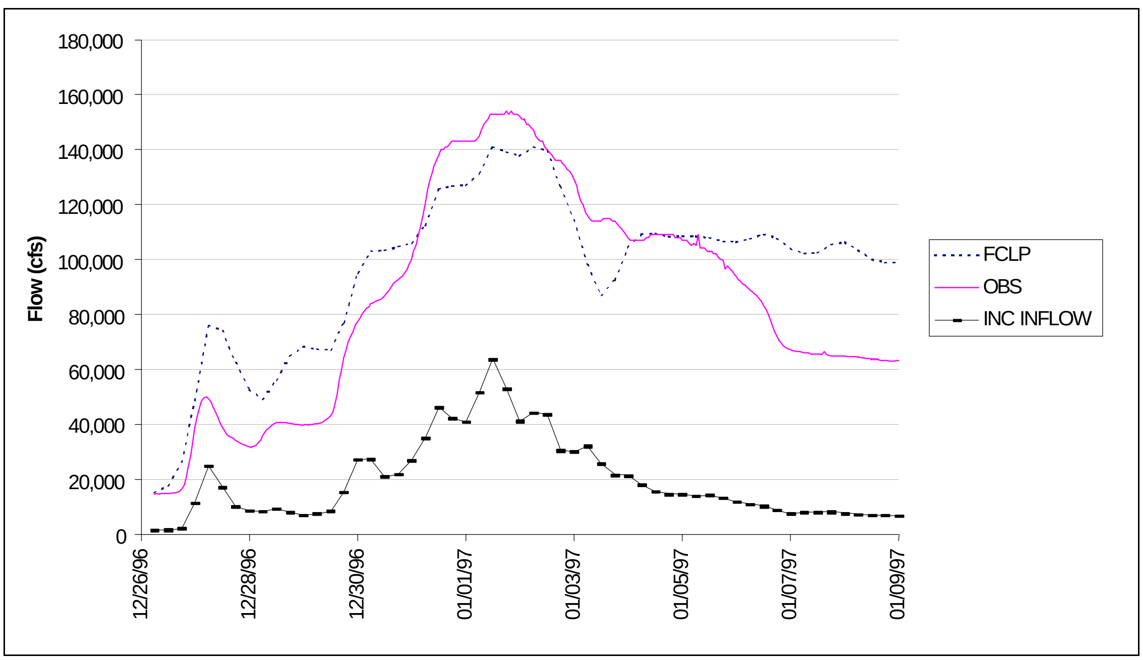

Figure 20. Flow at Vina-Woodson.

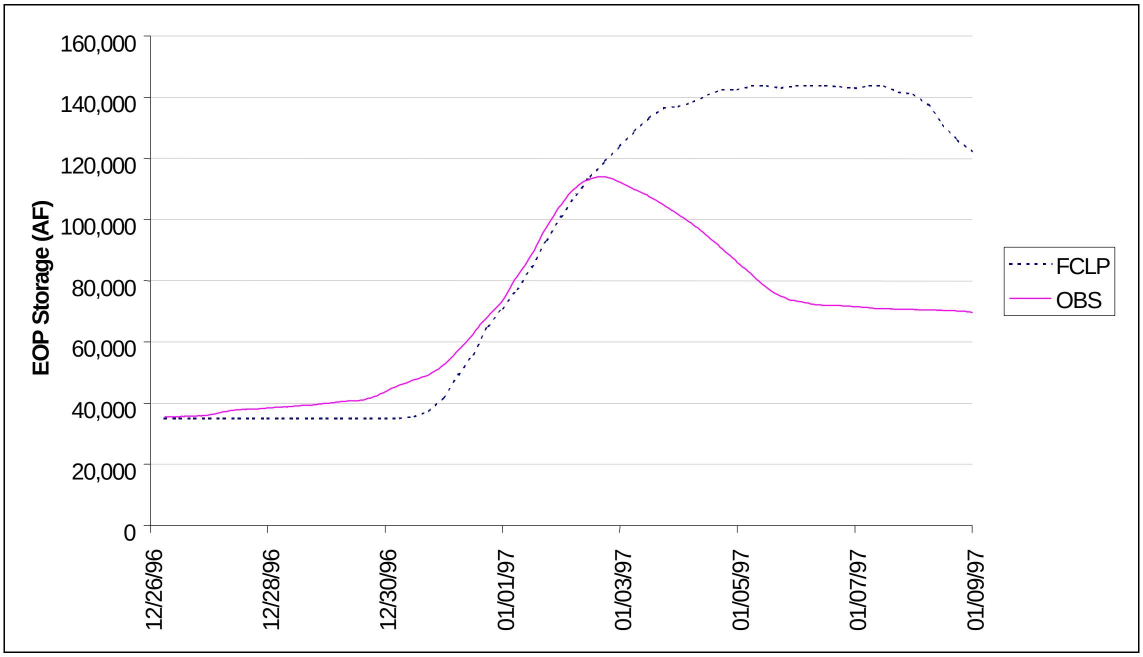

Figure 21. Black Butte Dam EOP storage.

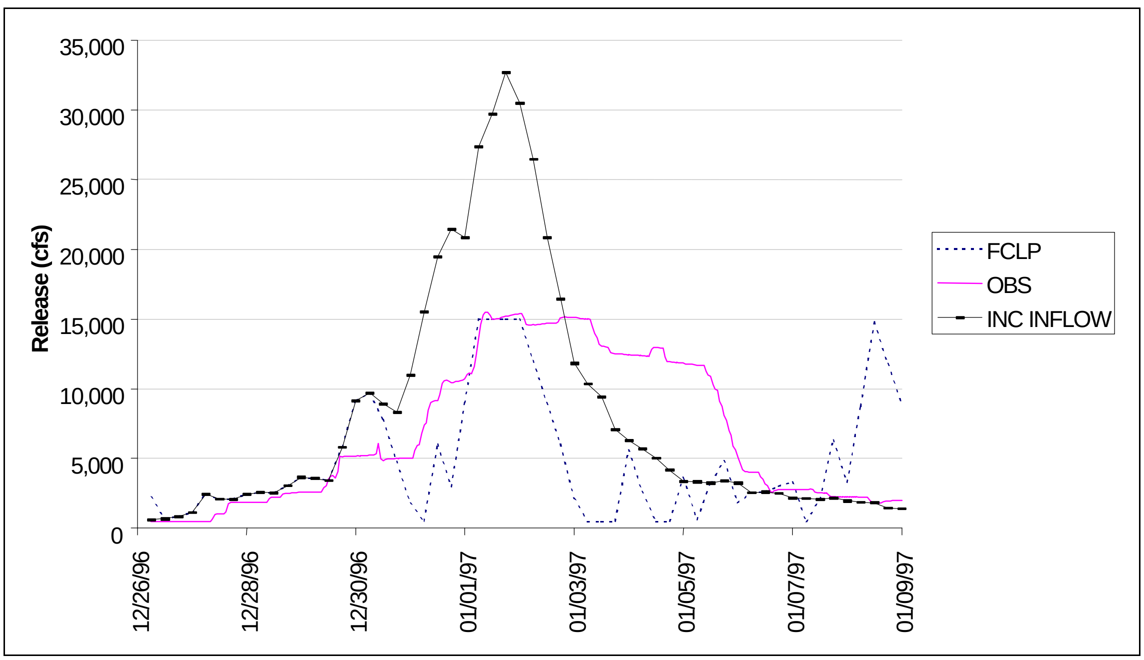

Figure 22. Black Butte Dam release.

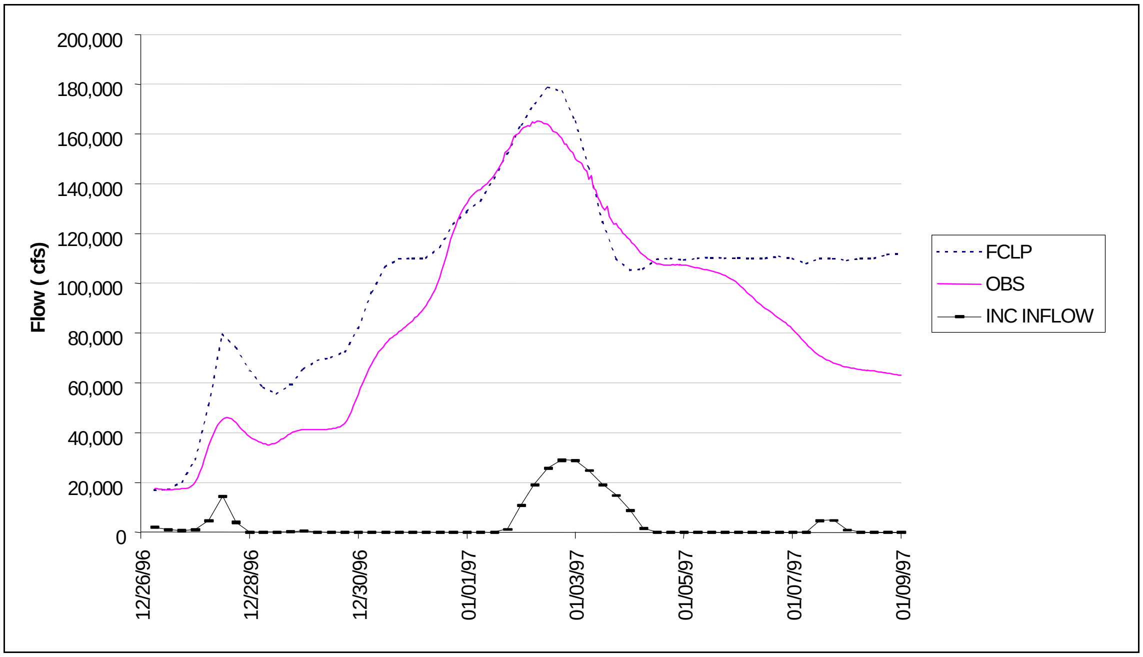

Figure 23. Flow at Ord Ferry.

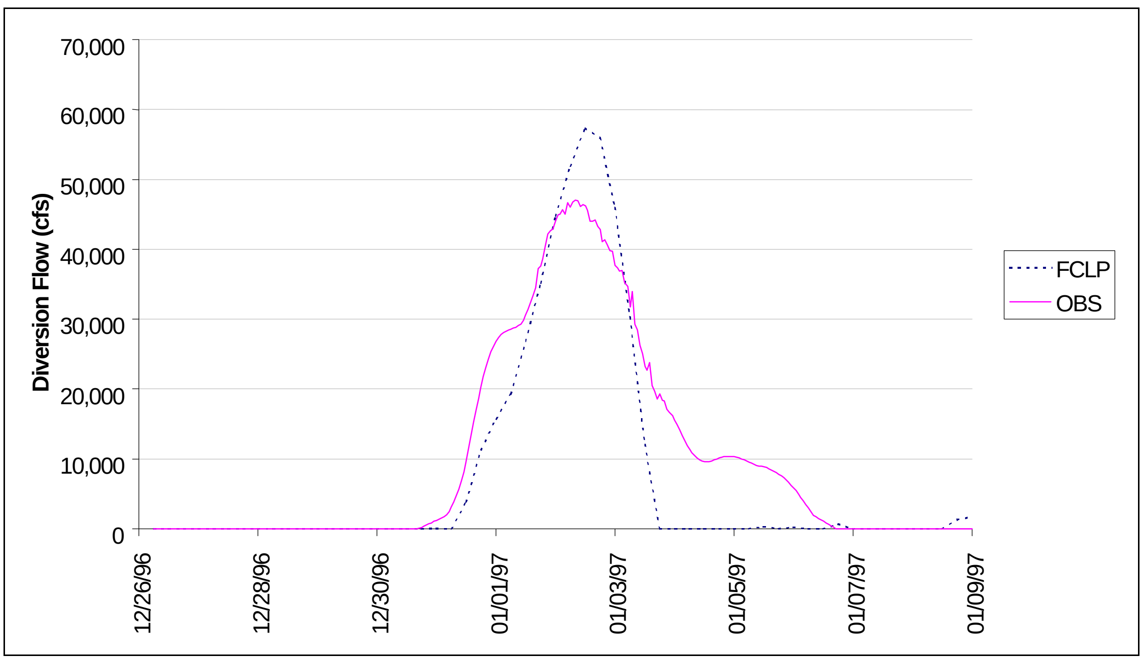

Figure 24. Ord Ferry overflow to Butte Basin.

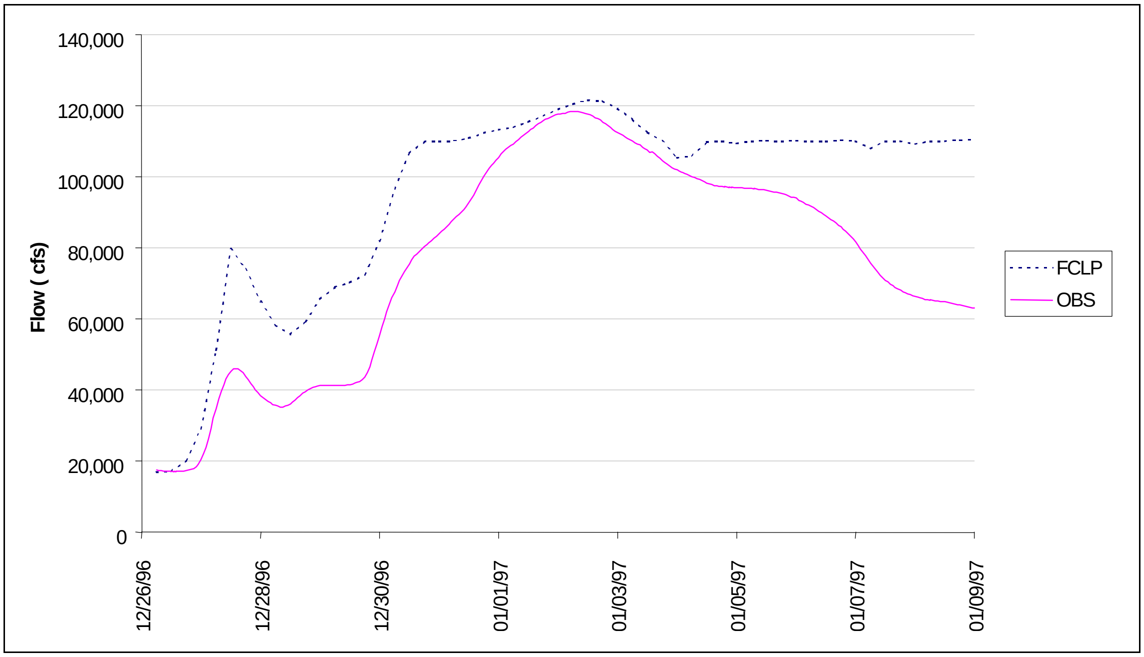

Figure 25. Flow past Ord Ferry in Sacramento River.

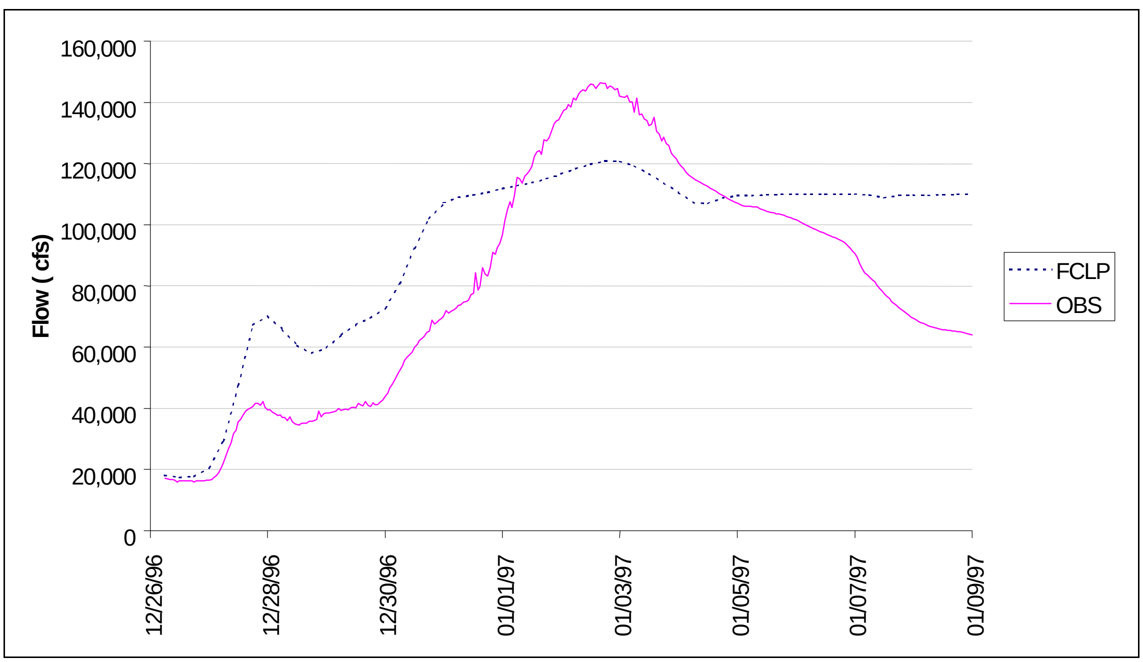

Figure 26. Flow at Butte City.

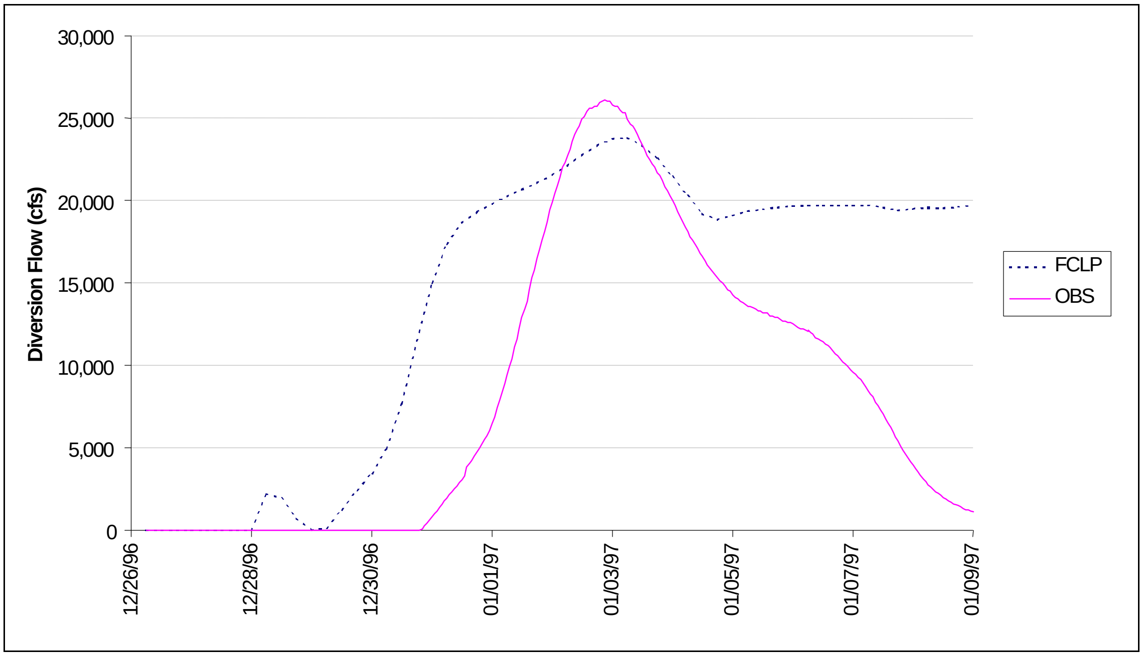

Figure 27. Flow over Moulton Weir.

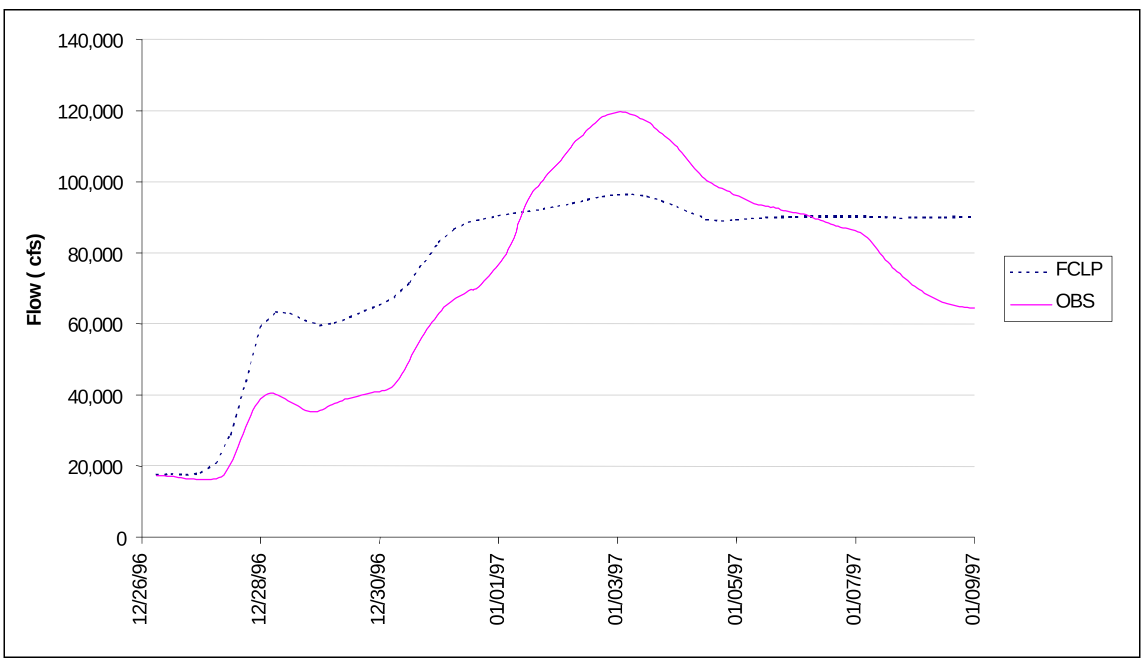

Figure 28. Flow past Moulton Weir in Sacramento River.

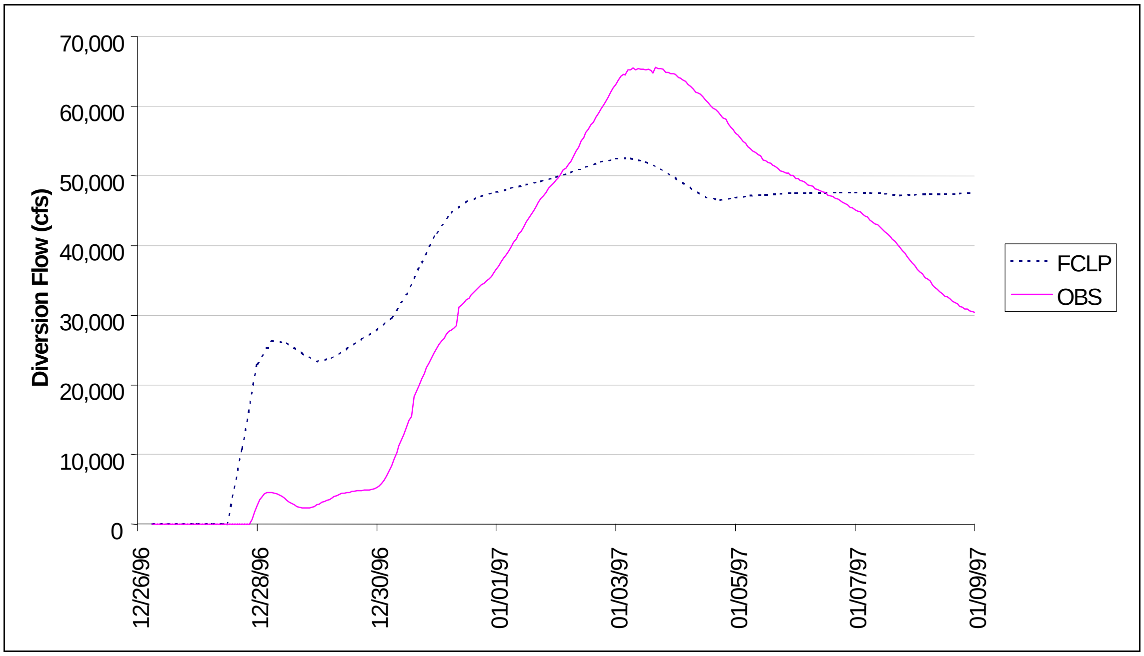

Figure 29. Flow over Colusa Weir.

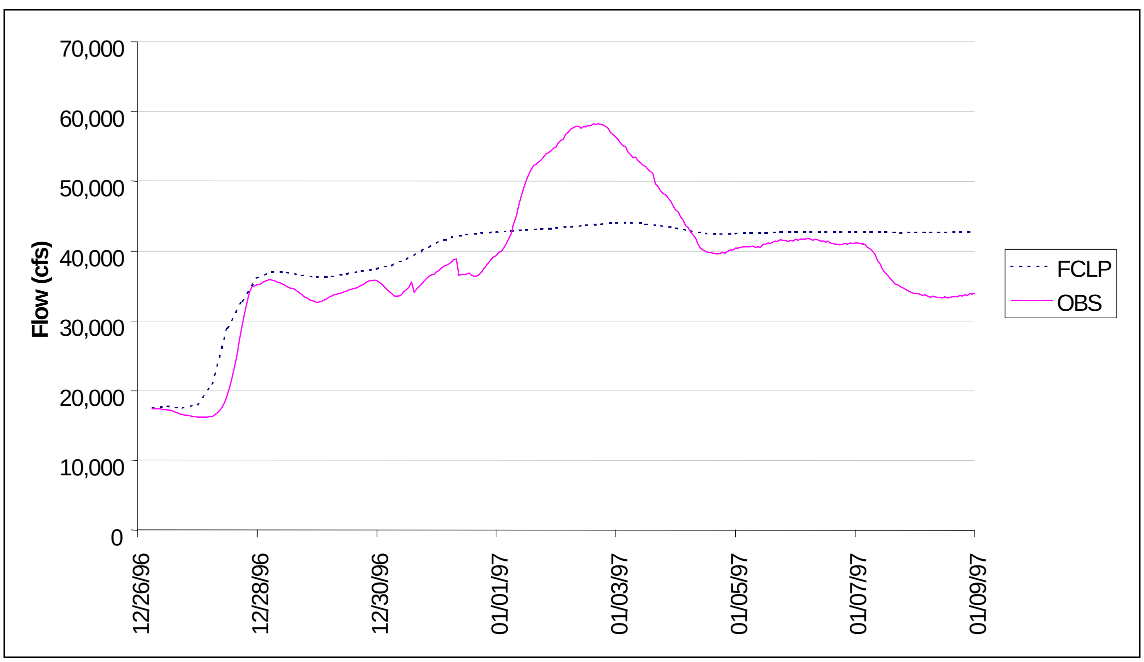

Figure 30. Flow past Colusa Weir in Sacramento River.

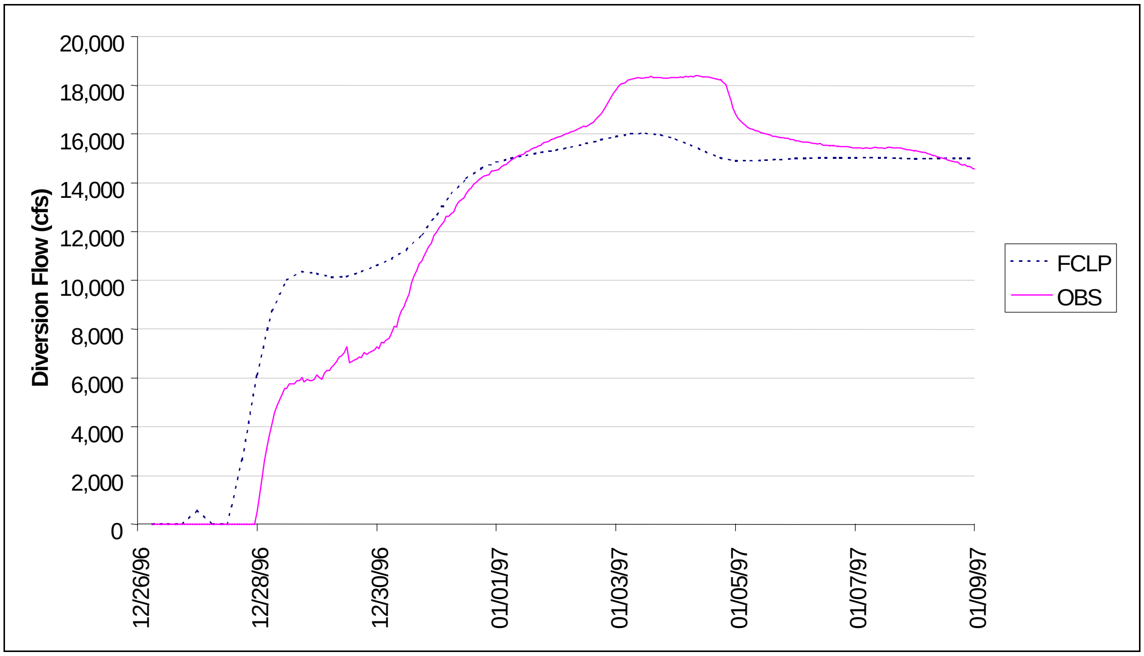

Figure 31. Flow over Tisdale Weir.

Figure 32. Flow past Tisdale Weir in Sacramento River.

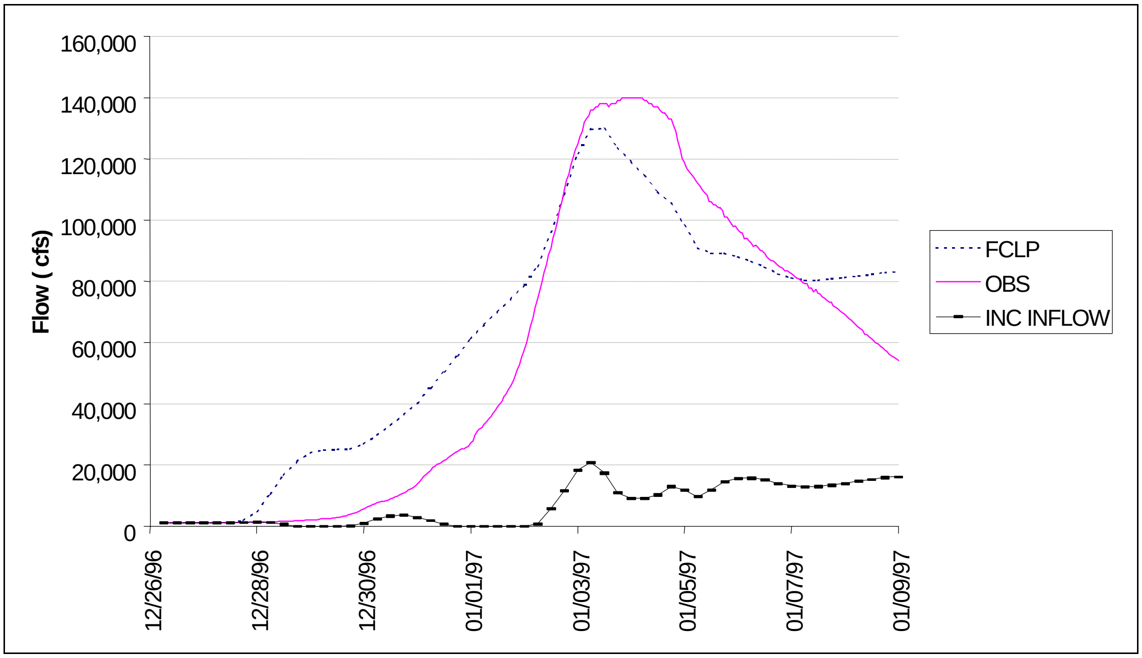

Figure 33. Flow at Meridian.

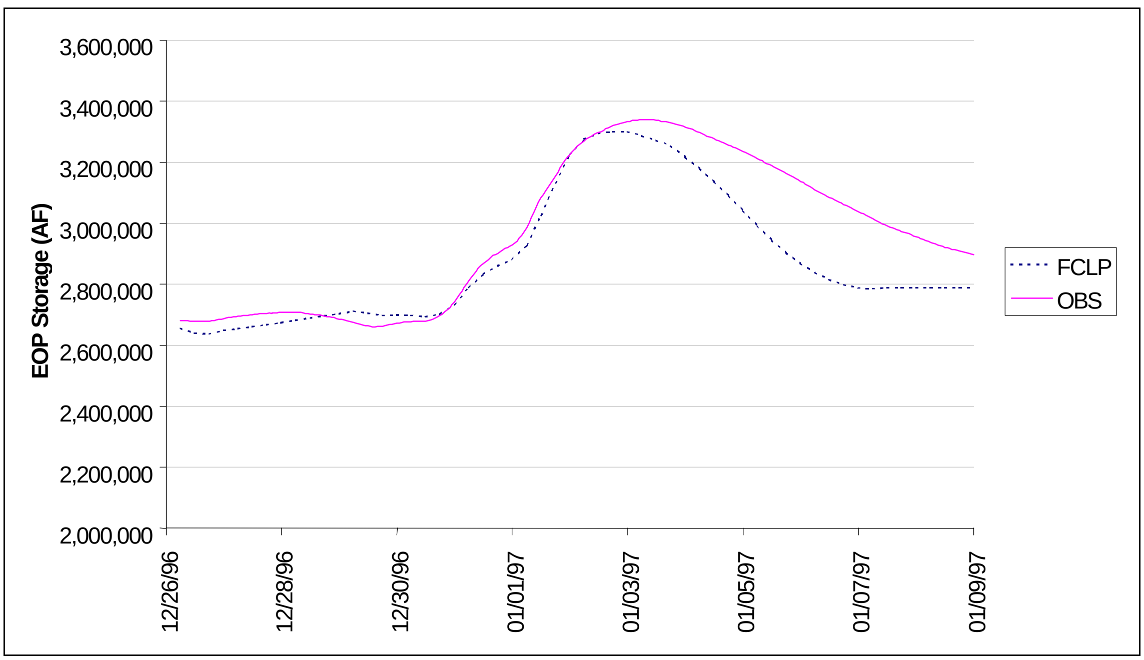

Figure 34. Oroville Dam EOP storage.

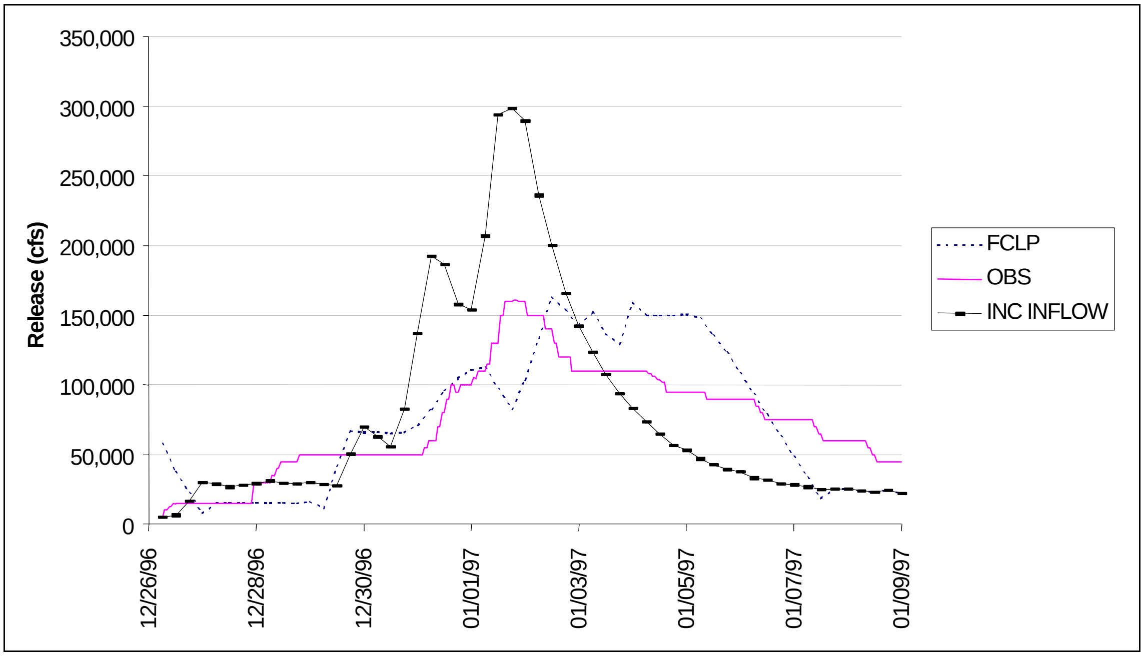

Figure 35. Oroville Dam release.

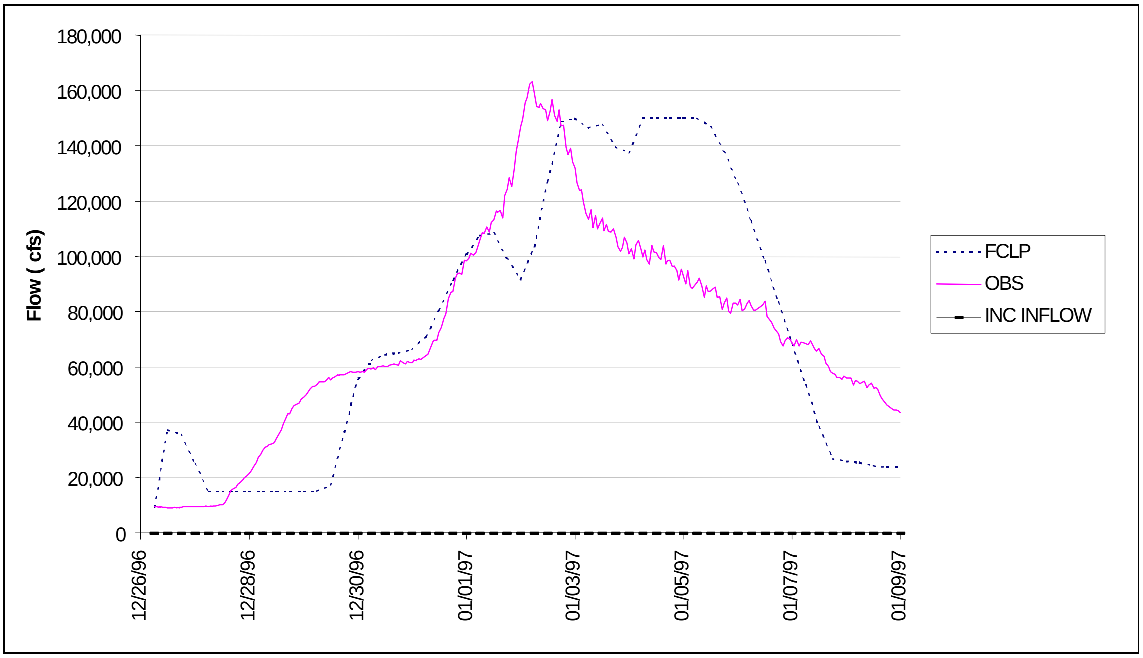

Figure 36. Flow at Gridley.

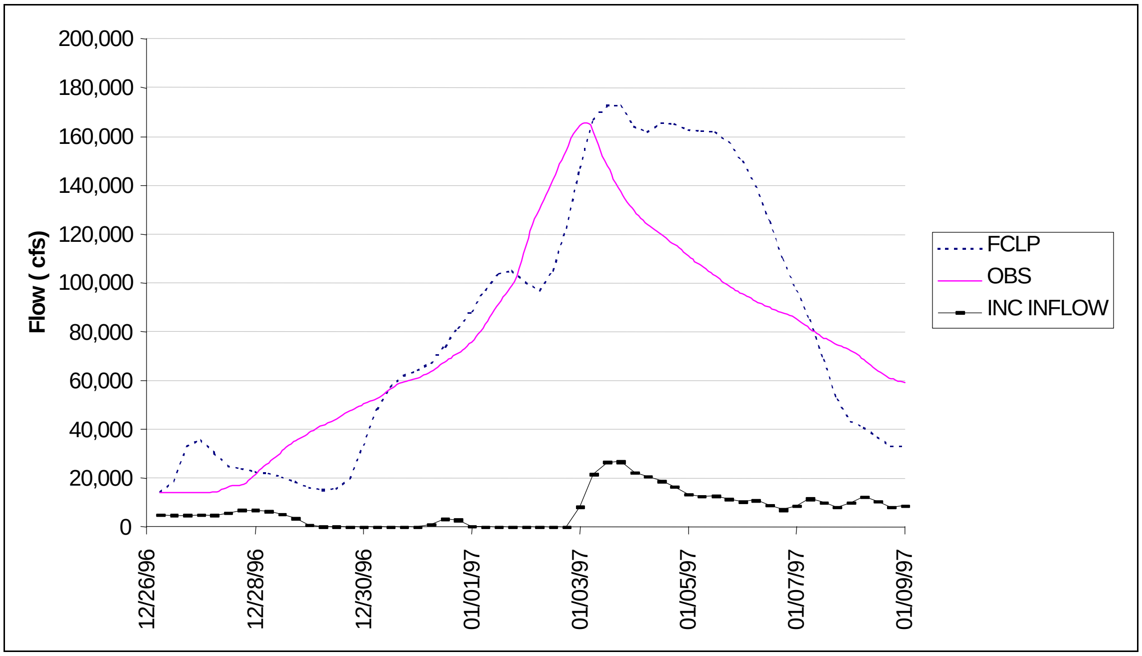

Figure 37. Flow at Yuba City.

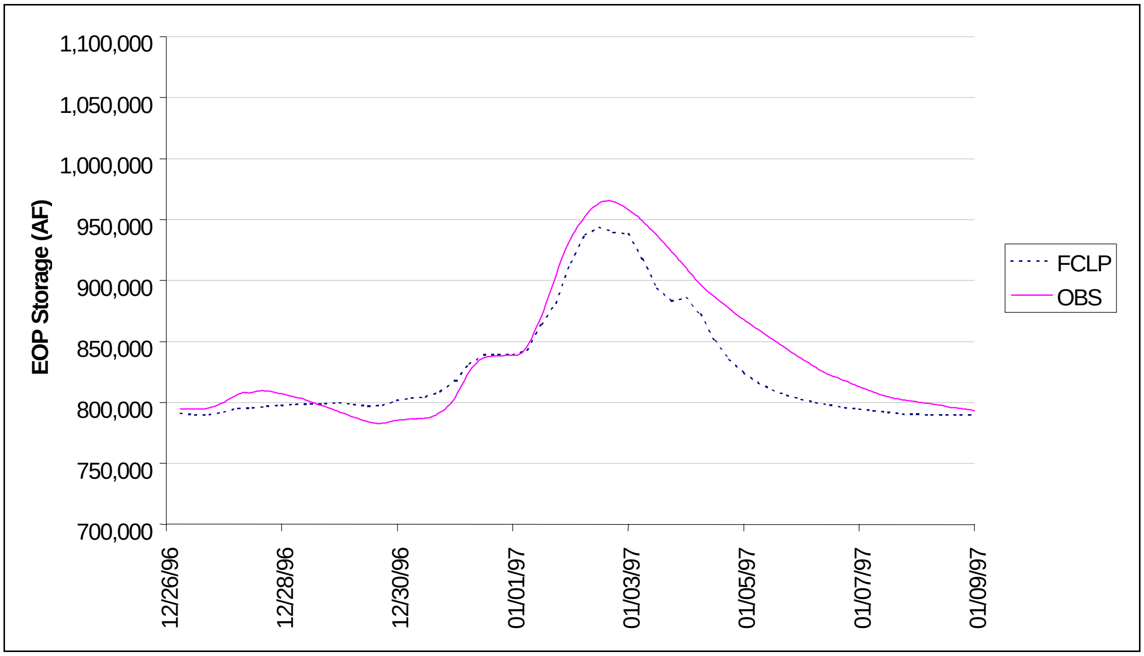

Figure 38. New Bullards Bar Dam EOP storage.

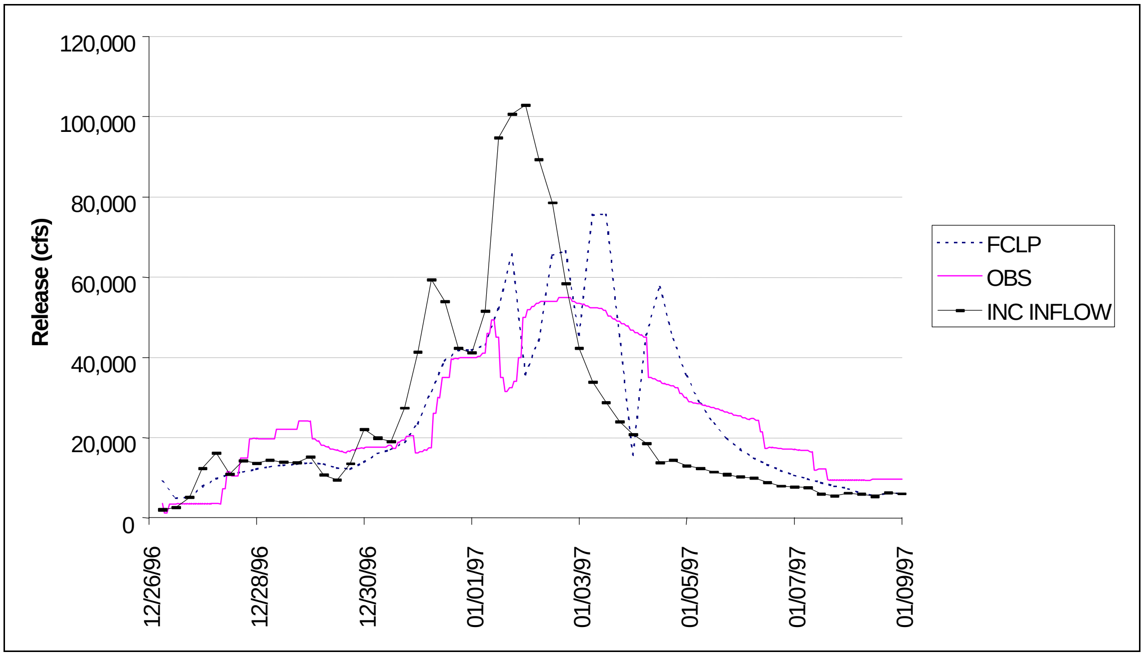

Figure 39. New Bullards Bar Dam release.

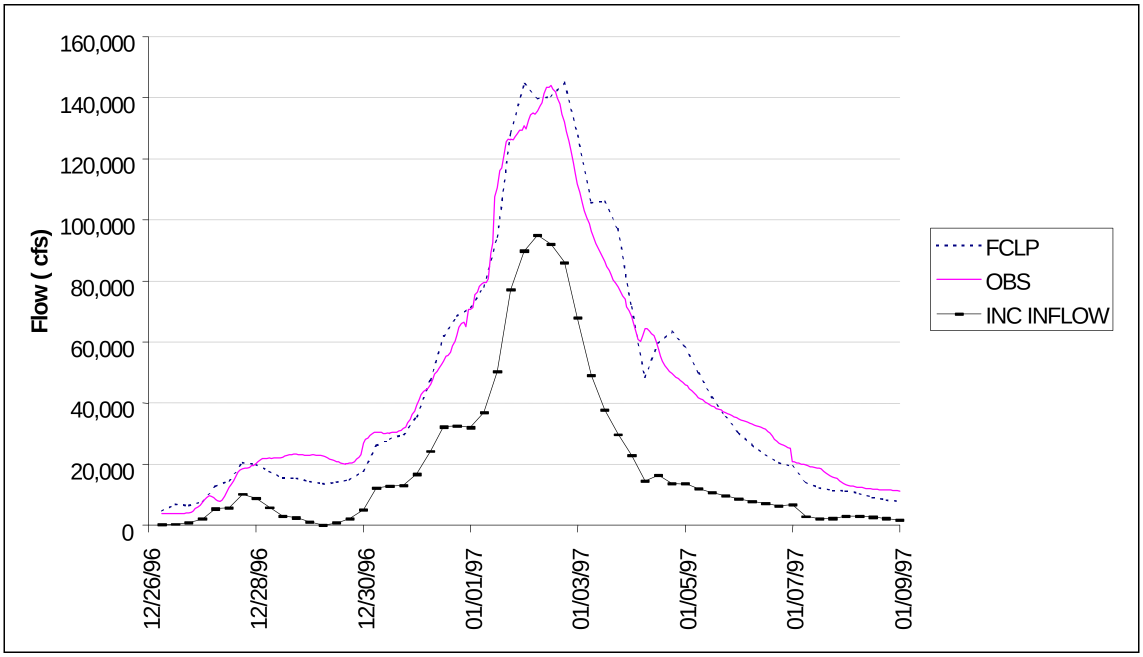

Figure 40. Flow at Marysville.

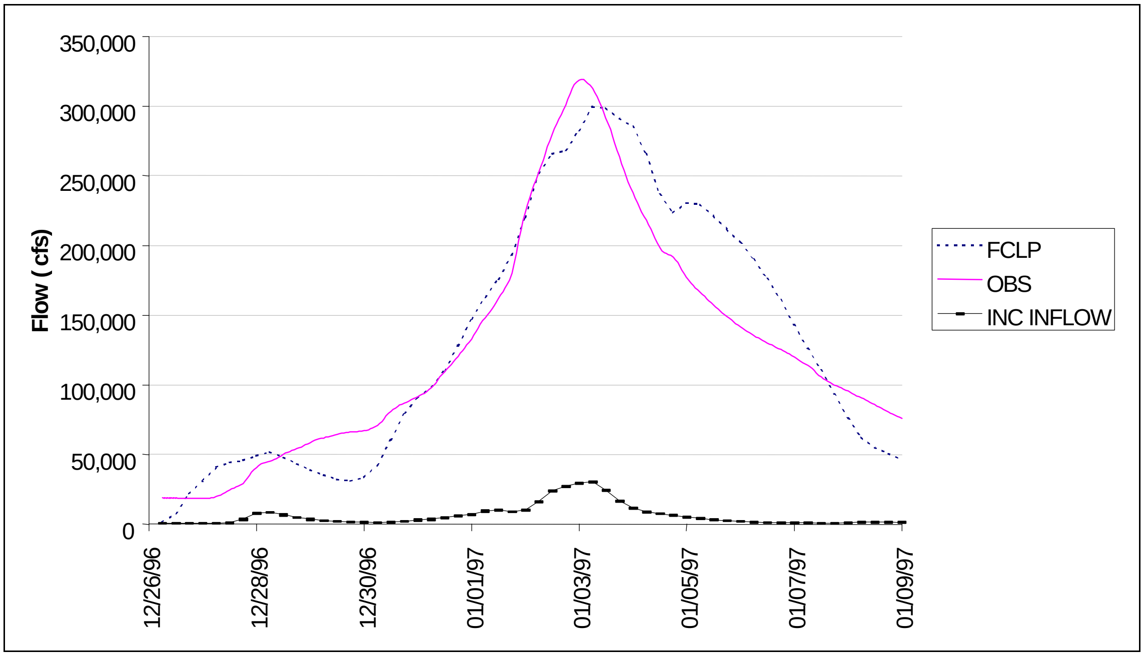

Figure 41. Flow at Nicolaus.

Figure 42. Flow over Fremont Weir into Yolo Bypass.

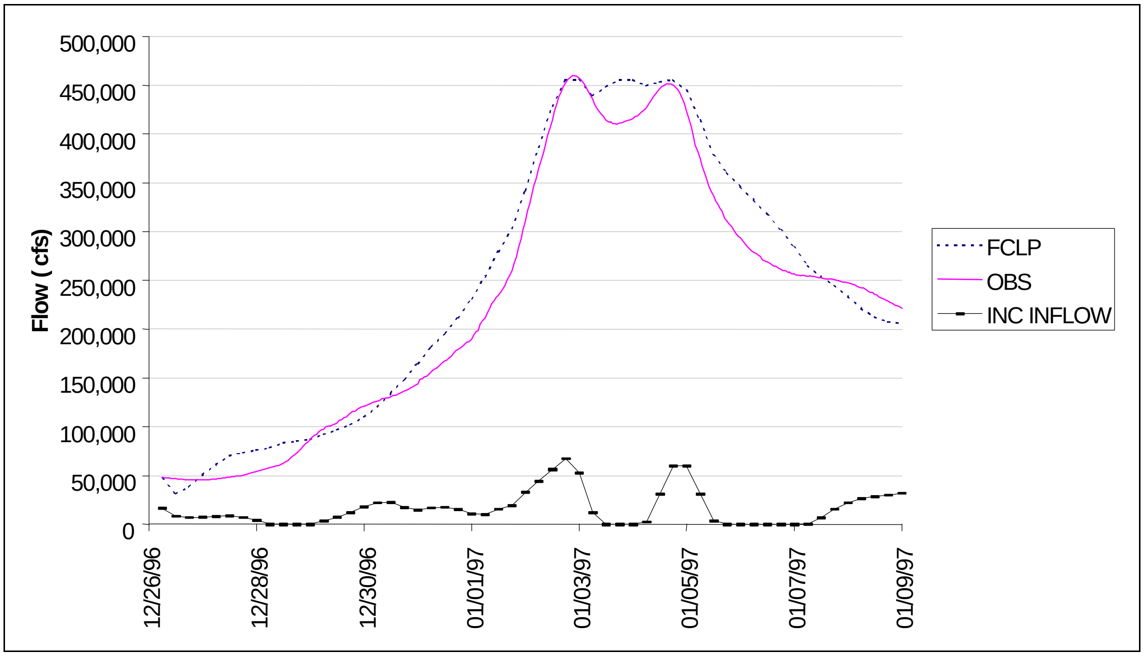

Figure 43. Flow past Fremont Weir/Verona in Sacramento River.

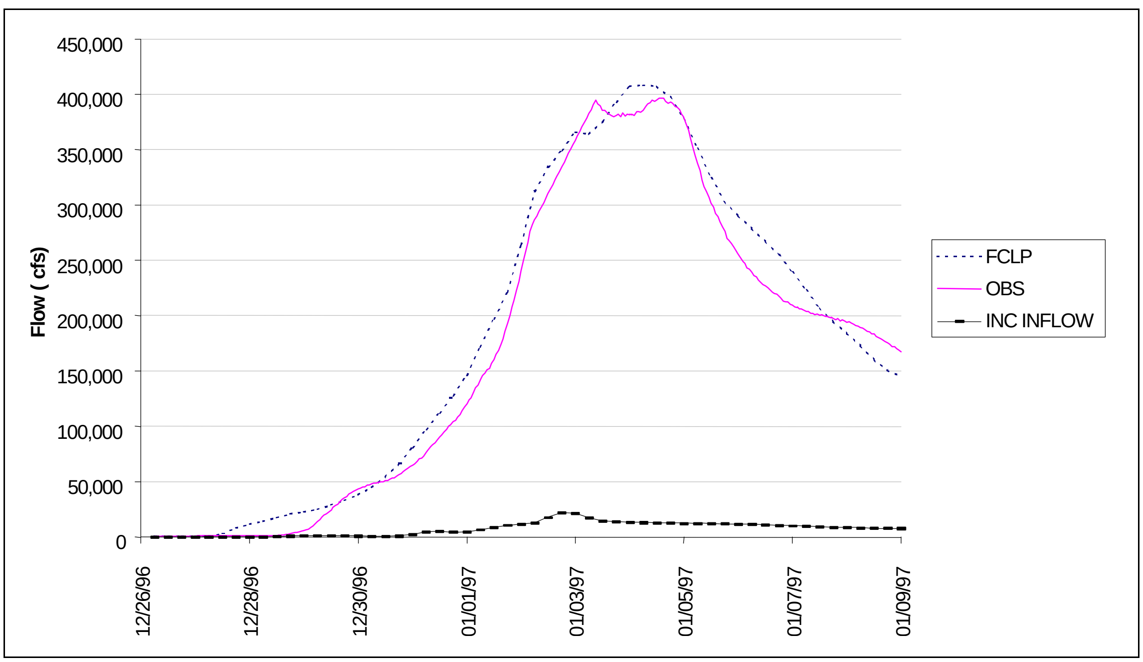

Figure 44. Flow at Woodland.

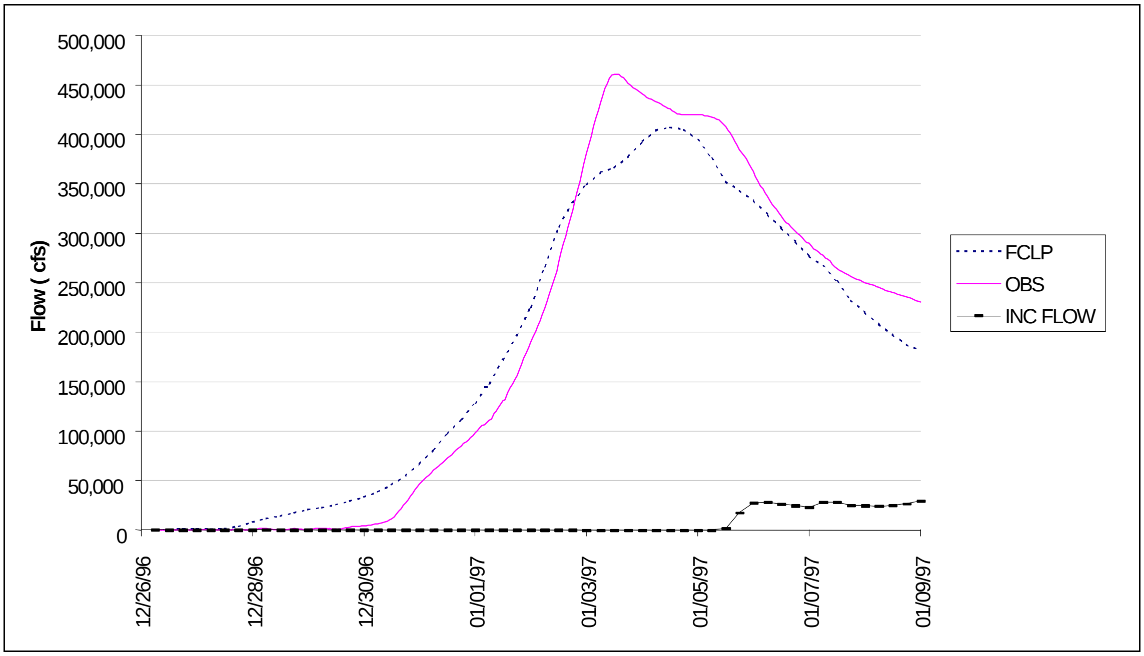

Figure 45. Flow at Lisbon.

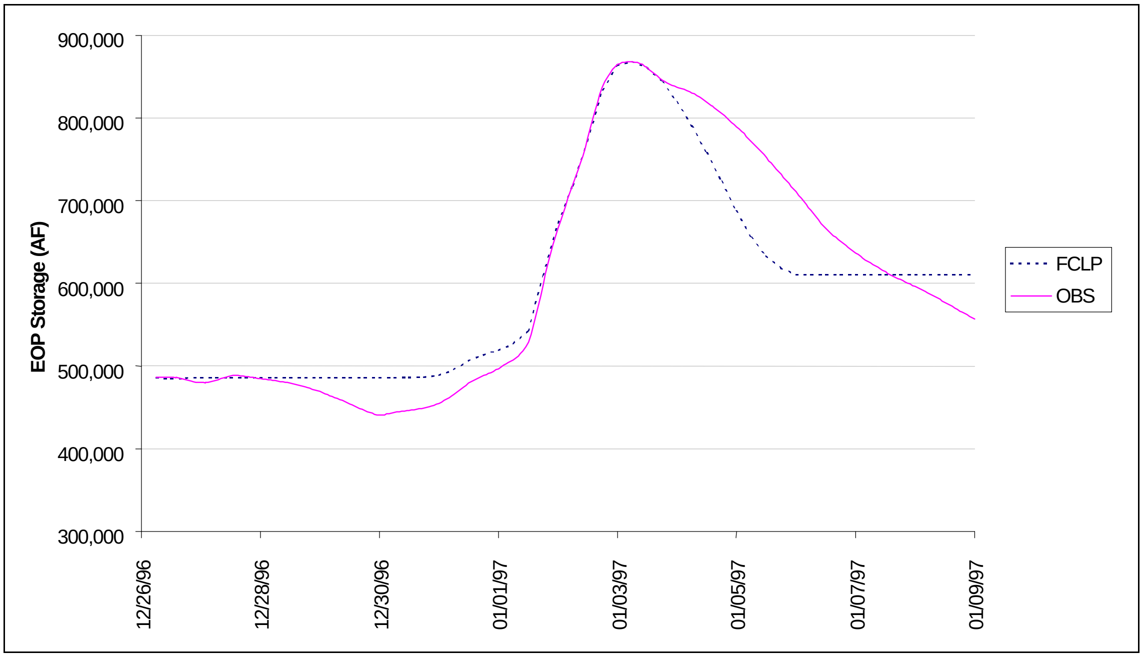

Figure 46. Folsom Dam EOP storage.

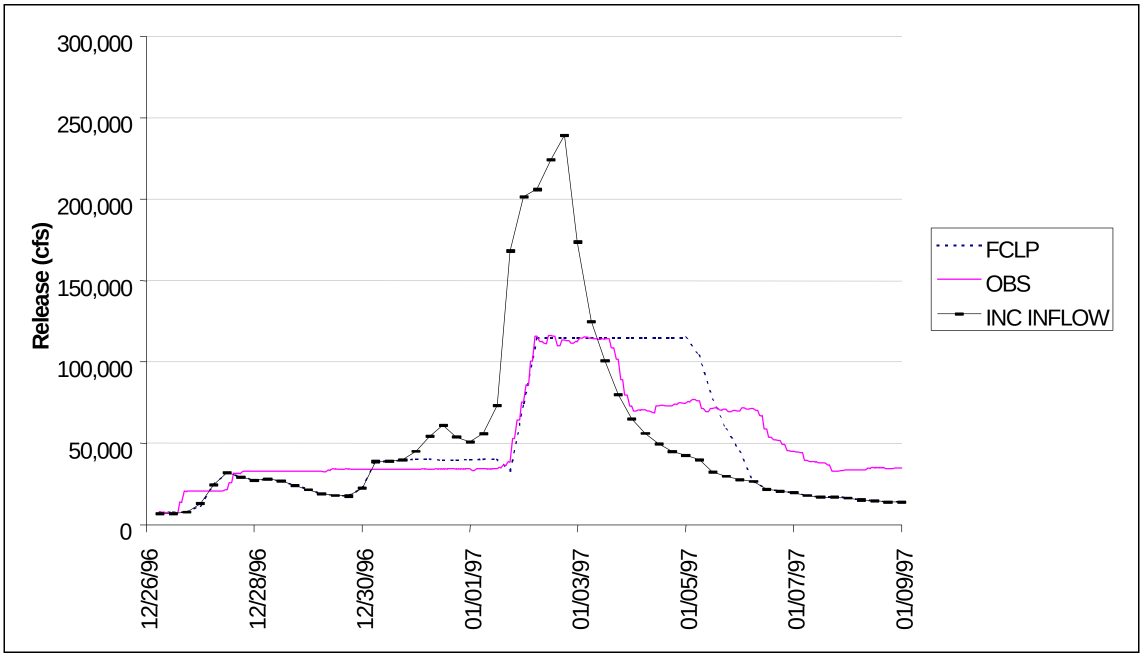

Figure 47. Folsom Dam release.

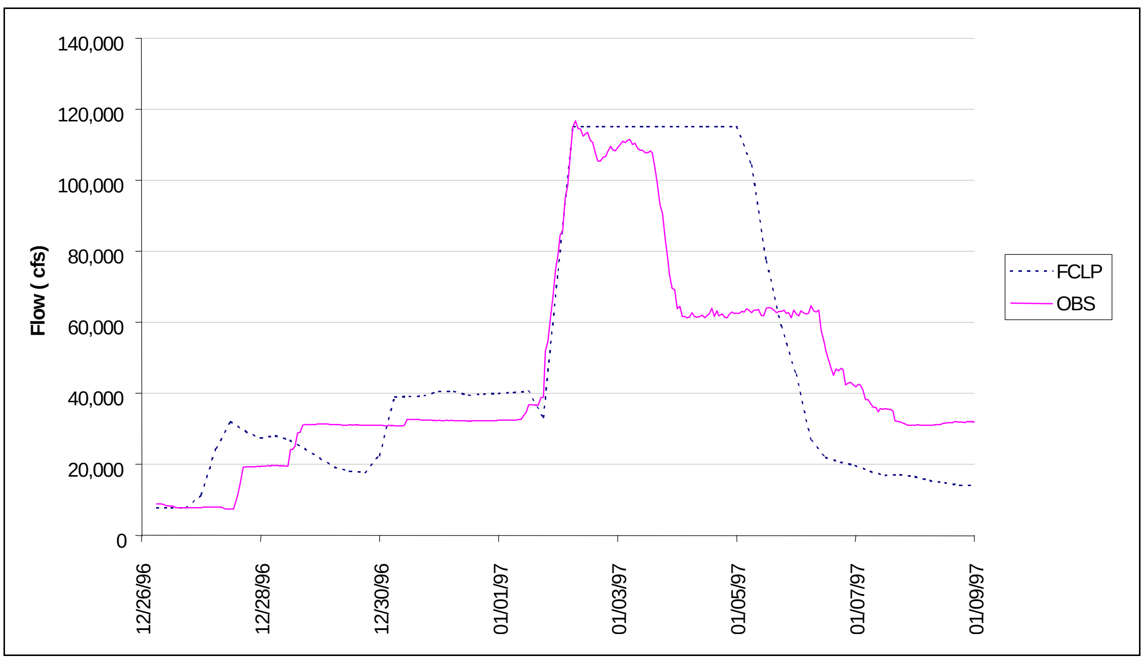

Figure 48. Flow at Fair Oaks.

Figure 49. Flow over Sacramento Weir.

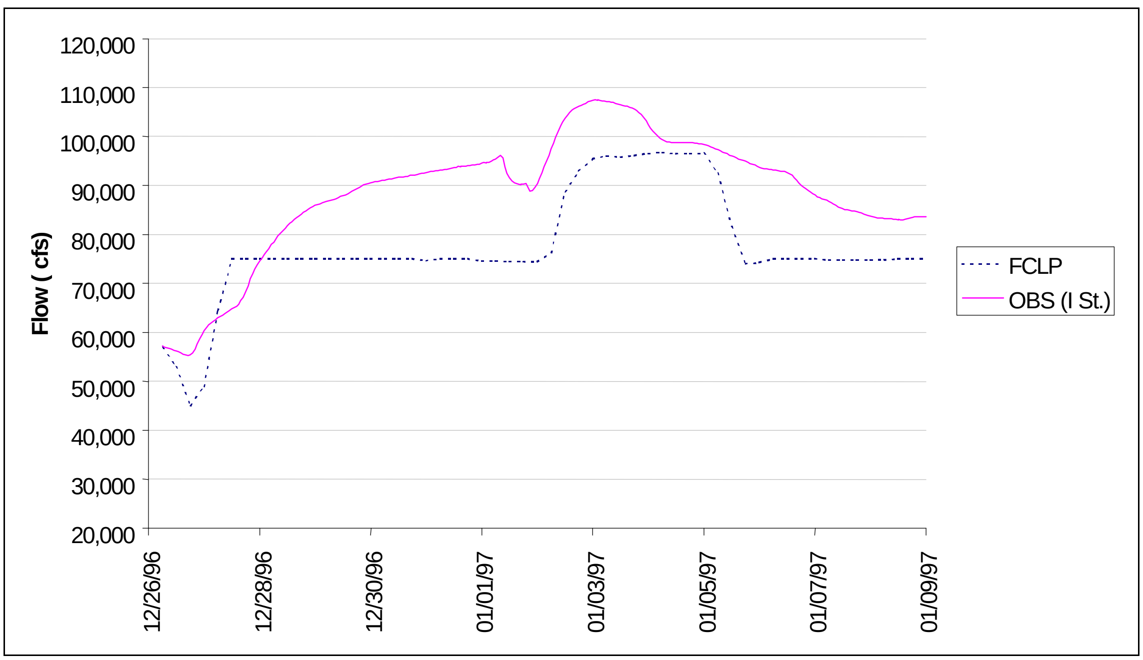

Figure 50. Flow past Sacramento Weir in Sacramento River.

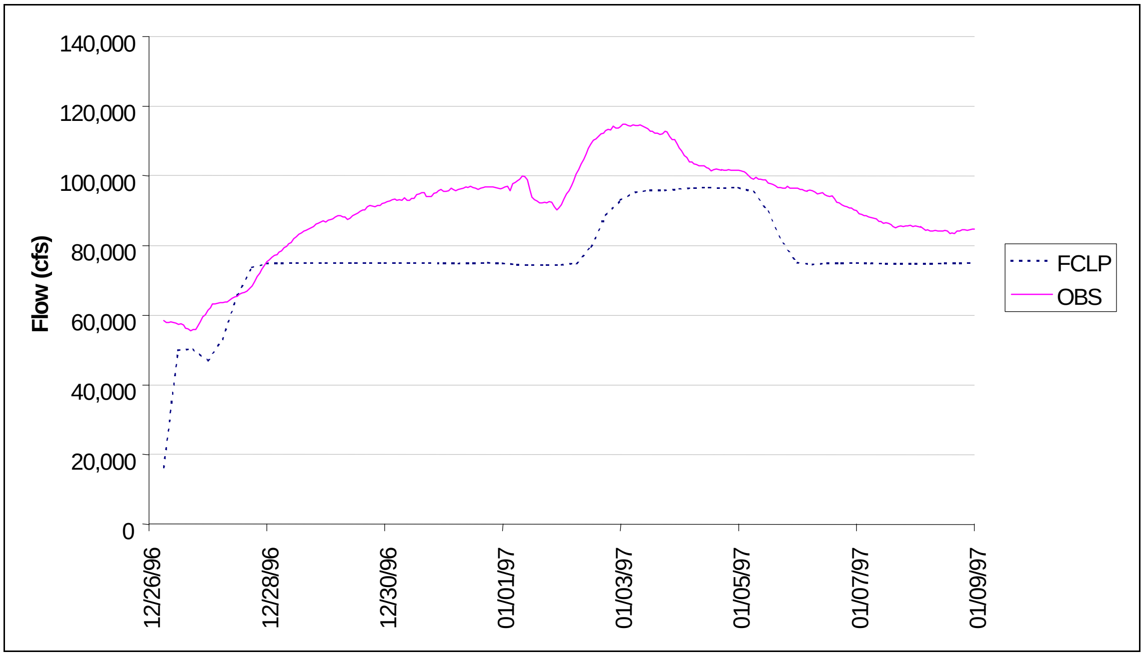

Figure 51. Flow at Freeport.

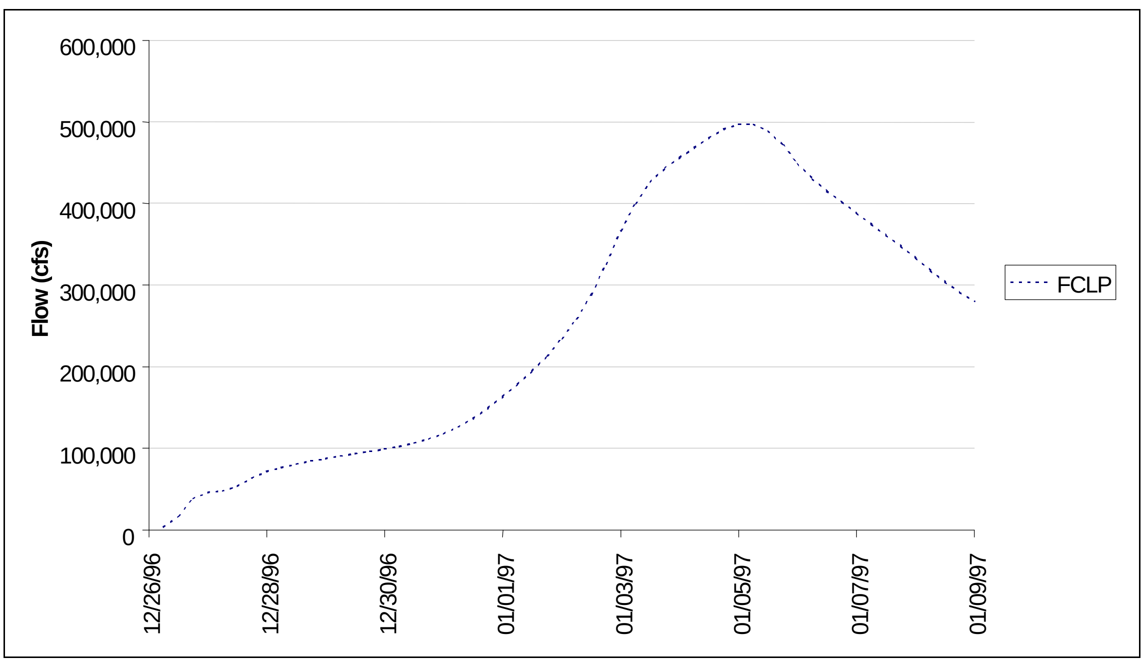

Figure 52. Model flow at Rio Vista.

Chapter 6

1995 MODEL RESULTS AND DISCUSSION

The model was run with the 1995 incremental inflows using the 1997 calibration criteria developed previously. This was done to determine whether there would have been any benefit to managing the 1995 event with the updated operating policies (e.g. conservation pool levels). The results of the model and observed operation are shown in Figures 53 through 88.

Figure 54 shows that the model makes much larger releases from Shasta Dam in the beginning of the event to make storage available for the coming inflows. Model storage does not exceed 3,900,000 AF. Both model and observed flows at Bend Bridge exceed the listed channel capacity of 100,000 cfs (Figure 55), although the model peak flow is slightly less than the observed. However at Vina-Woodson, the observed peak flow is lower than the model flow, as shown in Figure 56. This contradicts the 1997 results which showed the model was able to reduce the peak flow at Vina-Woodson. The difference in the apparent effect of Shasta Dam releases on the flow at Vina-Woodson can be attributed to a greater amount of incremental inflow below the dam during the 1995 event.

Figure 58 shows Black Butte Dam releases are limited to 15,000 cfs by both the model and observed operation. The observed and model flows past Ord Ferry are both below the channel capacity of 160,000 cfs, as shown in Figure 61. By comparing Figure 61 and Figure 62, it appears that the neglect of incremental inflows to Butte City is less significant in the 1995 event than in the 1997 event.

The model is able to keep Oroville Dam storage at the top of conservation level (2,788,000 AF) throughout most of the event, as shown in Figure 70. Flows at Gridley and Yuba City are kept below their channel capacities, by both the model and historic operation, with historic operation having the lower peak flow in both instances. These are shown in Figure 72 and Figure 73.

The storage at New Bullards Bar Dam is kept well below the gross pool capacity of 960,000 AF for both the model and historic operation (Figure 74). Peak flows downstream of New Bullards Bar Dam at Marysville also are kept far below the capacity flow of 120,000 cfs as shown in Figure 76. Flows farther downstream at Nicolaus are kept below the channel capacity limits as well (Figure 77).

Figure 83 shows that the model has a higher release from Folsom Dam at the beginning of the event to bring the storage down to the 486,000 AF top-of-conservation level. Flows downstream of Folsom Dam and past the Sacramento Weir on the Sacramento River are kept below capacity (Figure 84 and Figure 86). Model flow at the farthest downstream control point of Rio Vista is shown in Figure 88. A summary of channel capacities, along with model and historic peak flows for the 1995 event, is listed in Table 11.

Table 11. Summary of 1995 results.

Control Point	Channel Capacity (cfs)	Observed Peak (cfs)	Model Peak (cfs)
Bend Bridge	100,000	126,580	120,848
Vina-Woodson	260,000	141,563	174,999
Ord Ferry	160,000	130,437	121,185
Butte City	160,000	124,100	120,185
Moulton Weir	160,000	105,511	96,148
Colusa Weir	60,000	55,541	43,938
Tisdale Weir	30,000	40,912	27,914
Yuba City	210,000	90,728	106,197
Marysville	120,000	34,600	37,957
Nicolaus	320,000	129,738	153,568
Fair Oaks	115,000	51,051	41,097
Sacramento (I st.)	110,000	94,880	75,000
Freeport	110,000	102,162	74,997
Woodland	377,000	240,850	290,862
Lisbon	490,000	270,828	288,776
Rio Vista	579,000	N/A	358,975

* 180,000 cfs when flow on Feather R. is low

Figure 53. Shasta Dam EOP storage.

Figure 54. Shasta Dam release.

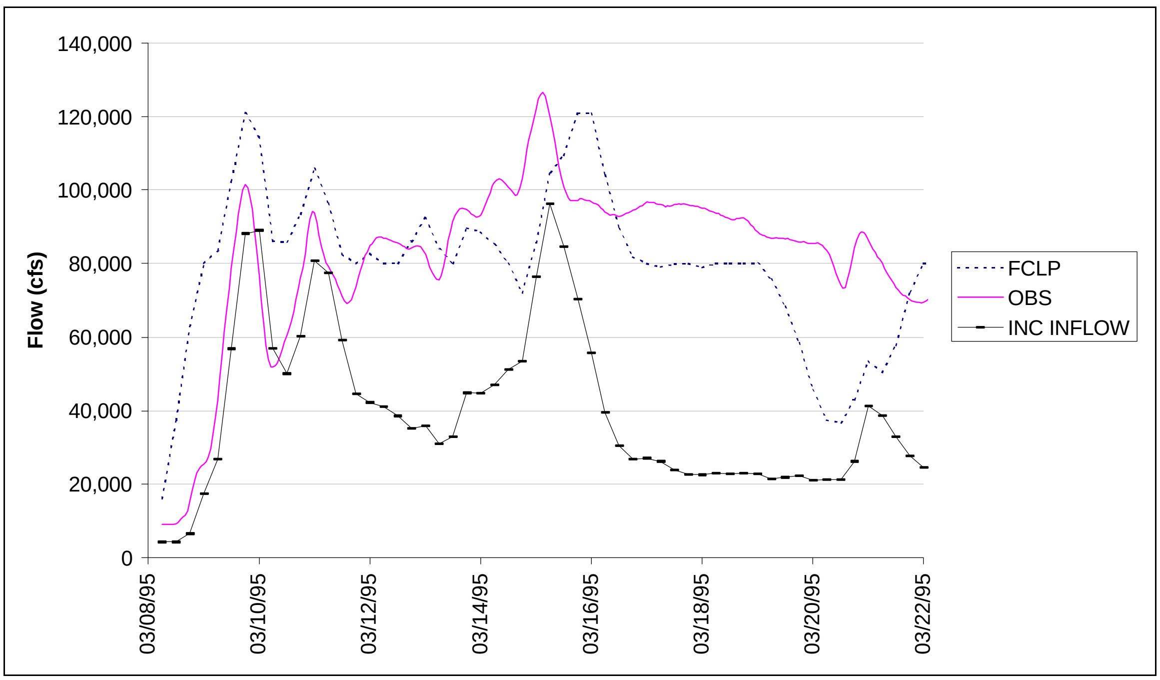

Figure 55. Flow at Bend Bridge.

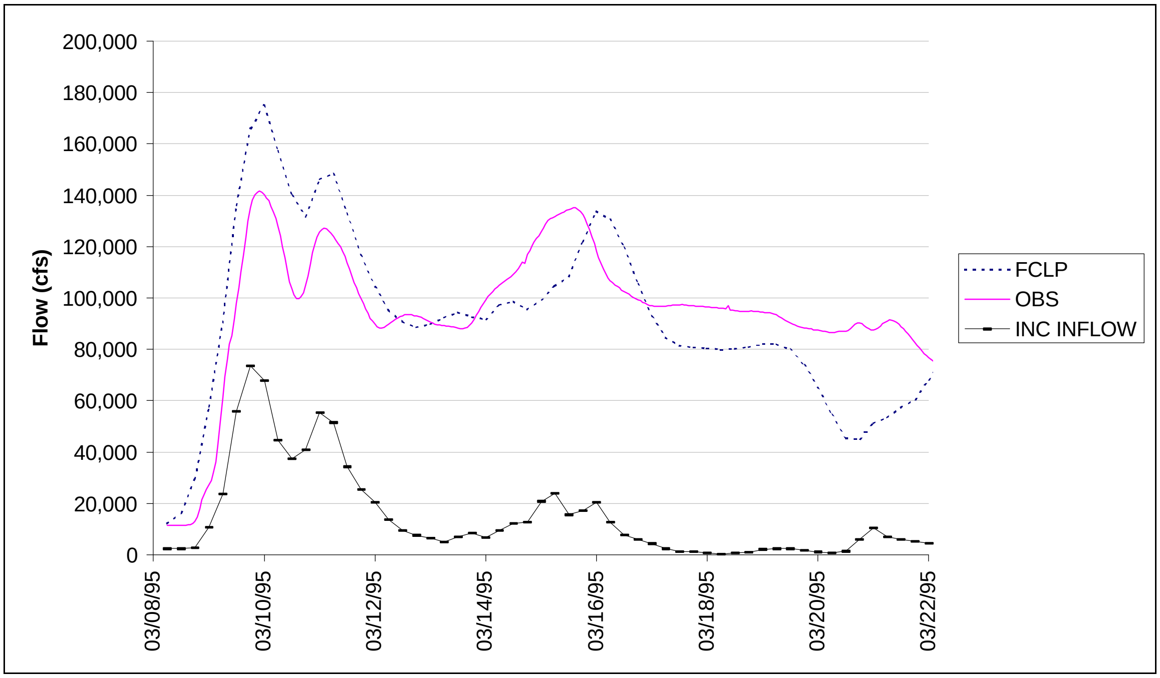

Figure 56. Flow at Vina-Woodson.

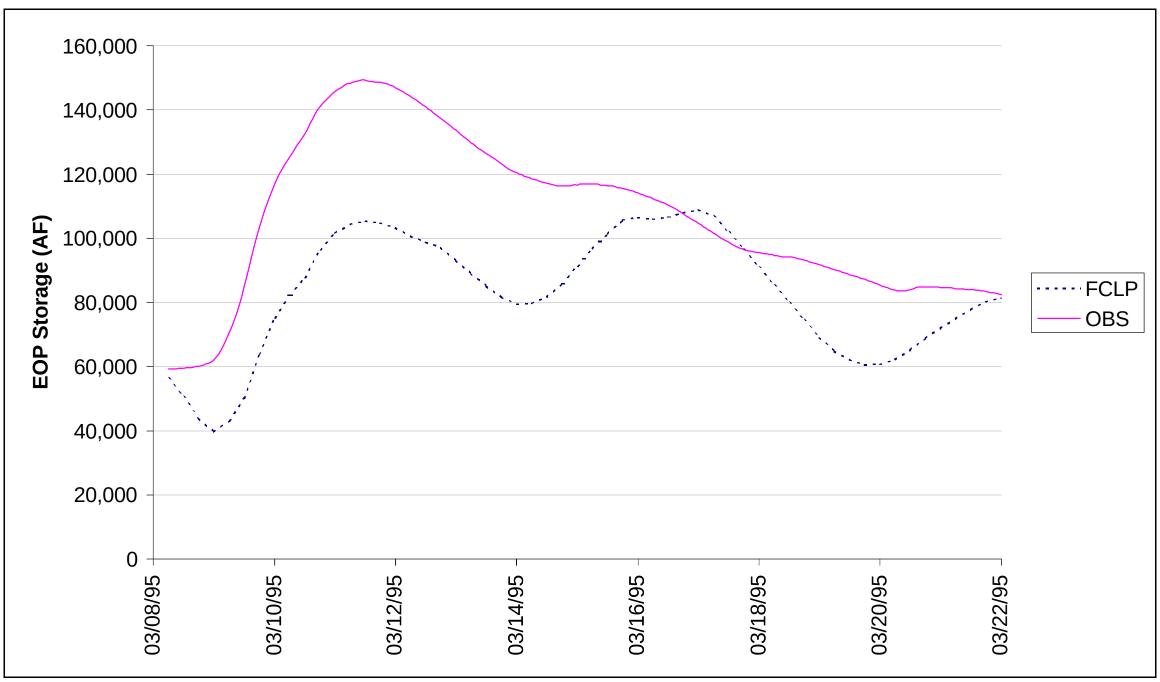

Figure 57. Black Butte Dam EOP storage.

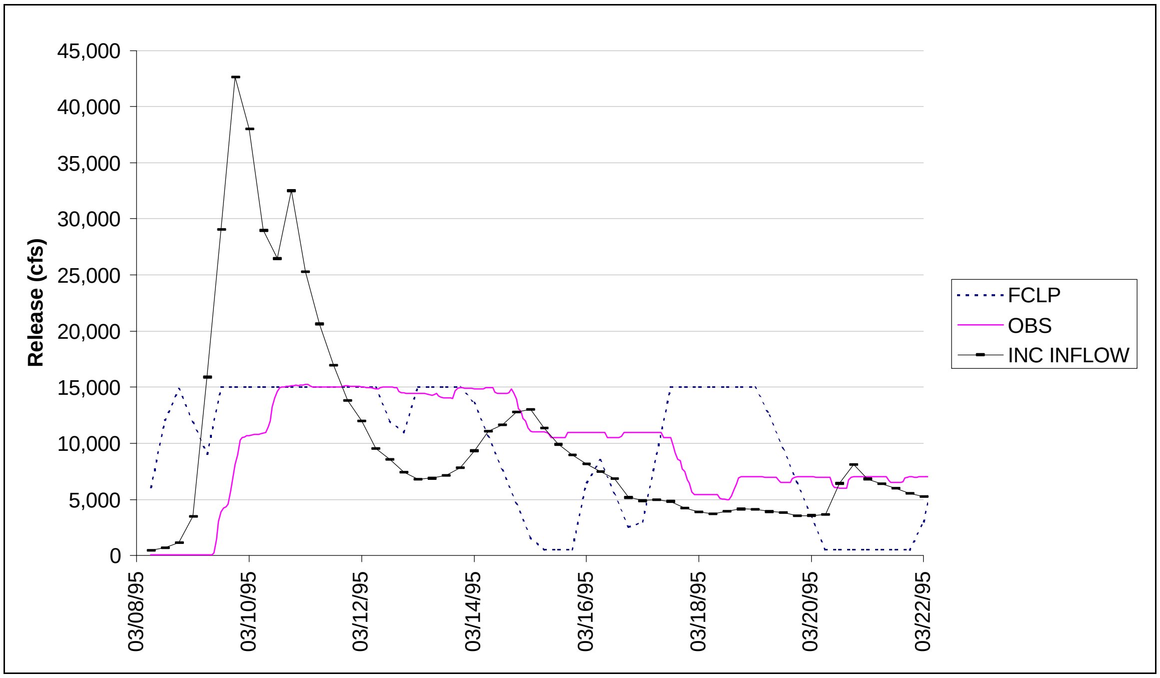

Figure 58. Black Butte Dam release.

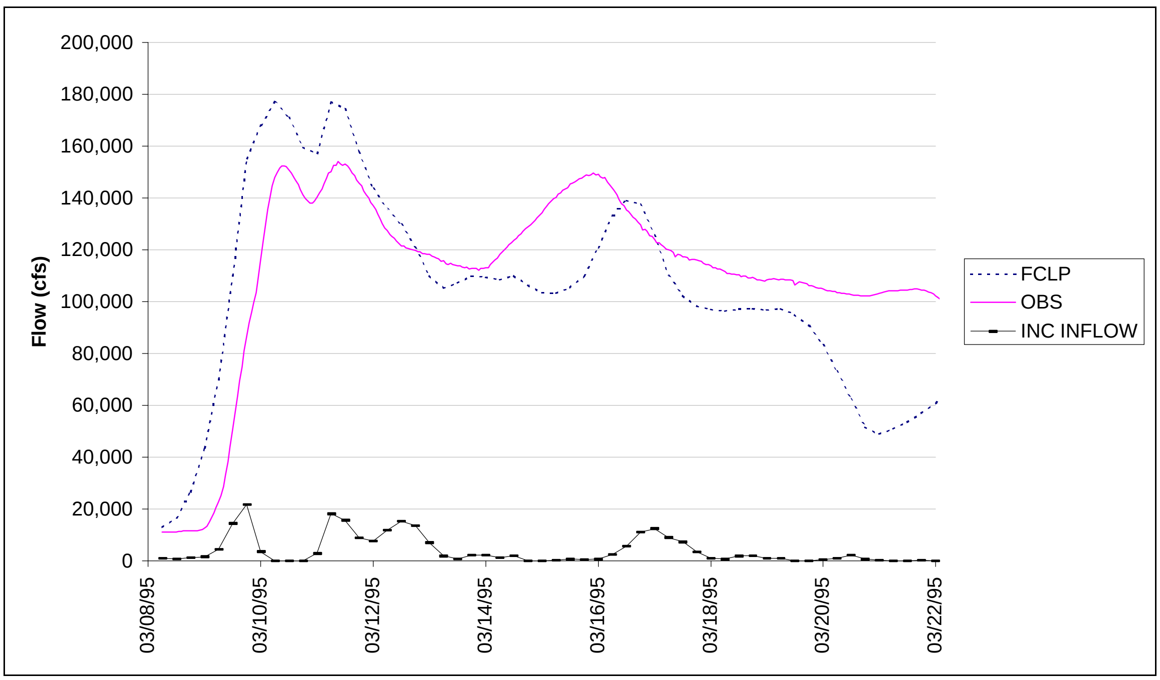

Figure 59. Flow at Ord Ferry.

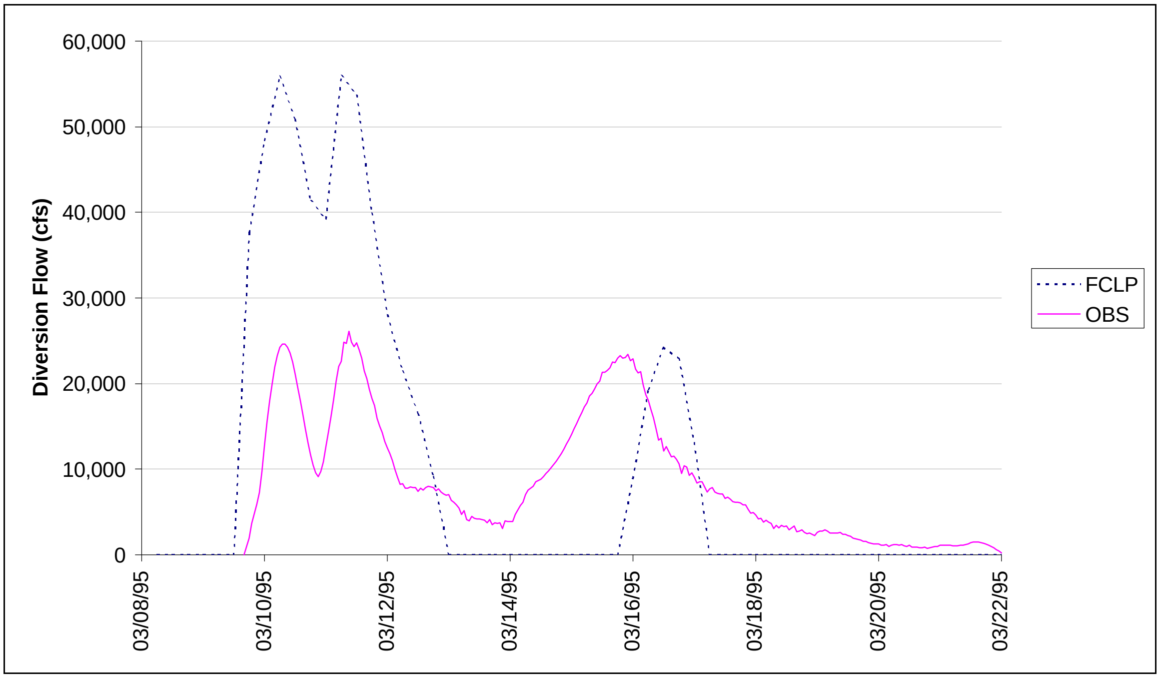

Figure 60. Ord Ferry overflow to Butte Basin.

Figure 61. Flow past Ord Ferry in Sacramento River.

Figure 62. Flow at Butte City.

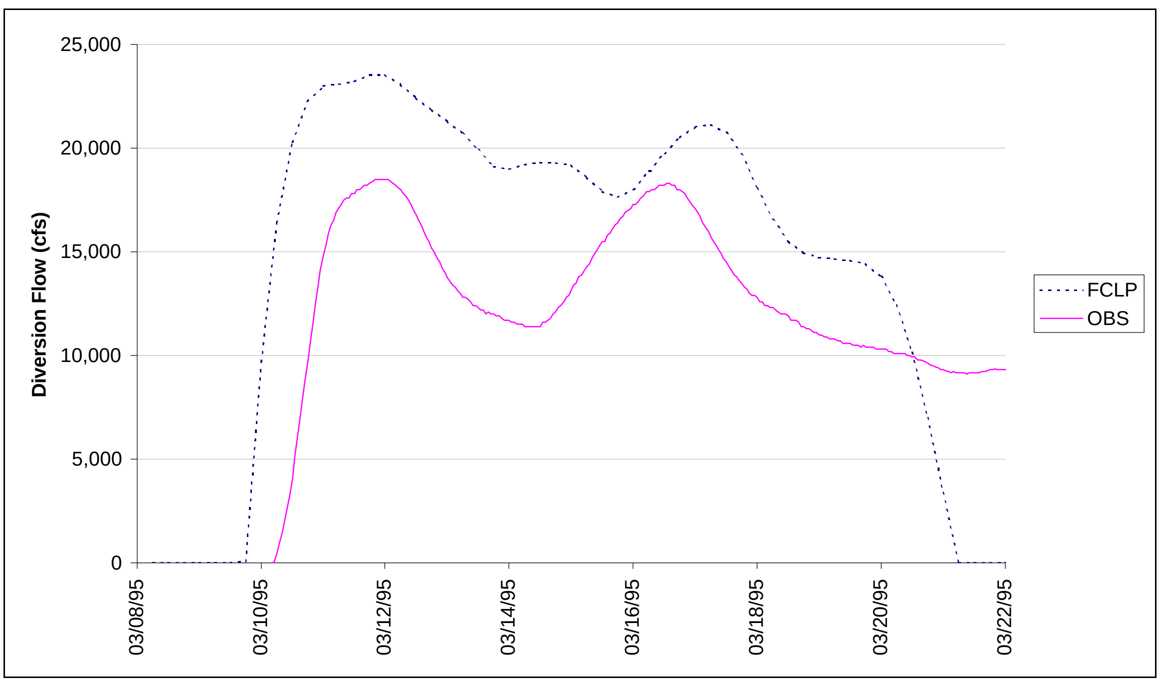

Figure 63. Flow over Moulton Weir.

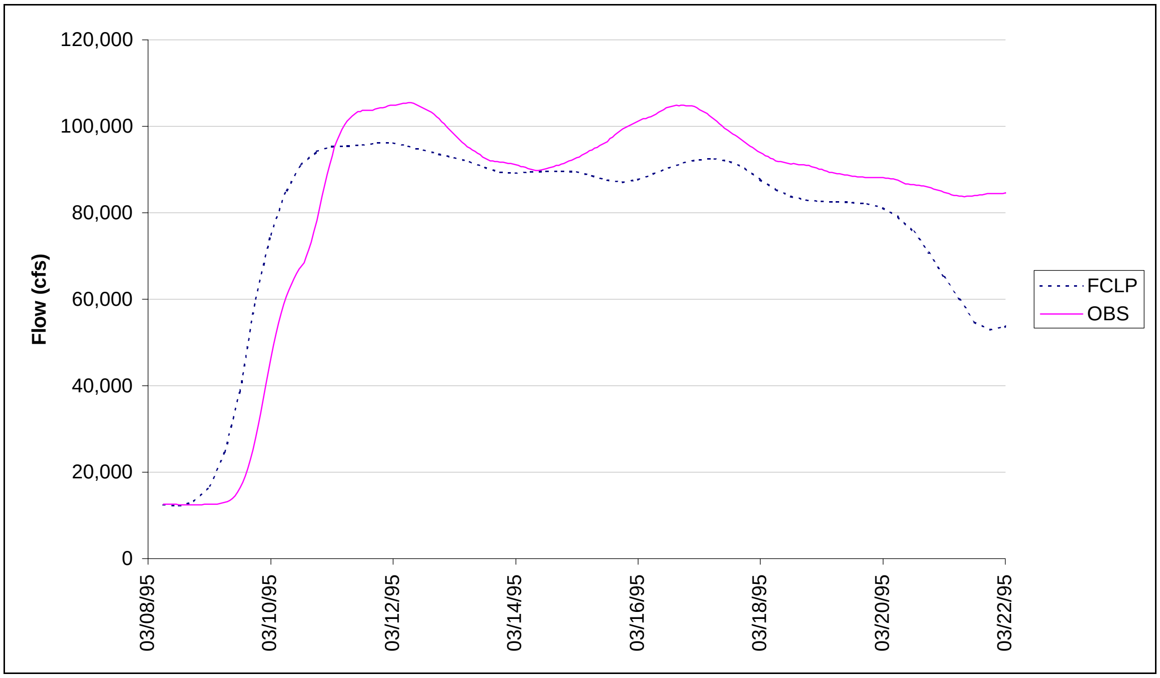

Figure 64. Flow past Moulton Weir in Sacramento River.

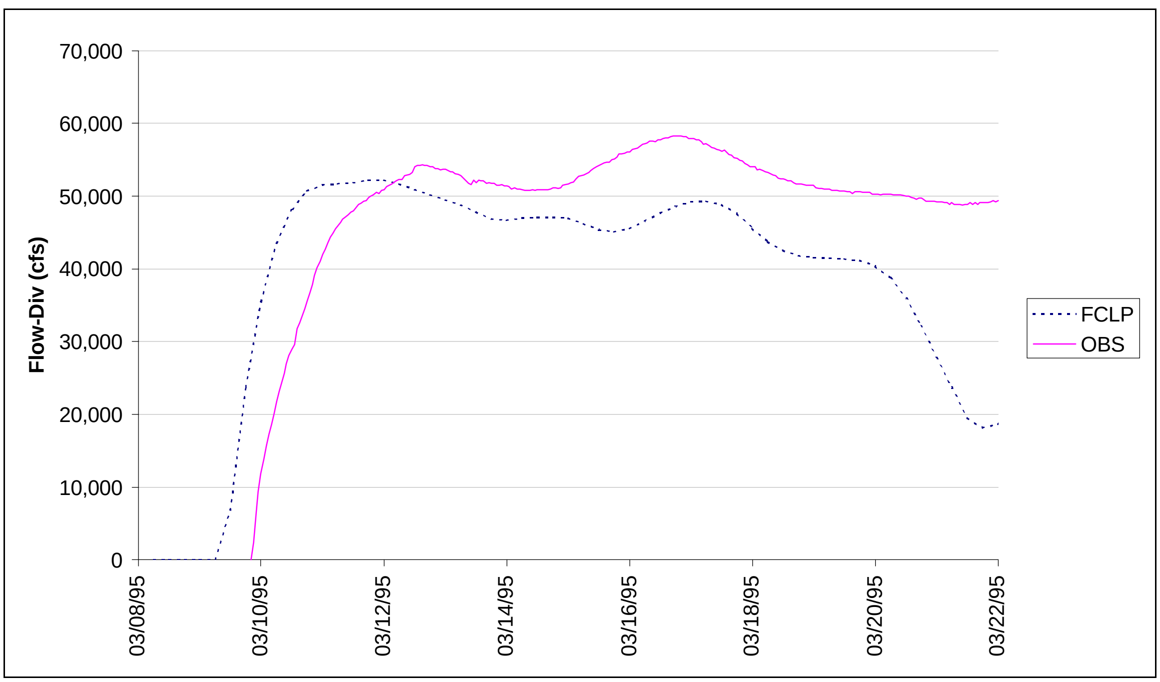

Figure 65. Flow over Colusa Weir.

Figure 66. Flow past Colusa Weir in Sacramento River.

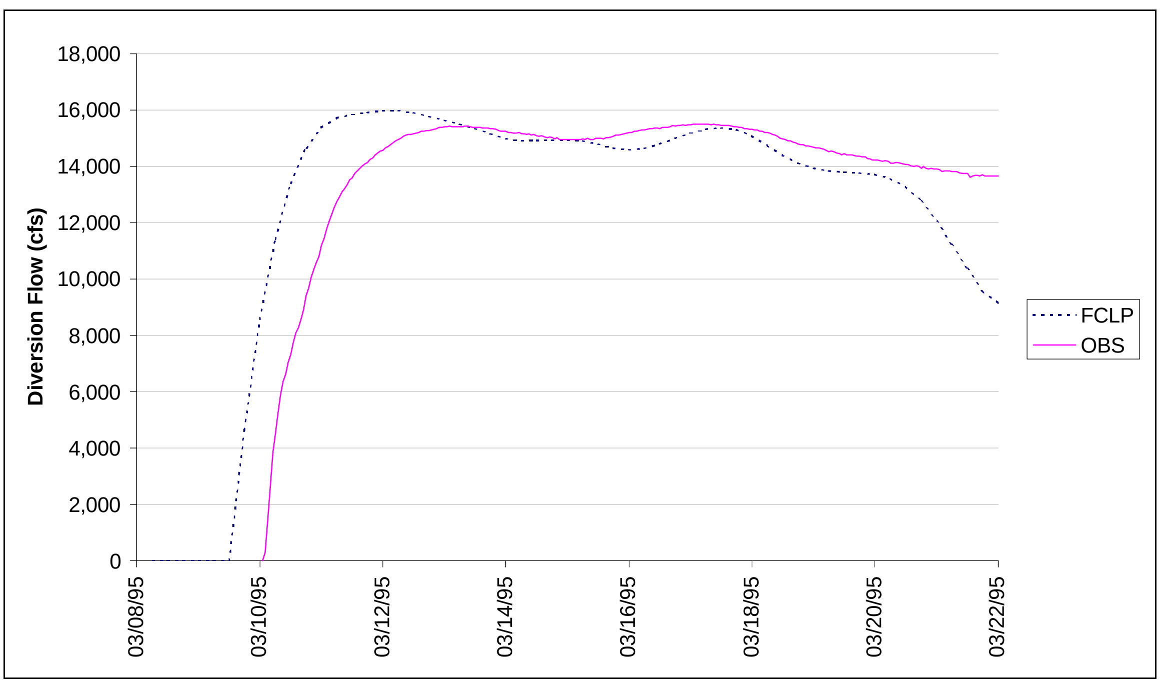

Figure 67. Flow over Tisdale Weir.

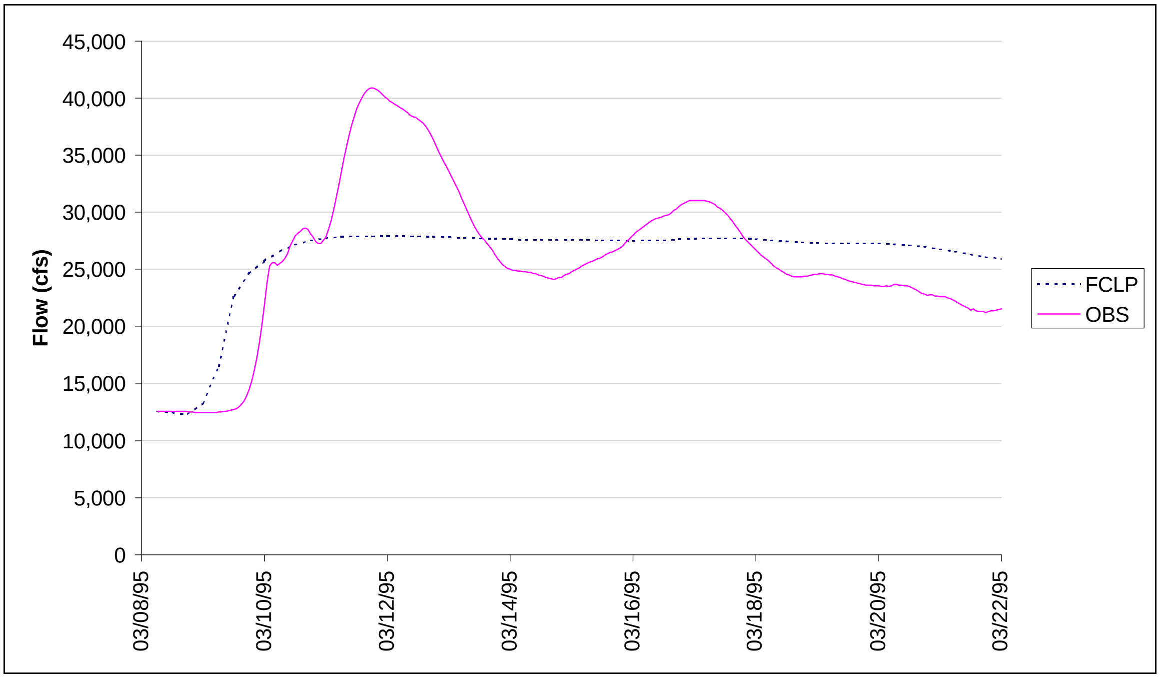

Figure 68. Flow past Tisdale Weir in Sacramento River.

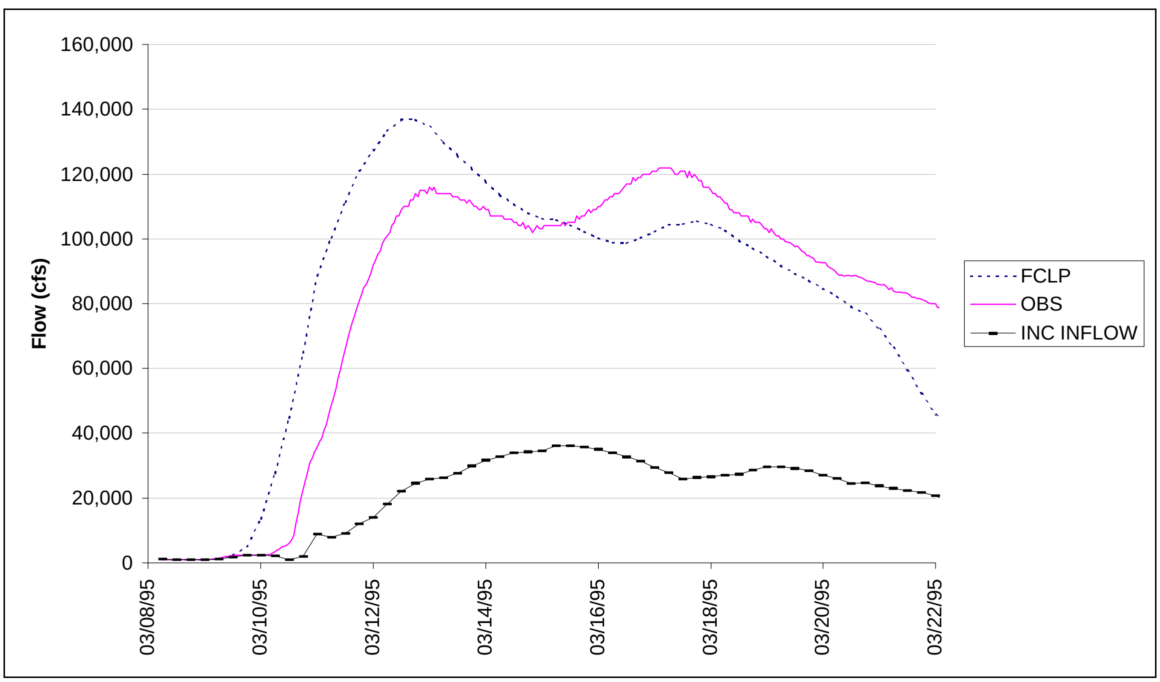

Figure 69. Flow at Meridian.

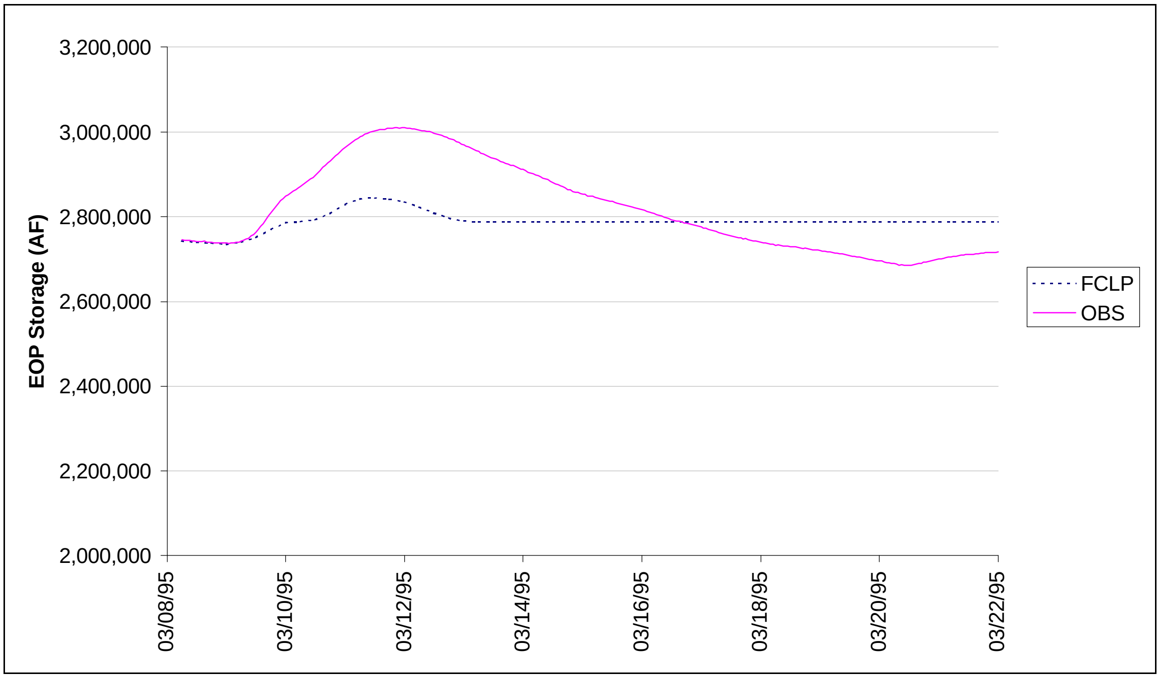

Figure 70. Oroville Dam EOP storage.

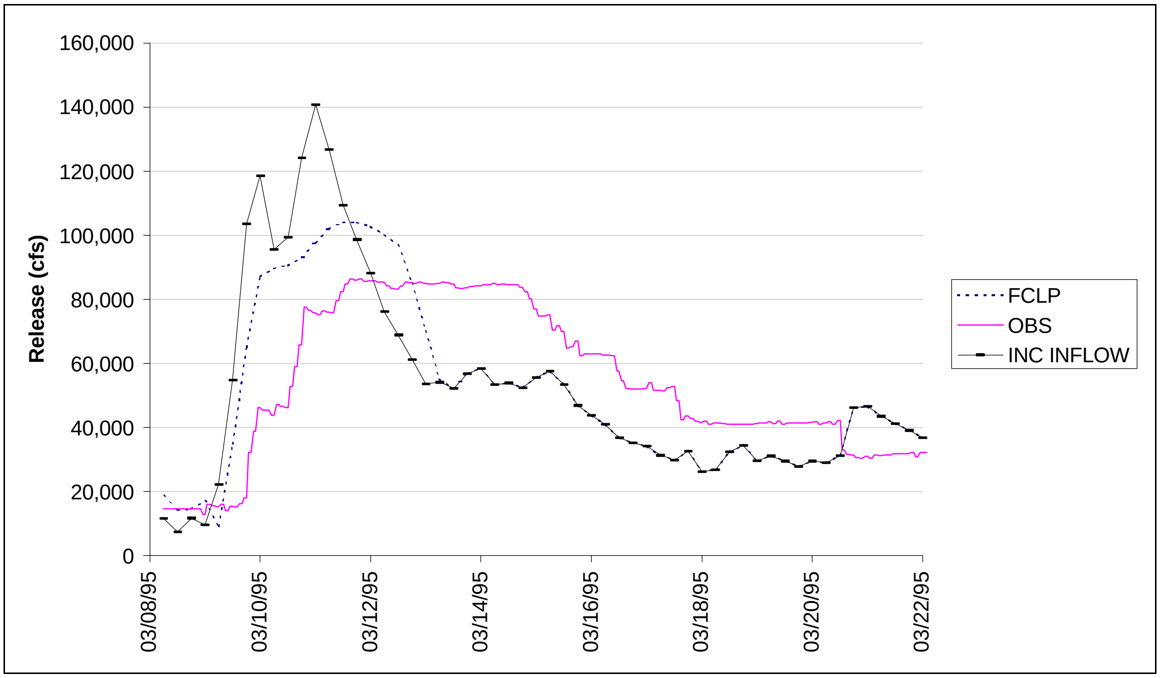

Figure 71. Oroville Dam release.

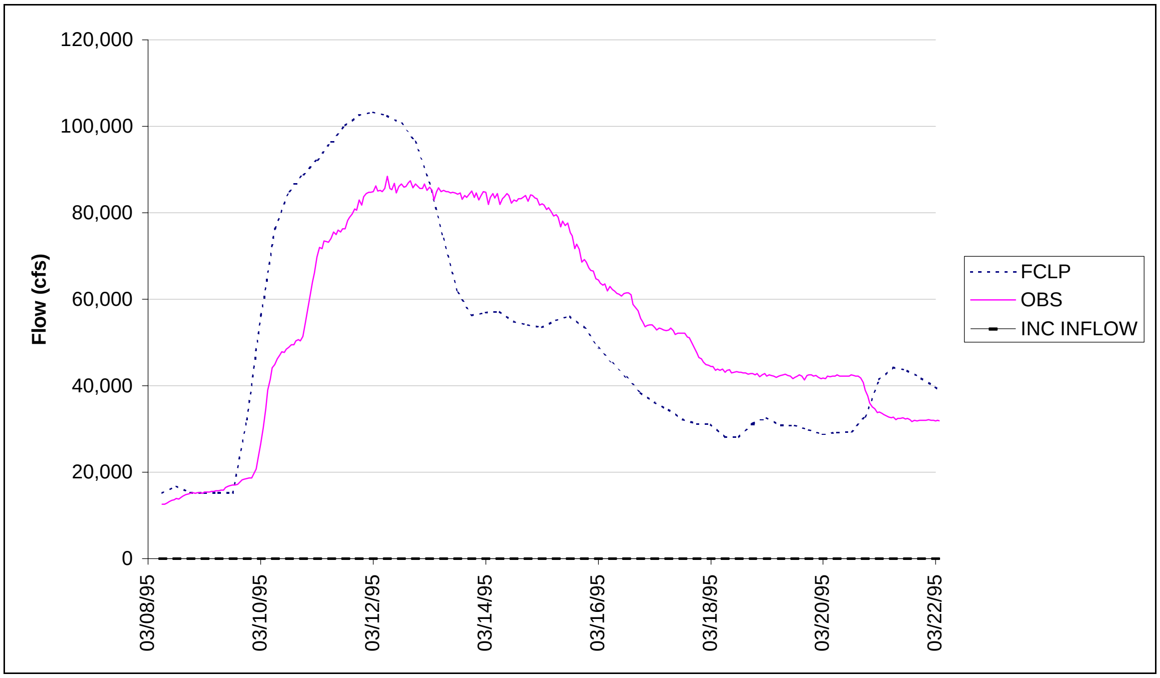

Figure 72. Flow at Gridley.

Figure 73. Flow at Yuba City.

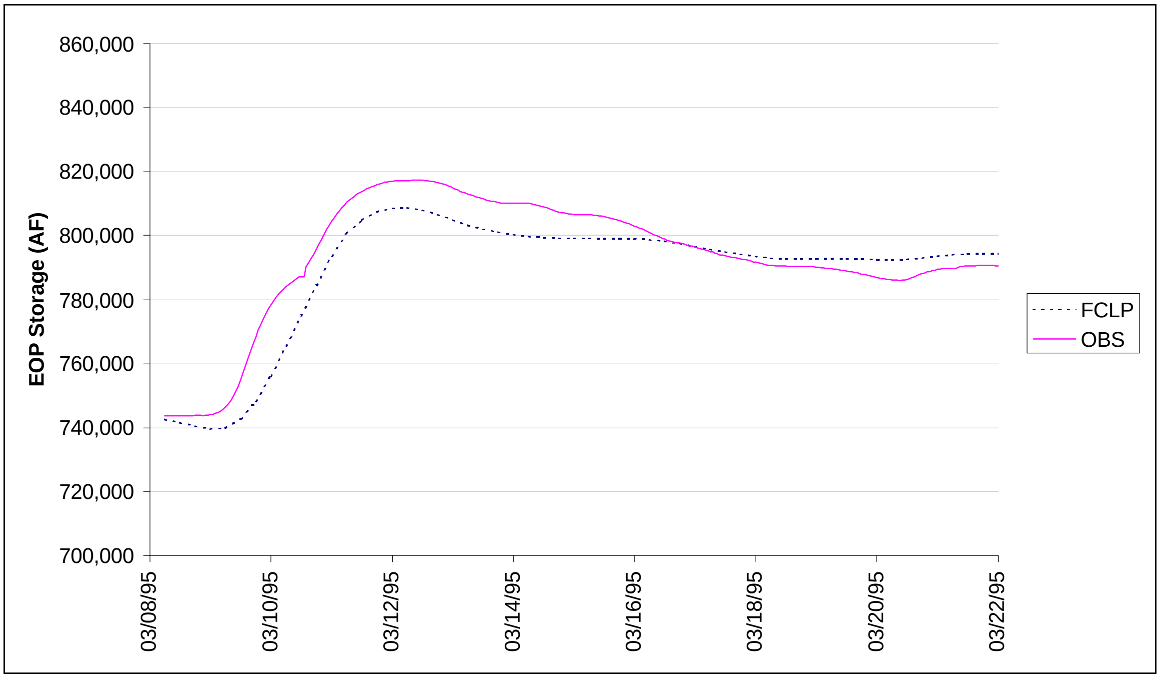

Figure 74. New Bullards Bar EOP storage.

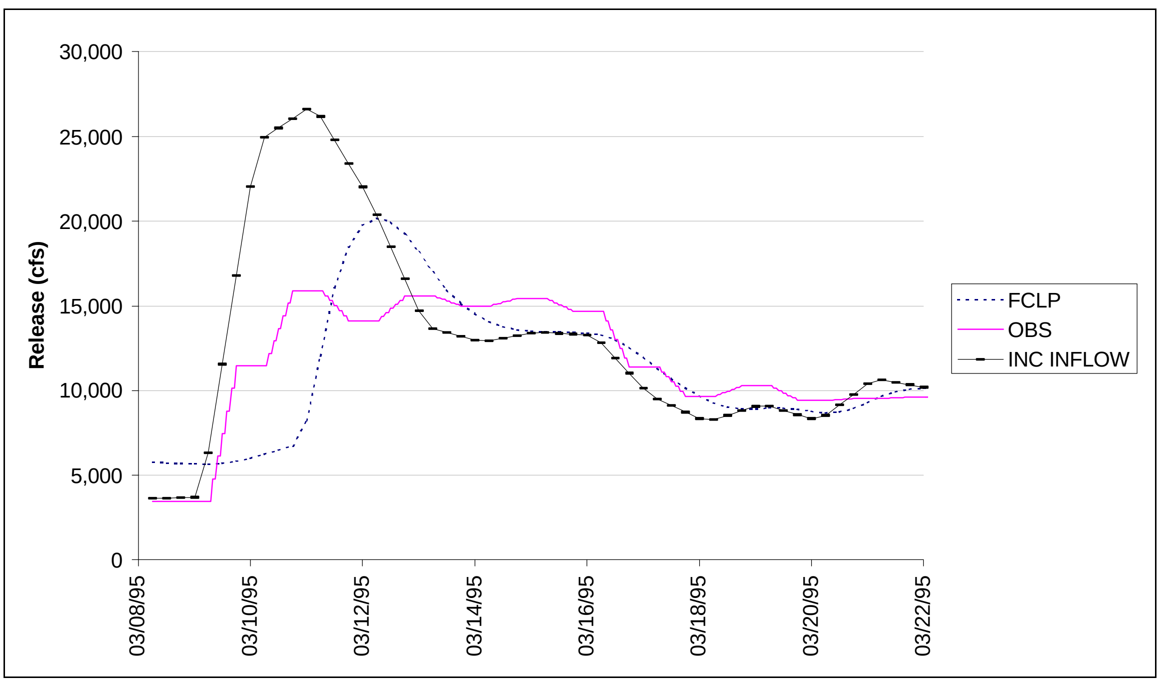

Figure 75. New Bullards Bar Dam release.

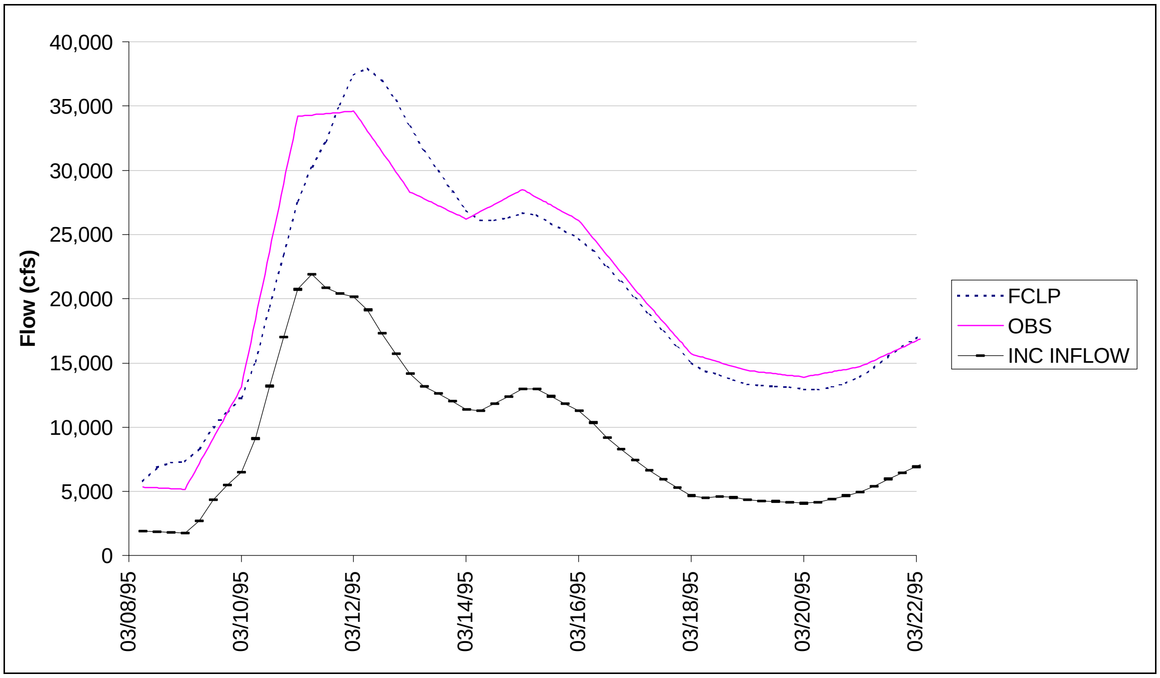

Figure 76. Flow at Marysville.

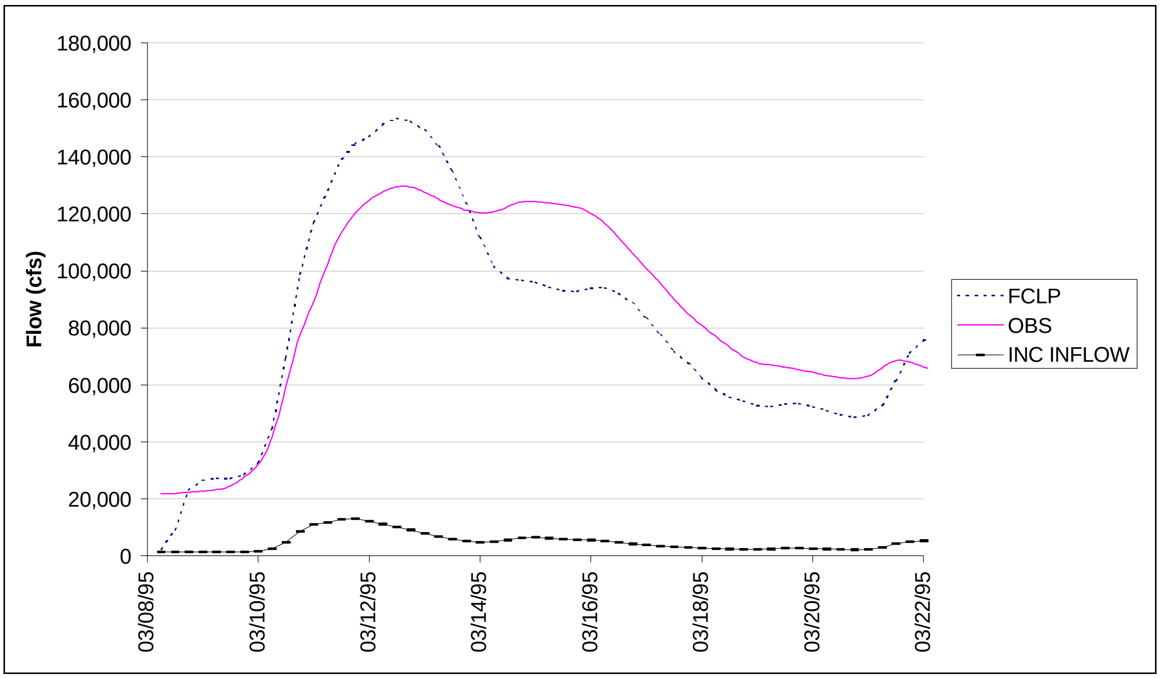

Figure 77. Flow at Nicolaus.

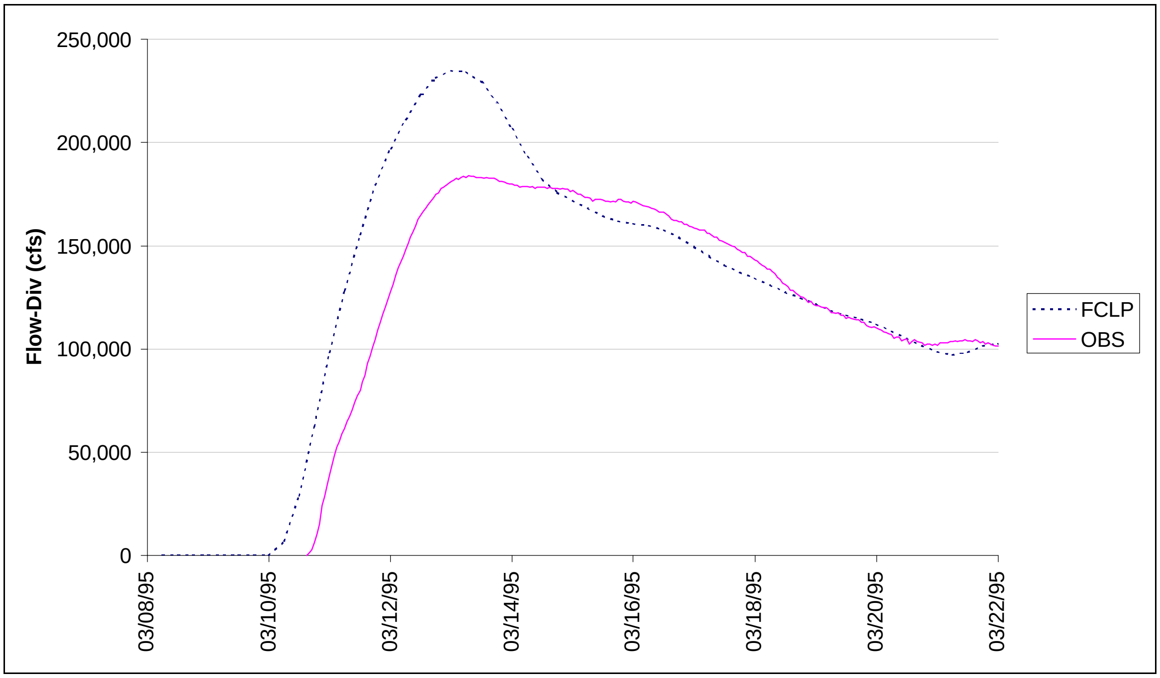

Figure 78. Flow over Fremont Weir into Yolo Bypass.

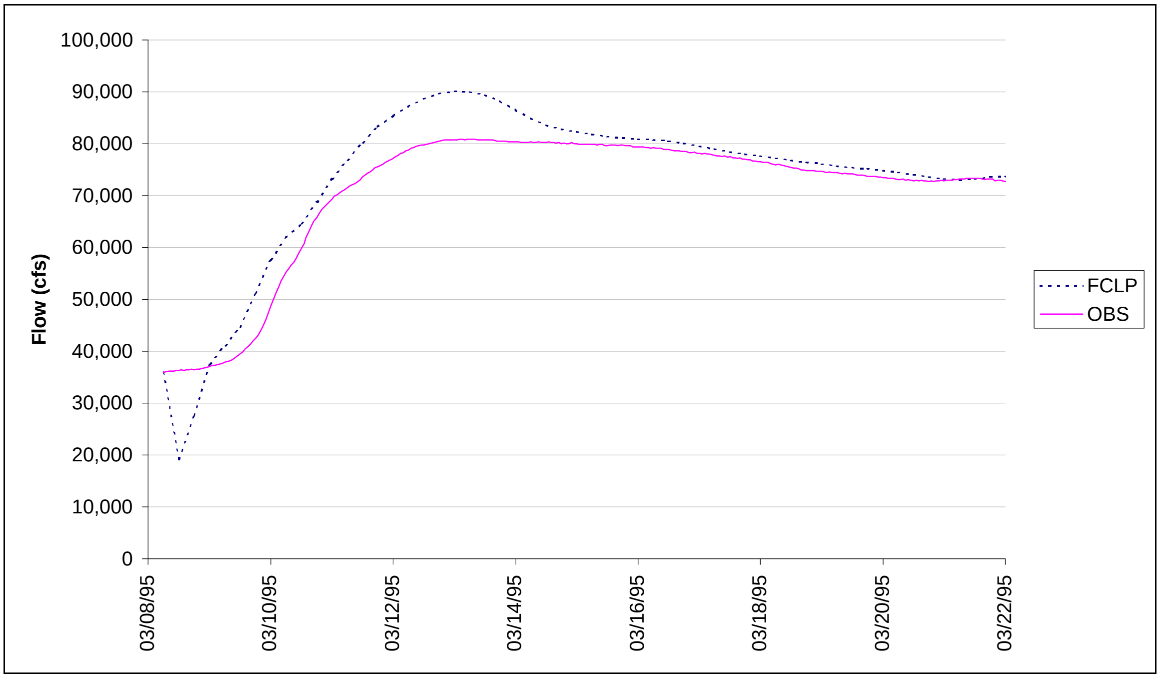

Figure 79. Flow past Fremont Weir/Verona in Sacramento River.

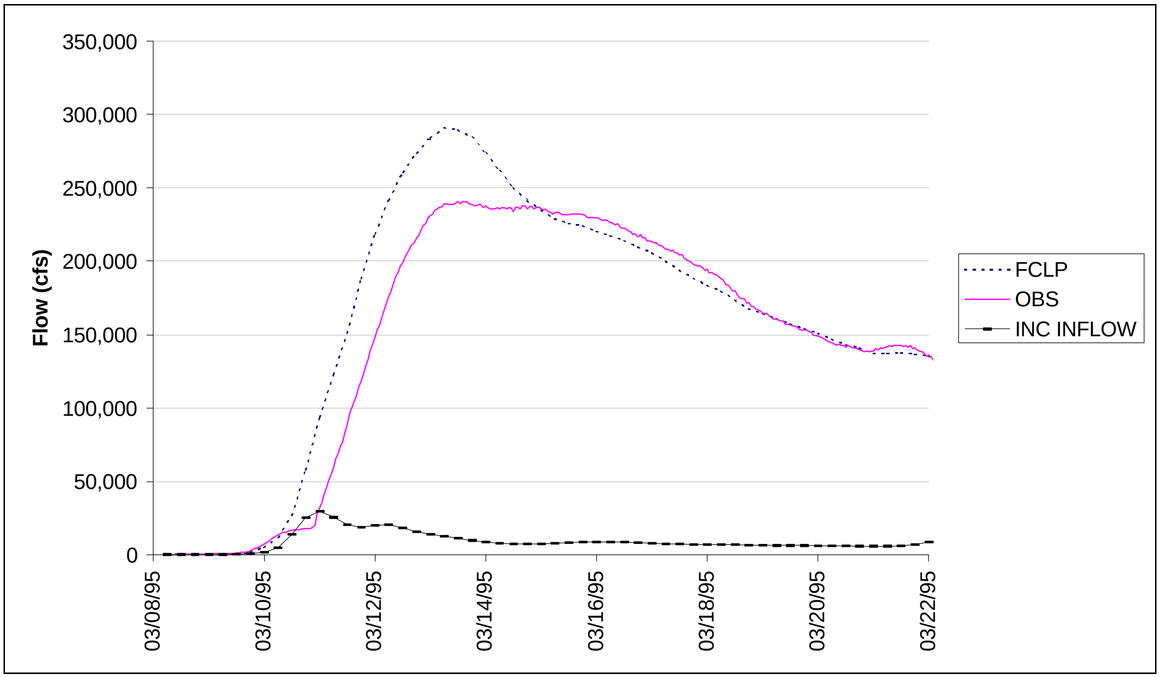

Figure 80. Flow at Woodland.

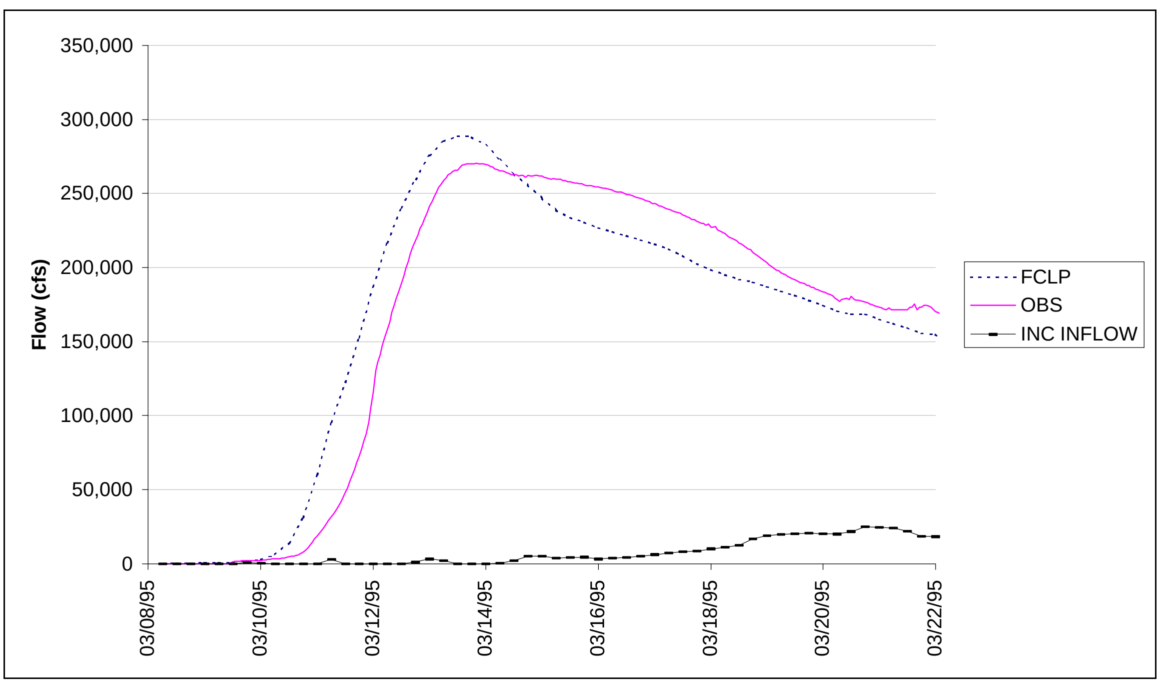

Figure 81. Flow at Lisbon.

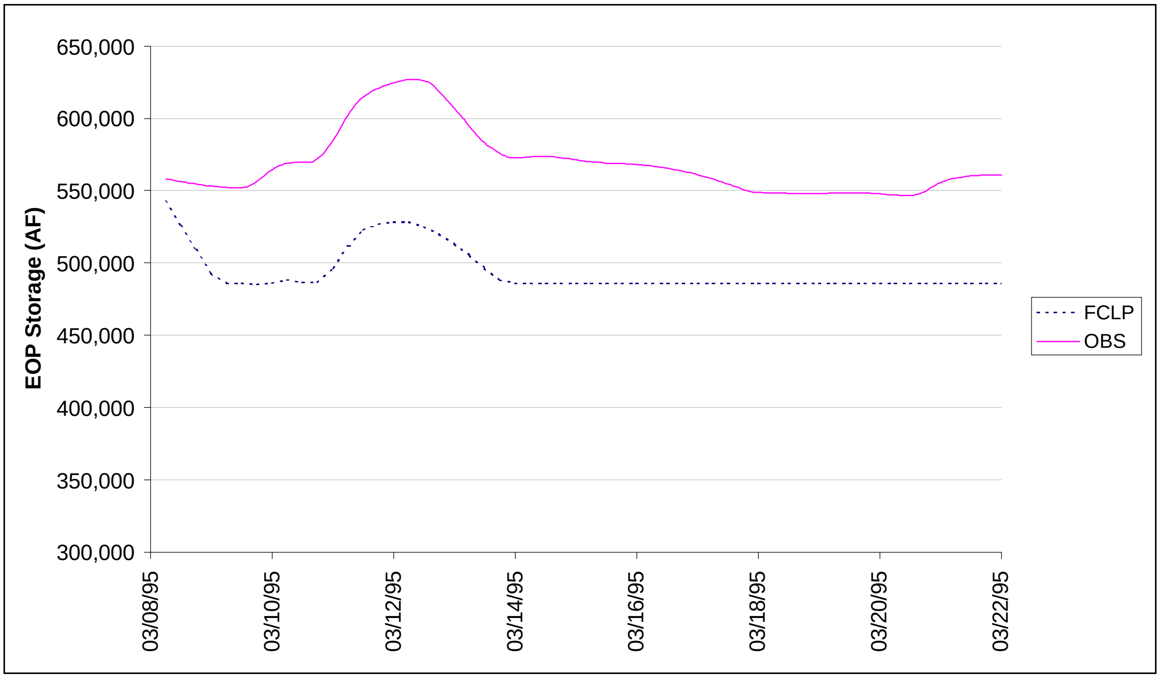

Figure 82. Folsom Dam EOP storage.

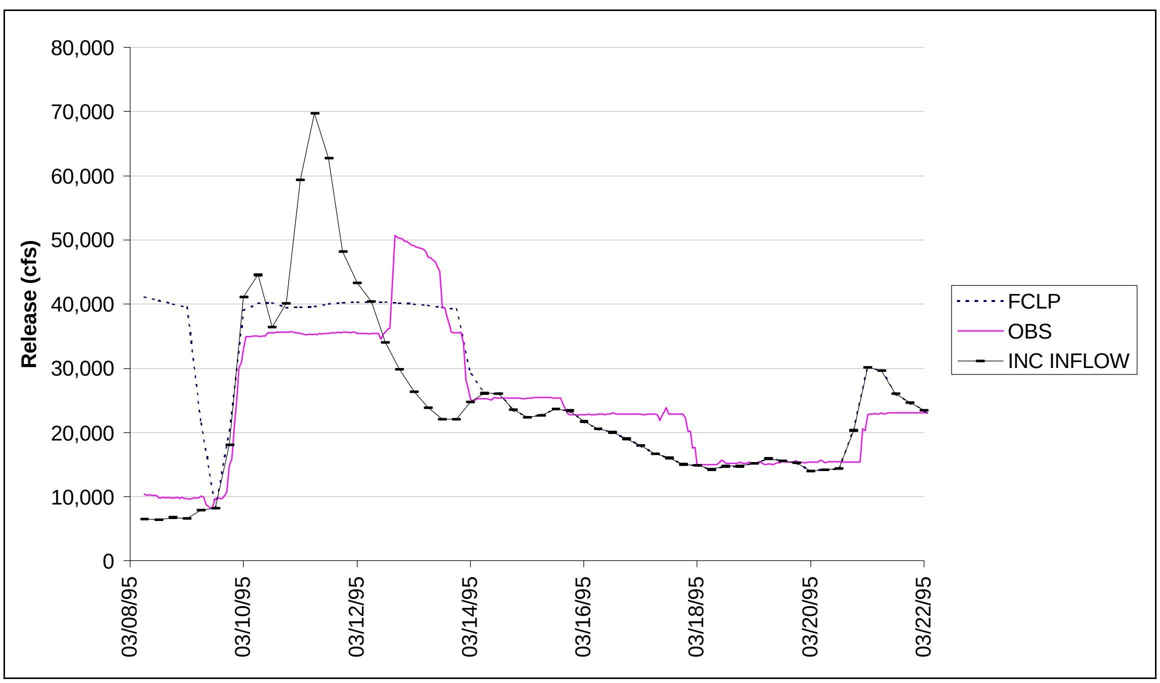

Figure 83. Folsom Dam release.

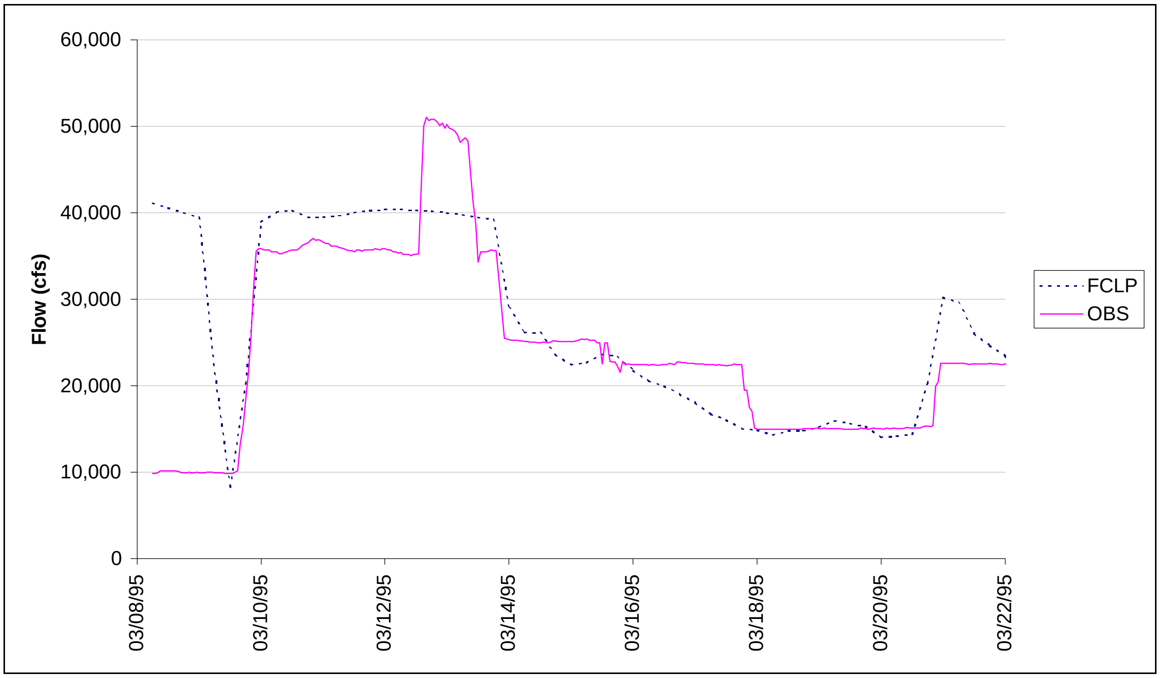

Figure 84. Flow at Fair Oaks.

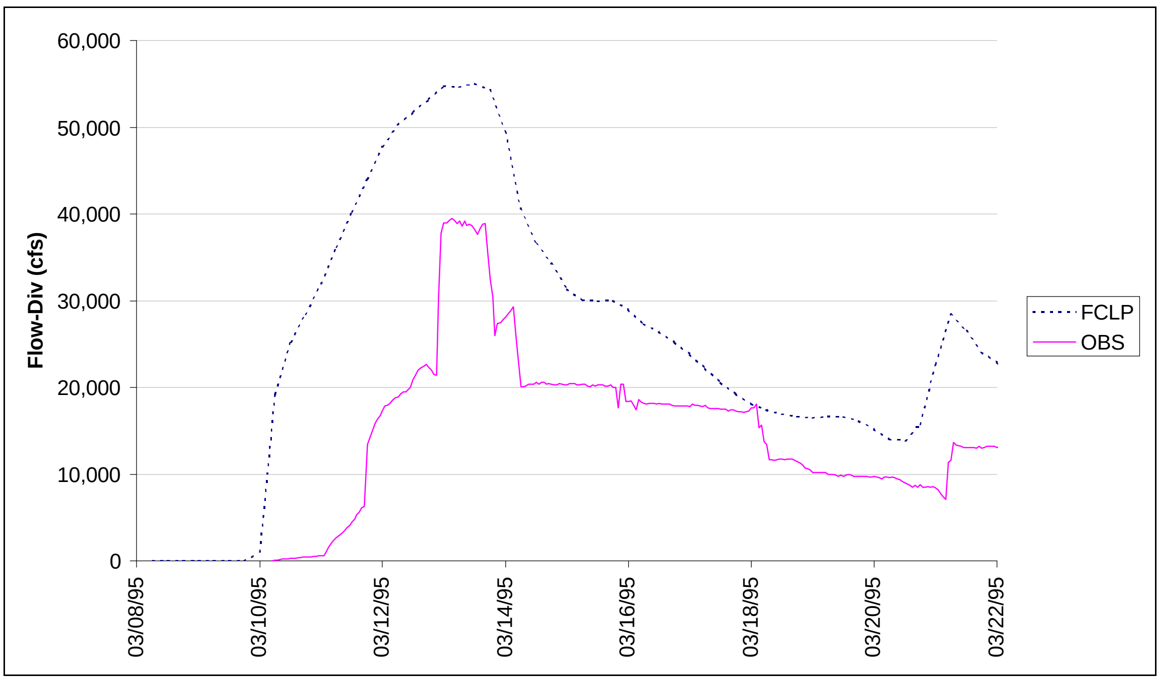

Figure 85. Flow over Sacramento Weir into Yolo Bypass.

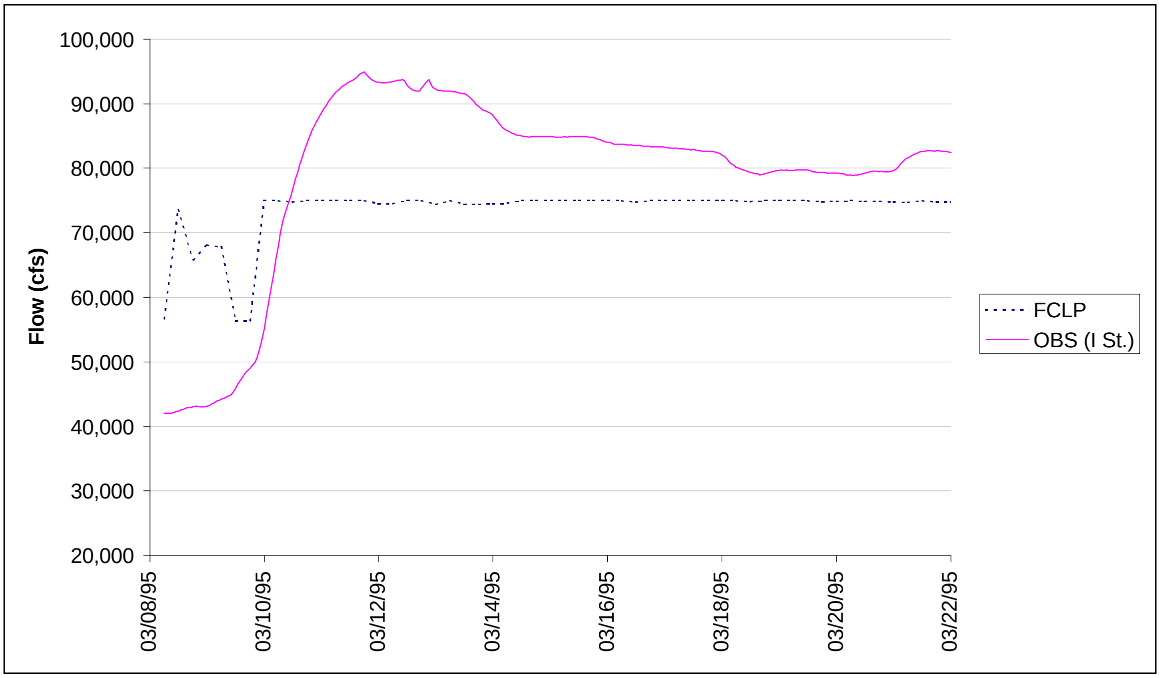

Figure 86. Flow past Sacramento Weir in the Sacramento River.

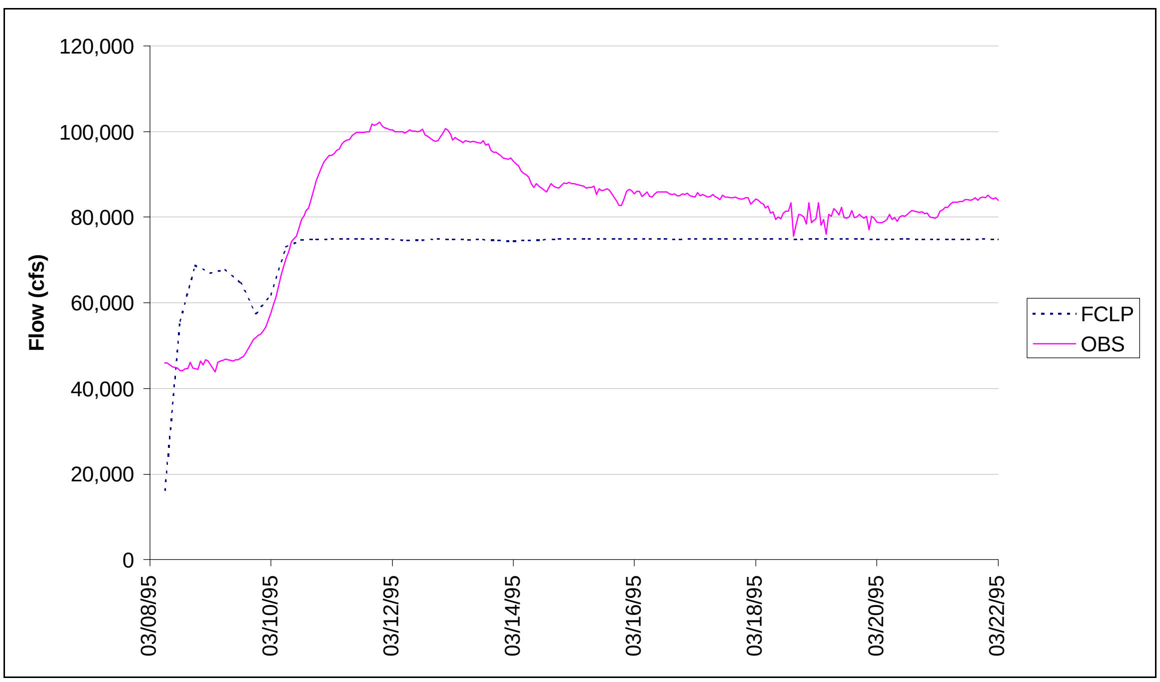

Figure 87. Flow at Freeport.

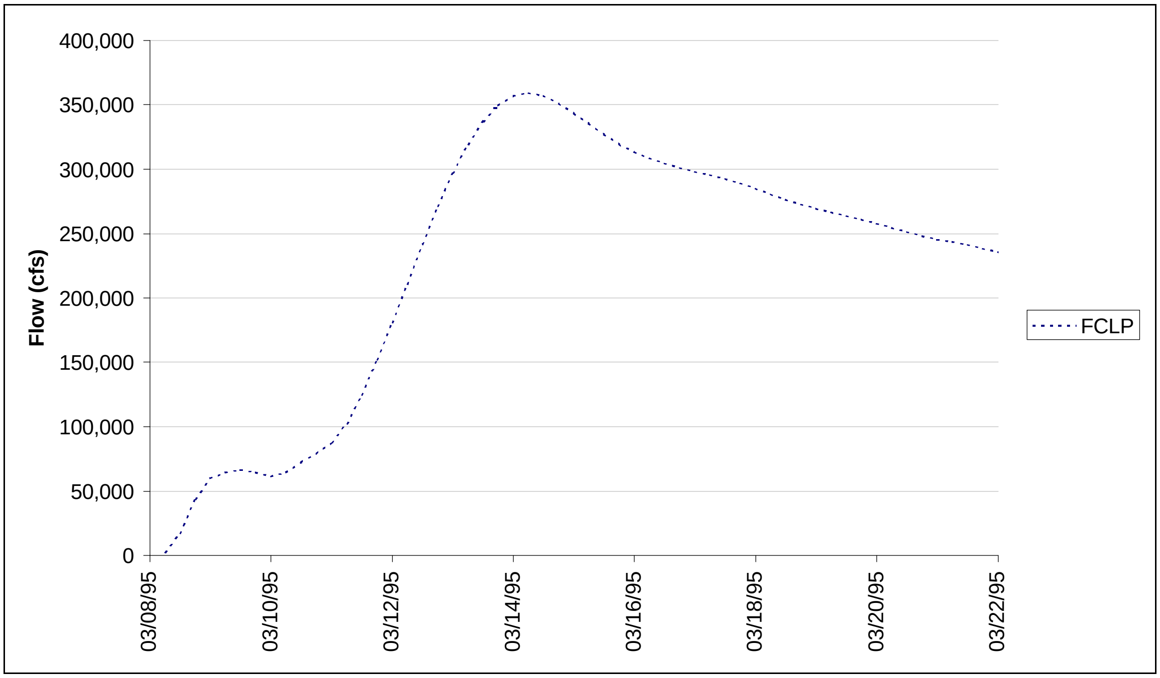

Figure 88. Model flow at Rio Vista.

Chapter 7

SUBSYSTEM ANALYSIS

Analyses were done using two subsystems to see if independent operation of Shasta Dam and the Oroville and New Bullards Bar dams would produce the same results as coordinated system operation. Based on analyses of the 1995 and 1997 results, it was assumed that the operation of Shasta Dam has little effect below Bend Bridge. It was further assumed that the coordinated operation of Oroville Dam and New Bullards Bar Dam has negligible effects below Nicolaus. In both instances, the limited operating range is due to large incremental inflows entering the system below the dams. Two subsystems were set up, as shown in Figure 9, and an analysis was done using the 1997 and 1995 events to test these assumptions.

The 1997 model operation of the Shasta and Feather/Yuba subsystems agree reasonably well with the total system operation, as shown in Figures 89 through 91. The systemwide operation tends to reduce Shasta Dam releases sooner and for a longer time than for the Shasta subsystem alone, as shown in Figure 89. This implies that the system operates for points beyond Bend Bridge during this event. The Feather/Yuba operation is apparently affected by flows at the Fremont Weir/Verona control point, as shown by the operation of Oroville Dam and New Bullards Bar Dam in Figures 90 and 91.

The 1995 subsystem results, shown in Figures 92 through 94, also show that model operation of the Shasta subsystem is comparable to the entire system operation. The model operation of Oroville Dam and New Bullards Bar Dam in the Feather/Yuba subsystem, however, is the same as the complete system model operation.

Based on these results, the assumption that the Shasta and Feather/Yuba subsystems can be operated individually seems valid when there are high incremental inflows to the system, as in the 1995 event. However, this conclusion is based on the operation during only two particular flood events. These assumptions should be tested with numerous events and varying levels of incremental inflows for more general insights and conclusions.

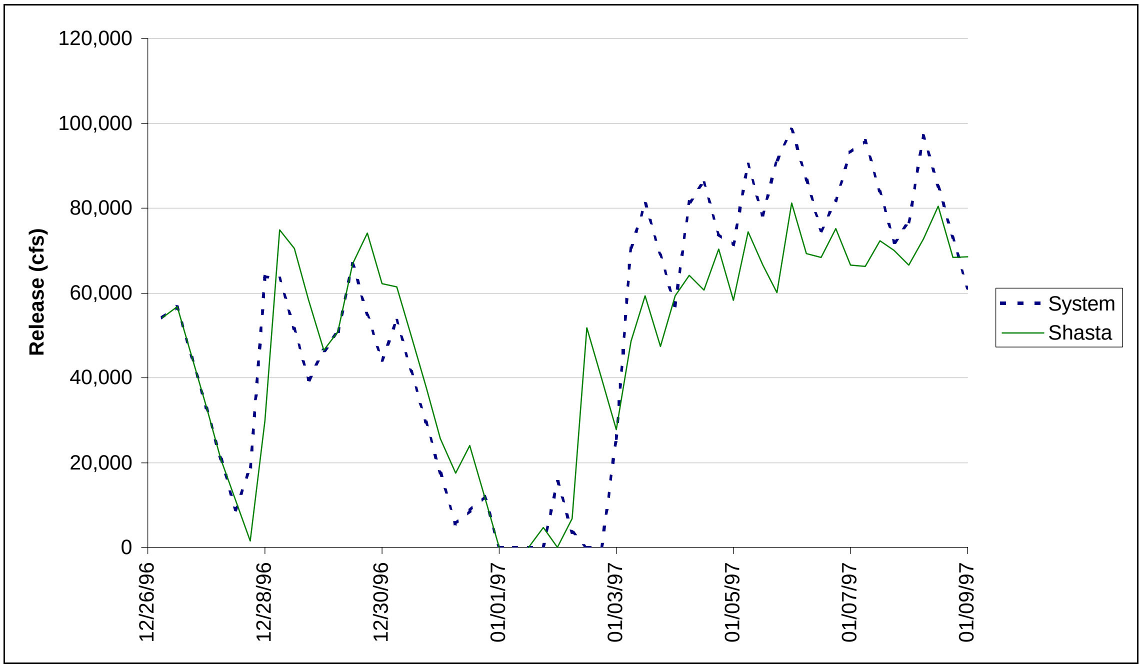

Figure 89. 1997 Shasta Dam releases under system and subsystem operation.

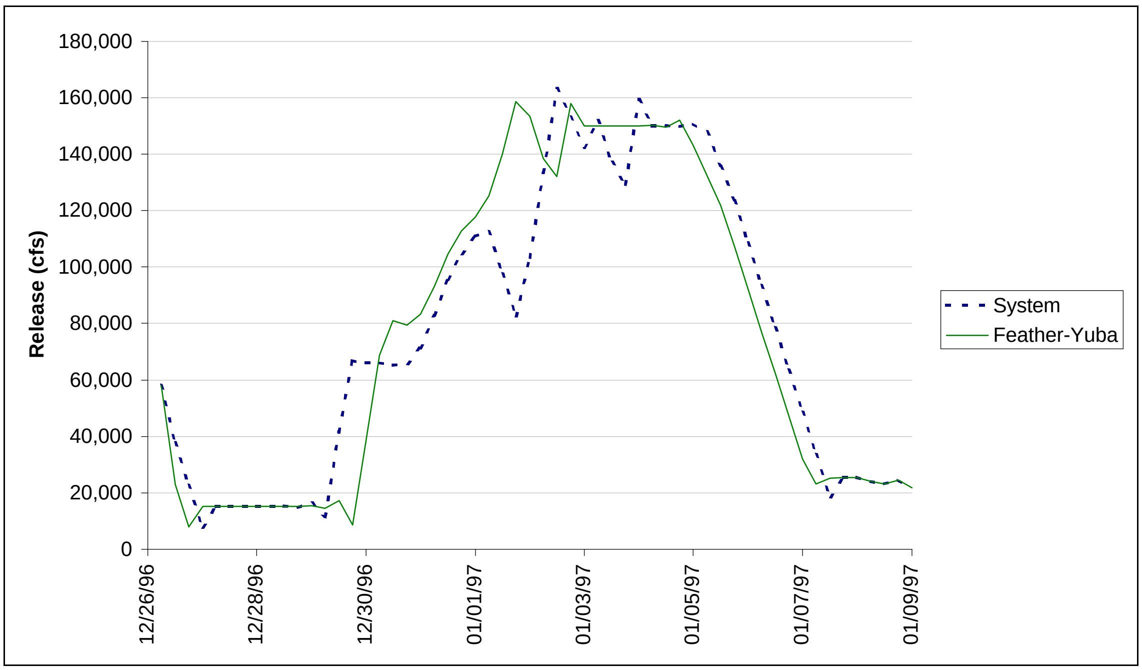

Figure 90. 1997 Oroville Dam releases under system and subsystem operation.

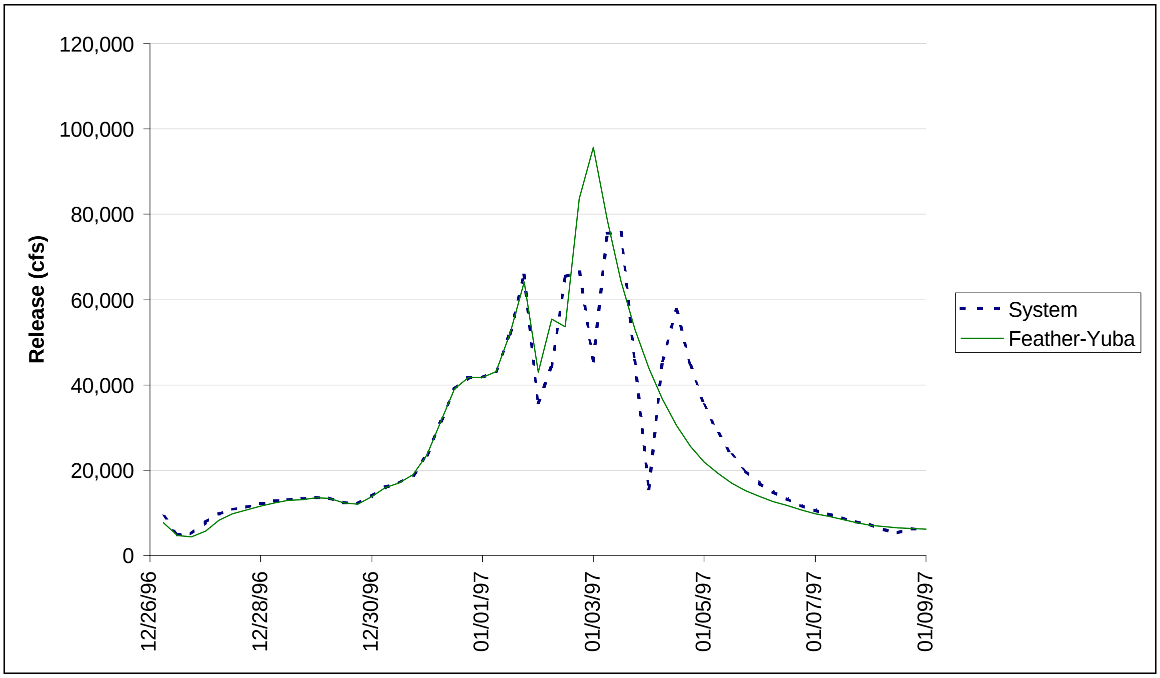

Figure 91. 1997 New Bullards Bar Dam releases under system and subsystem operation.

Figure 92. 1995 Shasta Dam releases under system and subsystem operation.

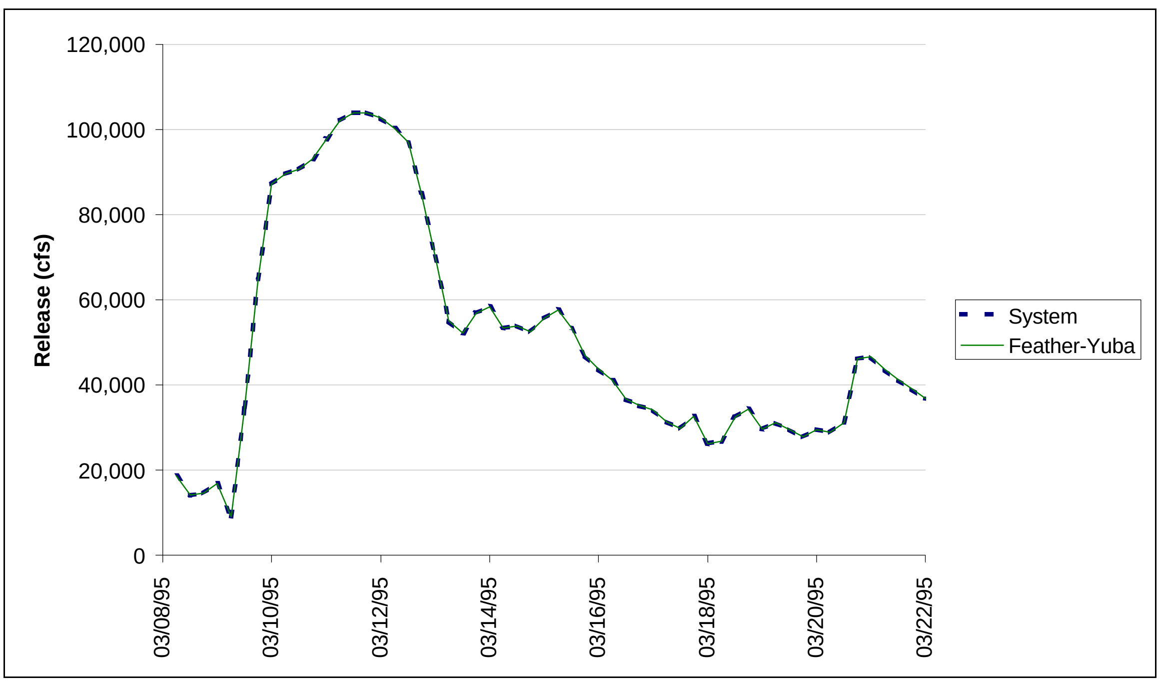

Figure 93. 1995 Oroville Dam releases under system and subsystem operation.

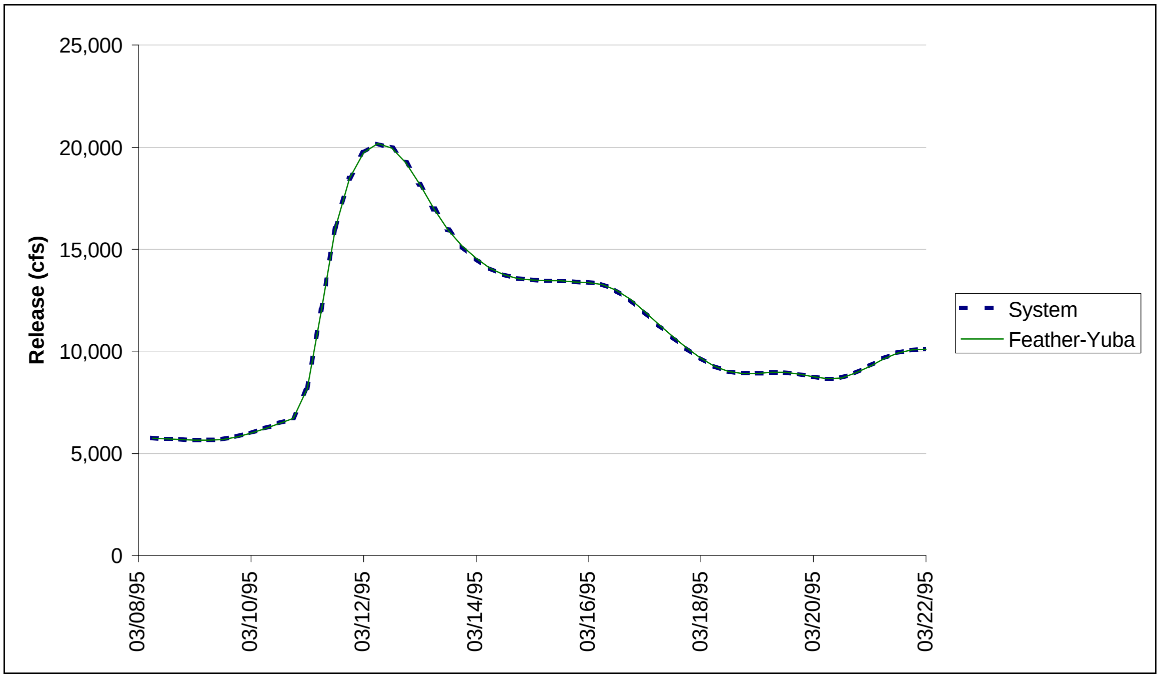

Figure 94. 1995 New Bullards Bar Dam releases under system and subsystem operation.

Chapter 8
CONCLUSIONS

As expected, the optimization model is generally able to reduce peak flows throughout the system by coordinating reservoir releases and anticipating inflows. Although the preliminary results appear to validate the model, in the sense that any differences between computed and observed flows can be attributed to data limitations and the objectives of the mathematical programming problem, the optimization results must be interpreted carefully.

As with any deterministic optimization model, the results represent an ideal operation that is usually unachievable in practice. The model does not consider uncertainty in flood routing, channel capacities, or even inflow forecasts. In essence, for each reservoir, the optimization model uses knowledge of the entire sequence of inflows to determine the entire sequence of releases. In several instances the model used higher releases at the beginning of an event to clear storage for upcoming inflows. This would not be done in practice due to the limited foresight of the operators, who must make decisions using imperfect inflow forecasts.

The optimization results obtained in this study have been encouraging. However, it is recommended that a more thorough study be done using refined data. This would include a more detailed analysis of damage potential for each impact area in the Sacramento Basin. Once this has been accomplished, the optimization model should be applied to several events which vary in both magnitude and duration. These could be historical events or hypothetical events of various occurrence frequencies. Post-processing should then be done to infer improved system operating rules. The optimization model could also be used to estimate the potential benefits of structural enhancements, as well as insights to revised operating policies accounting for the physical changes to the system. However, it must be emphasized that results obtained from optimization methods should be verified with more precise simulation models. This is especially true when inferring new operating rules. Due to the limited data available for this study, no attempt was made to infer new operating rules.

The use of mixed-integer programming for flood control is an improvement over previous methods that were unable to represent accurately operational constraints (i.e. outlet capacity curves and diversion functions). A limitation of the MIP approach, however, is the excessive computation time required to solve complex systems. Simplifications have to be made to solve the model in a practical amount of time. In this study, it was assumed that certain reservoirs will not spill and that all damage functions are convex, thereby reducing the number of binary variables and the number of nodes that are searched in the branch-and-bound algorithm.

Working with subsystems also is useful for reducing solver times. Since the subsystems comprise fewer components (and fewer binary variables) than the complete system, they can be optimized relatively quickly. The analyses of the Shasta and Feather/Yuba subsystems imply that these portions of the complete system can be optimized separately when incremental inflows to the system are high. The optimal subsystem results can then be combined for final analysis of the complete system using flow conservation at the connecting nodes. Before subsystems are used in an analysis, however, it must be determined by complete system optimization that model operation of the subsystems truly is independent.

REFERENCES

Georgakakos, A. P., H. Yao, M. G. Mullusky, and K. P. Georgakakos (1998). Impacts of climate variability on the operational forecast and management of the upper Des Moines River basin, *Water Resources Research*, 34(4): 799-821.

Hillier, F. S., and G. J. Lieberman (1990). *Introduction to Operations Research*, 5th ed., McGraw-Hill, Inc., New York.

Ikura, Y., and G. Gross (1984). Efficient large-scale hydro system scheduling with forced spill conditions, *IEEE Trans. on Power App. and Sys.*, PAS-103(12): 3502-3520.

Israel, M. S., and J. R. Lund (1999). Priority Preserving Unit Penalties in Network Flow Modeling, *Journal of Water Resources Planning and Management*, ASCE (in press).

Karbowski, A. (1993). Optimal Flood Control in Multireservoir Cascade Systems with Deterministic Inflow Forecasts, *Water Resources Management*, 7: 207-223.

Optimization Subroutine Library Release 2.1, *Guide and Reference* (1995). IBM Corporation, Poughkeepsie, NY.

U.S. Army Corps of Engineers (1970). *Oroville Dam and Reservoir: Report on Reservoir Regulation for Flood Control*, Sacramento District, Sacramento, Calif.

U.S. Army Corps of Engineers (1972). *New Bullards Bar Reservoir: Report on Reservoir Regulation for Flood Control*, Sacramento District, Sacramento, Calif.

U.S. Army Corps of Engineers (1977). *Cottonwood Creek: Hydrology Report*, Sacramento District, Sacramento, Calif.

U.S. Army Corps of Engineers (rev 1977). *Shasta Dam and Lake: Report on Reservoir Regulation for Flood Control*, Sacramento District, Sacramento, Calif

U.S. Army Corps of Engineers (1987). *Black Butte Dam and Lake: Water Control Manual*, Sacramento District, Sacramento, Calif.

U.S. Army Corps of Engineers (1987). *Folsom Dam and Lake: Water Control Manual*, Sacramento District, Sacramento, Calif.

U.S. Army Corps of Engineers (1993). *Sacramento River Flood Control Project Hydrology*. Civil Projects Branch, U.S. Army Corps of Engineers, Sacramento District, Sacramento, Calif.

U.S. Army Corps of Engineers (1994). *Operating Rules from HEC-PRM Results for the Missouri River System: Development and Preliminary Testing*, Hydrologic Engineering Center, Davis, Calif.

U.S. Army Corps of Engineers (1995). Annual District Water Control Management Report, Sacramento District, Sacramento, Calif.

U.S. Army Corps of Engineers (1995). “HEC-DSS, User’s Guide and Utility Manuals,” *CPD-45*, Hydrologic Engineering Center, Davis, Calif.

U.S. Army Corps of Engineers (1997). Annual District Water Control Management Report, Sacramento District, Sacramento, Calif.

U.S. Army Corps of Engineers (1998). “HEC-5 Simulation of Flood Control and Conservation Systems, User’s Manual Ver 8.0,” *CPD-5*, Hydrologic Engineering Center, Davis, Calif.

U.S. Army Corps of Engineers (1999). *Analysis of Flood Control Operation of the Iowa/Des Moines River Reservoir System Using Linear Programming Techniques PR-38*, Hydrologic Engineering Center, Davis, Calif.

Unver, O. I., and L. W. Mays (1990). Model for Real-Time Optimal Flood Control Operation of a Reservoir System, *Water Resources Management*, 4: 21-46.

Wasimi, S. A., and P. K. Kitanidis (1983). Real-time forecasting and daily operation of a multireservoir system during floods by linear quadratic Gaussian control, *Water Resources Research*, 19(6): 1511-1522.

Watkins, D. W., D. J. Jones, and D. T. Ford (1999). “Flood control optimization using mixed-integer programming,” *26th Annual Water Resources Planning & Management Conference*, ASCE, Tempe, AZ.

Windsor, J. S. (1973). Optimization models for the operation of flood control systems, *Water Resources Research*, 9(5): 1219-1226.

Wurbs, R. A. (1993). Reservoir-System Simulation and Optimization Models, *Journal of Water Resources Planning and Management*, ASCE 119(4): 455-472.

Yeh, W. W.-G. (1986). Reservoir Management and Operations Models: A state of the art review, *Water Resources Research*, 21(12): 1797-1818.

Appendix A

MODEL INPUT

FCLP input description

This appendix describes the input required for the FCLP program. This input is in the form of an ASCII text file. That file follows the standard format established by the HEC for its computer programs:

- Each line in the file (record) consists of 80 characters and is divided into fields.
- Field 0 consists of the first two characters. These characters identify the type of information included on the record.
- Field 1 consists of characters 3-8. Fields 2-10 consists of 8 characters each.
- Numbers are right justified in each field.

Each input record required for FCLP is described briefly herein. Required records are indicated with double asterisks (**); optional records are indicated with a single asterisk (*).

To the extent possible, the FCLP input is based on input required for the Corps' program HEC-5. Exhibit 8 of the HEC-5 user's manual describes the HEC-5 input in detail. Here we have repeated parts of that description as appropriate, noting fields of the HEC-5 records that are not used by FCLP. Any HEC-5 records not described herein are not necessary for FCLP; if they are included in the input, they are ignored. In a few cases, additional records are required to provide information specific to FCLP. Those records are described in more detail here.

B.1 DOCUMENTATION RECORDS

*T1, T2, T3 records***

Three job title records are required. Both alphabetic and numeric information may be entered in columns 3-80 of these records. Information on these records is printed in the output for the user's reference.

*C records**

These are optional comment records. They may be included anywhere within the input file to provide documentation of the input data. The record includes C in column 1, blank in column 2, and any alphabetic and numeric information in columns 3-80. The comment record is printed with the input listing at the beginning of the FCLP output.

B.2 JOB RECORDS

*J1 record***

Field	HEC-5 variable name	Value	Description
0		J1	Card identification.
1-2			Not used by FCLP.
3	NULEV	+	Number of index levels used in specifying storage penalty functions for project purposes and in apportioning reservoir releases amongst reservoirs.
4-9			Not used by FCLP.
10	NFL (new for FCLP)		Number of index levels used in specifying flow penalty functions for project purposes and in apportioning reservoir releases amongst reservoirs. Note that FCLP automatically adds one additional flow zone that represents all flow in excess of the final flow level.

*J3 record***

All values in fields 1-9 of the J3 record are ignored by FCLP. Field 10 controls the printed output from FCLP. Three levels are available, each providing more output.

Field	HEC-5 variable name	Value	Description
0		J3	Card identification.
1-9			Not used by FCLP.
10	LPPRINT (new for FCLP)	1, 2, or 3	Output level. Level 1 is minimum output, 3 is maximum, and 2 is intermediate.

*BF record***

Following the HEC-5 precedent, the BF record defines the time period for analysis.

Field	HEC-5 variable name	Value	Description
0		BF	Card identification.
1			Not used by FCLP.
2	NPER	+	Number of periods of flow data (forecast).
3			Not used by FCLP.
4	CNSTI	+	Factor which is multiplied times all inflows and local flows.
5	FLDAT	+	Date corresponding to the beginning of the time interval of the first flow. This date is specified as an 8-digit number: 2 digits for year, month, day, and hour, respectively. Thus 54120223 represents December 2, 1954, 11 PM.
6			Not used by FCLP.
7	IPER	+	Time interval, in hours, between data in all time series. Must be in whole hours, > 1.
8-9			Not used by FCLP.
10	NBAK (new for FCLP)	+	Number of time periods to look back to establish initial flow conditions throughout the system. FCLP reads control point flows for NBAK periods *prior* to the first period of the analysis and routes these flows to establish the initial conditions throughout the system.

B.3 RECORDS FOR ALL RESERVOIRS

*RL record***

As with HEC-5, this record defines reservoir levels that define the manner in which system reservoirs are to be operated. FCLP input is limited to one RL record per reservoir.

Field	HEC-5 variable name	Value	Description
0		RL	Card identification.
1	MM	+	Control point identification number.
2	STOR1	+	Initial storage of reservoir MM in acre-feet.
3	STORL(1)	+	Cumulative reservoir capacity for level 1 for control point MM, in acre feet. This defines storage zone 1 as the storage between zero and STORL(1).
4	STORL(2)	+	Cumulative reservoir capacity for level 2 for control point MM, in acre-feet. This defines storage zone 2 as the storage between STORL(1) and STORL(2).
5-10 (as needed)	STORL(3) ... STORL(NULEV)	+	Cumulative reservoir capacity for each of NULEV levels (J1.3) for control point MM, in acre-feet. Each successive value defines a storage zone that is between that storage and the value in the previous field. Storage corresponding to level NULEV is assumed to be the reservoir capacity. FCLP will not prescribe an operation in which this value would be exceeded.

*S$ record (new for FCLP)***

This record defines the penalties for storage in zones that have been delineated by the reservoir levels specified on the RL record.

Field	HEC-5 variable name	Value	Description
0		S$	Card identification.
1-2			Not used by FCLP
3	PEN(1)	+	Penalty per ac ft of storage in zone delineated by STORL(1) in field 3 of RL record.
4	PEN(2)	+	Penalty per ac ft of storage in zone delineated by STORL(2) in field 4 of RL record.
5 - 10	PEN(3)... PEN(NULEV)	+	Penalty per ac ft of storage in zones delineated by STORL values in corresponding fields of RL record.

*RS record***

Values on RS and RQ records define the relationship of storage and maximum possible outflow.

Field	HEC-5 variable name	Value	Description
0		RS	Card identification.
1	NK	≥ 2	Number of values of that will be specified for the storage-outflow relationship for this reservoir.
2	STOR(1)	+	Reservoir capacity in acre-feet for first point on storage-outflow relationship for control point MM.
3-10 (as needed)	STOR(2) ... STOR(NK)	+	Reservoir capacity in acre-feet for remaining NK points on storage-outflow relationship for control point MM.

*RQ record***

Field	HEC-5 variable name	Value	Description
0		RS	Card identification.
1	NK	≥ 2	Number of values of that will be specified for the storage-outflow relationship for this reservoir.
2	QCAP(1)	+	Total reservoir outlet capacity for control point MM in cfs, corresponding to storage in field 2 of RS record.
3-10 (as needed)	QCAP(2) ... QCAP(NK)	+	Total reservoir outlet capacity for control point MM in cfs, corresponding to storage in fields 3-10 of RS record.

*R2 record***

This record defines the allowable rate of change for releases. Penalties for exceeding these rates are defined on the P$ record.

Field	HEC-5 variable name	Value	Description
0		R2	Card identification.
1	RTCHGR	+	Allowable rate of change of reservoir release, in cfs per hour, when the release from this reservoir increases from the previous period.
2	RTCHGF	+	Allowable rate of change of reservoir release, in cfs per hour, when the release from this reservoir decreases from the previous period.
3-10			Not used by FCLP.

*P$ record (new for FCLP)***

Values on this record define the penalty for exceeding allowable rates of release change that are shown on the R2 record.

Field	HEC-5 variable name	Value	Description
0		P$	Card identification.
1	PENRA (new for FCLP)	+	Penalty per cfs for exceeding RTCHGR (field 1 of the R2 record).
2	PENFA (new for FCLP)	+	Penalty per cfs for exceeding RTCHGF (field 2 of the R2 record).

B.4 CONTROL POINT RECORDS FOR HYDROLOGIC DATA

CP, ID, and RT records are required for all control points, including reservoirs.

*CP record***

Field	HEC-5 variable name	Value	Description
0		CP	Card identification.
1	MM	+	User integer identification number.
2-10			Not used by FCLP

*ID record***

Field	HEC-5 variable name	Value	Description
0		ID	Card identification.
1-4	CPT	any	Title (alphanumeric) of control point in record columns 3-32. This will be printed in summary output.
5-10			Not used by FCLP

*LQ record (new for FCLP) ***

Values on the LQ and L$ record define the flow penalty function for an information center.

Field	HEC-5 variable name	Value	Description
0		LQ	Card identification.
1	Q(1)	+	Cumulative flow rate for flow level 1 for control point MM, in cfs. This defines flow zone 1 as the flow between flow = zero and flow = Q(1) cfs.
2	Q(2)	+	Cumulative flow rate for flow level 2 for control point MM, in cfs. This defines flow zone 2 as the flow between Q(1) and Q(2) cfs.
3-10 (as needed)	Q(3) ... Q(NLF)		Cumulative flow rate for flow levels 3, ... NFL (J1.10) for control point MM, in cfs. Each successive value defines a flow zone between that value and the flow in the previous field. Flow may exceed the value shown as level NLF.

*L$ record (new for FCLP)***

Field	HEC-5 variable name	Value	Description
0		L$	Card identification.
1	PEN(1)	+	Penalty per cfs for flow in first flow zone defined by values on LQ record.
2	PEN(2)	+	Penalty per cfs for flow in second flow zone defined by values on LQ record.
3-10	PEN(3)... PEN(NLF+1)		Penalty per cfs for flow in successive flow zones defined by values on LQ record + an additional penalty per cfs for flow that exceeds Q(NLF), the maximum flow level specified on the LQ record.

*MQ record (new for FCLP) **

Values on the MQ and M$ record define the peak flow penalty function for an information center.

Field	HEC-5 variable name	Value	Description
0		MQ	Card identification.
1	MQ(1)	+	Cumulative flow rate for flow level 1 for control point MM, in cfs. This defines flow zone 1 as the flow between flow = zero and flow = Q(1) cfs.
2	MQ(2)	+	Cumulative flow rate for flow level 2 for control point MM, in cfs. This defines flow zone 2 as the flow between Q(1) and Q(2) cfs.
3-10 (as needed)	MQ(3) ... MQ(NLF)		Cumulative flow rate for flow levels 3, ... NFL (J1.10) for control point MM, in cfs. Each successive value defines a flow zone between that value and the flow in the previous field. Flow may exceed the value shown as level NLF.

*M$ record (new for FCLP)**

Field	HEC-5 variable name	Value	Description
0		M$	Card identification.
1	MPEN(1)	+	Penalty per cfs for peak flow in first flow zone defined by values on MQ record.
2	MPEN(2)	+	Penalty per cfs for peak flow in second flow zone defined by values on MQ record.
3-10	MPEN(3)... MPEN(NLF+1)		Penalty per cfs for peak flow in successive flow zones defined by values on MQ record + an additional penalty per cfs for flow that exceeds Q(NFL), the maximum flow level specified on the MQ record.

*DR record**

Values on this record define parameters for diverting flow from control point MM. The routing of the diversion flow is restricted to either (1) user specified linear routing coefficients or (2) the Muskingum method. If the first option is selected, a CR record must be provided.

Field	HEC-5 variable name	Value	Description
0		DR	Card identification.
1	DRTFR(NDIV)	+	Control point number where diversion is made from. Equal to MM on the CP record.
2	DRTTO (NDIV)	0,+	Control point number where diversion returns to system. Can be zero if there is no return flow.
3	DRTMD (NDIV)	+	Routing method for diversion (see RTMD of RT Record, Field 3). Only linear methods are allowed.
4	DRTCOF (NDIV)	+	Routing coefficient “X” for diversion (see RT Record, Field 4).
5	DRMUSK (NDIV)	+	Routing coefficient “K” for diversion (see RT Record, Field 5).
6			Not used by FCLP
7	KDTY(NDIV)	-1	Diversion quantity is a function of the inflows at control point MM according to the tables of CHQ (QS Records) and FDQ (QD Records).
8-10			Not used by FCLP

*QS record**

QS and QD records are used with the DR Record to specify the flow-diversion relationship at a control point.

Field	HEC-5 variable name	Value	Description
0		QS	Card identification.
1	NPTSQ	2-18	Number of river discharges on QS Record.
2-19	CHQ(M,N)	+	Channel flows used with the QD Record to define the flow-diversion relationship.

*QD Record**

Field	HEC-5 variable name	Value	Description
0		QD	Card identification.
1	NUMDQ	2-18	Number of diversion values on QD Record.
2-19	FDQ(M,N)	+	Diversion flows corresponding to values of channel flow on the QS Record

*RT record***

Values on this record define parameters of the routing model for the reach downstream of control point MM. The routing of FCLP is restricted to either (1) user specified linear routing coefficients or (2) the Muskingum method. If the first option is selected, a CR record must be provided.

Field	HEC-5 variable name	Value	Description
0		RT	Card identification.
1	RTFR	+	Control point number of upstream end of routing reach. Equal to MM on the CP record.
2	RTTO	+	Control point number of downstream end of routing reach MM. Equal to MM of the CP record for the next downstream control point. May be left blank for the most downstream control point in the system.
3	RTMD	+X.Y	Number of sub-reaches (X) and code for method of routing (Y). For FCLP, X must equal 1, and Y is restricted to the following: Y = 2 for Muskingum routing Y = 9 for user-specified coefficients; in this case, the RT record must be followed by CR record.
4	X	$0 \leq X \leq 0.5$	Muskingum routing model parameter X. Must be specified if RT.3 = 1.2.
5	K	+	Muskingum routing model parameter K. Must be specified if RT.3 = 1.2.
6-10			Not used by FCLP

*CR record**

Linear routing coefficients are specified on this record, if required. Note that (1) each coefficient must be between 0.0 and 1.0; (2) one to five coefficients can be specified; and (3) the sum of the coefficients must be 1.0 to maintain continuity in the routing.

Field	HEC-5 variable name	Value	Description
0		CR	Card identification.
1	NUMCOF	≤ 5	Number of routing coefficients specified on this record.
2-5	TRTCOF(1) ... TRTCOF(NUMCOF)	+	Routing coefficients (as coefficients of inflow).

B.5 TIME SERIES

*ZR record***

Read time series data from HEC-DSS.

Record Columns	HEC-5 variable name	Value	Description
0	ID	ZR	Card identification.
3-5	Data type	Blank	
		Data Type	An equal sign and the time series record ID indicating what data type to read from DSS (i.e. =IN or =QA).
6-8	MM	+	Up to three digit CP number (left justified) as defined on CP record, causes data for only that location to be read from DSS.
10+	Pathname Parts		Free form identification of pathname parts. Each pathname part is separated by a comma or space. Unspecified pathname parts will assume values specified on previous ZR cards.

Examples: ZR=IN1 A=IOWA B=IOWA CITY C=FLOW-INC F=COM
ZR=IN15 B=DES MOINES
ZR=IN20 B=OTTUMWA C=FLOW F=NATURAL

For each reservoir, ZR records specify the pathname for observed releases prior to the time and date of the start of the analysis (as specified on the BF record) and the pathname of forecasted inflows for the analysis. To differentiate what data are to be read from the HEC-DSS record with the given pathname, the ZR record must include either =QA to indicate that the pathname is for observed releases or =IN to indicate that the pathname is for forecasted inflows. If ZR records are not provided for the reservoir, if the record does not include either =QA or =IN, or if HEC-DSS records with the specified pathnames do not exist, FCLP assumes that the observed releases and/or forecasted inflows are zero. Likewise, for each information center, ZR records specify the pathname for observed total flows prior to the time and date of the start of the analysis (as specified on the BF record) and the pathname of forecasted local flows for the analysis. To differentiate what data are to be read from the HEC-DSS record with the given pathname, the ZR record must include either =QA to indicate that the pathname is for observed flow or =IN to indicate that the pathname is for forecasted local flows. If ZR records are not provided for the information center, if the record does not include either =QA or =IN, or if HEC-DSS records with the specified pathnames do not exist, FCLP assumes that the observed flows and/or forecasted local flows are zero.

*ZW record***

Write data to the HEC-DSS file. FCLP is designed to file all results in the HEC-DSS. FCLP will automatically assign the B, C, D, and E-part as appropriate. If the ZW record is included, FCLP will automatically file release, storage, and flows for reservoirs, and flows for information centers.

Record Columns	HEC-5 variable name	Value	Description
0	ID	ZW	Card identification.
3+	Pathname Parts		Free form identification of pathname parts. Each pathname part is separated by a comma or space. Only parts A and F can be specified. Other parts will be constructed using control point ID, type of data being written, and date specified on BF record.

Example: ZW A=FCX F=BASE RUN

Model input

```
T1    Sacramento Basin Model for 6 hr time periods
T2    By: Dustin Jones & David Watkins
T3    Last edited 4/19/99
J1            1       6       2       4       1                              3
J3                                                                           2
C  ---------------------------------------------------------------
C                              Stony Creek
C  ---------------------------------------------------------------
C  =====   Black Butte Dam, Stony Creek   =====
C  (Operating levels and S-O from manual, SPK)
C  Level 1: Match point
C        2: Top of Gross Pool
C        3: Match point
C        4: Top of Std Proj Flood Pool
C        5: Spillway Design Pool
C        6: Top of Dam
C 1997:
C RL     2   35800   35000  143676  170000  190100  354000  389000
C 1995:
RL     2   59000   35000  143676  170000  190100  354000  389000
S$                 -0.05    0.01     0.5       1       2       3
RS     6   35000  143676  170000  190100  354000  389000
C  These discharges form a concave function
RQ     6   16000   23000   24600   25800   35500   37500
C  These discharges form a convex function
C RQ     6   16000   23000   24000   30500   83000   94000
C  (Release change taken from Cottonwood Study, Russ SPK)
R2  1000     500
P$     1       1
CP     2                                                                     5
ID Black Butte
RT     2       3     1.9
CR     1       1
C
C  =====   Black Butte release check   =====
C  (False point to monitor Black Butte's release)
CP     3
ID BB rel
LQ   500   15000   16000
L$  -100    0.00    0.50     1.0
RT     3      10     2.2     0.2       5
C
C  ---------------------------------------------------------------
C                  Sacramento River (above Fremont Weir)
C  ---------------------------------------------------------------
C  =====   Shasta Dam, Sacramento River   =====
C  (Operating levels and S-O from manual, SPK)
C  Level 1: Match point
C        2: Top of Conservation Pool
C        3: Match point
C        4: Gross Pool
C        5: Match point
C        6: Top of Dam
C 1997:
C RL     4 3333000 3200000 3250900 3900000 4552000 4750000 4850000
C 1995:
RL     4 3480000 3200000 3250900 3900000 4552000 4750000 4850000
S$                  -0.1   -0.05   0.015    0.08     2.0       3
```

```
RS     6 3200000 3250900 3900000 4552000 4750000 4850000
RQ     6   74000   75100   86660  292600  353000  383000
R2  7500    2000
P$     1       1
CP     4                                                                           2
ID Shasta Dam
RT     4       6     2.2     0.1       6
C
C  =====   Bend Bridge, Sacramento River   =====
C  (Cottonwood Study, Russ SPK)
CP     6
ID Bend Bridge
LQ  6090   80000  200000
L$ -1000    0.00     2.0     3.0
MQ  6090   80000  200000
M$ -1000    0.00    5.81   13.25
RT     6       8     2.2     0.2       5
C
C  =====   Vina-Woodson Bridge, Sacramento River   =====
C  (Cottonwood Study, Russ SPK)
CP     8
ID Vina Woodson
LQ 90000  100000  200000
L$  0.00     0.1     0.2     0.3
MQ 90000  100000  200000
M$  0.00    0.01    0.83    0.84
RT     8      10     1.2    0.15       8
C
C  =====   Ord Ferry, Sacramento River   =====
C  (Cottonwood Study, Russ SPK)
CP    10                                                                           7
ID Ord Ferry
LQ130000  211900  216300
L$  0.00     0.1     0.2     0.3
MQ130000  211900  216300
M$  0.00    0.01    1.94    1.95
RT    10      12     1.2     0.2       8
DR    10      24     2.2     0.1      20              -1
C  The first set of cards starts diverting at the origin.  The second set
C   forms a convex function.
C QS     1  200000
C QD     1   25000
QS     2  110000  500000
QD     2       0  325000
C QD     2       0       0
C
C  =====   Butte City, Sacramento River   =====
C  (Cottonwood Study, Russ SPK)
CP    12
ID Butte City
LQ160000  216500  221000
L$  0.00    0.01    0.02    0.03
MQ160000  216500  221000
M$  0.00    0.01    3.21    3.22
RT    12      14     1.2     0.2       8
C
C  =====   Moulton Weir, Sacramento River   =====
C  (Cottonwood Study, Russ SPK)
CP    14                                                                           8
ID Moulton Weir
LQ160000  279900  285600
L$   0.0    0.01    0.02    0.03
MQ160000  279900  285600
```

```
M$   0.0    0.01    4.78    4.79
RT    14      16     1.9
CR     1       1
DR    14      24     4.2     0.1       5              -1
C  The first set of cards starts diverting at the origin.  The second set
C   forms a convex function.
C QS     1  175000
C QD     1   20000
QS     2   60000  200000
QD     2       0   55200
C QD     2       0       0
C
C  =====   Colusa Weir, Sacramento River   =====
C  (Cottonwood Study, Russ SPK)
CP    16                                                                       9
ID Colusa Weir
LQ 60000   63100   64500
L$   0.0    0.01    0.02    0.03
MQ 60000   63100   64500
M$  0.00    0.02  107.85   107.9
RT    16      20     1.2    0.25       8
DR    16      24     2.2     0.1       8              -1
C  The first set of cards starts diverting at the origin.  The second set
C   forms a convex function.
C QS     1  170000
C QD     1   65000
QS     2   30000  170000
QD     2       0  110500
C QD     2       0       0
C
C  =====   Tisdale Weir, Sacramento River   =====
CP    20                                                                      11
ID Tisdale Weir
LQ 30000   48510   49500
L$  0.00    0.01    0.02    0.03
MQ 30000   48510   49500
M$  0.00    0.01   47.35   47.36
RT    20      40     1.2    0.37       8
DR    20      26     2.2     0.2       6              -1
C  The first set of cards starts diverting at the origin.  The second set
C   forms a convex function.
C QS     1   50000
C QD     1   12000
QS     2   23300   47000
QD     2       0   18390
C QD     2       0       0
C
C  =====   Butte Slough Nr Meridian, Sutter Bypass   =====
CP    24
ID Meridian
LQ130000  634800  647800
L$  0.00    0.01    0.02    0.03
MQ130000  634800  647800
M$  0.00    0.01    9.24    9.25
RT    24      26     2.2     0.2       8
C
C  =====   Rd 1500, Sutter Bypass   =====
CP    26
ID Rd 1500
LQ150000  380000  385000
L$  0.00    0.01    0.02    0.03
MQ150000  380000  385000
M$  0.00    0.01    0.02    0.03
```

```
RT    26      40     1.2     0.2       4
C
C  ----------------------------------------------------------------
C                              Yuba River
C  ----------------------------------------------------------------
C  =====   New Bullards Bar, Yuba River   =====
C  (Operating levels and S-O from manual, SPK)
C  Level 1: Match point
C        2: Top of Conservation
C        3: Match point
C        4: Gross Pool
C        5: Spillway Design Flood Pool
C        6: Top of Dam
C RL    28  794600  640000  790000  900000  960000  998000 1010000
C The following modification eliminates an integer variable
C 1997:
C RL    28  794600  790000  790001  900000  960000  998000 1010000
C 1995:
RL    28  743592  640000  790000  900000  960000  998000 1010000
S$                 -0.02   -0.01     0.1     0.3     2.0       3
RS     6  640000  790000  900000  960000  998000 1010000
RQ     6    3000    7000   85000  127000  153000  161000
C RS     6  790000  790001  900000  960000  998000 1010000
C RQ     6    7000    7001   85000  127000  153000  161000
R2  5000    5000
P$     1       1
CP    28                                                                      4
ID New Bullards
RT    28      30     1.2    0.15       8
C
C  =====   Marysville, Yuba River   =====
C  (New Bullards Bar OM, Russ SPK)
CP    30
ID Marysville
LQ  3510  145000  176400
L$  -100    0.00     2.0     3.0
MQ  3510  145000  176400
M$  -100    0.00    0.02   109.0
RT    30      37     1.9
CR     1       1
C
C  ----------------------------------------------------------------
C                             Feather River
C  ----------------------------------------------------------------
C  =====   Oroville Dam, Feather River   =====
C  (Operating levels and S-O from manual, SPK)
C  Level 1: Match Point
C        2: Top of Conservation
C        3: Match point
C        4: Gross Pool
C        5: Spillway Design Pool
C        6: Top of Dam
C 1997:
C RL    32 2681250 2600000 2788300 3300000 3537600 3814000 3870000
C 1995:
RL    32 2746100 2600000 2788300 3300000 3537600 3814000 3870000
S$                  -0.2    -0.1    0.05     0.5     2.0       3
RS     6 2600000 2788300 3300000 3537600 3814000 3870000
C  These discharges form a concave function
RQ     6   40000   90000  220000  262000  310650  320500
C  These discharges form a convex function
C RQ     6   40000   90000  220000  262000  650000  725000
R2  5000    2500
```

```
P$     1       1
CP    32                                                                       3
ID Oroville Dam
RT    32      34     1.2     0.2       8
C
C  =====   Gridley, Feather River   =====
C  (Oroville Reservoir OM, Russ SPK)
CP    34
ID Gridley
LQ 15150  150000  258900
L$  -100    0.00     0.5     1.0
MQ 15150  150000  258900
M$  -100    0.00     0.1    7.21
RT    34      36     1.2    0.17       8
C
C  =====   Yuba City, Feather River   =====
C  (Oroville Reservoir OM, Russ SPK)
CP    36
ID Yuba City
LQ200000  205800  210000
L$   0.0    0.01    0.02    0.03
MQ200000  205800  210000
M$  0.00    0.01  282.36   282.4
RT    36      37     1.9
CR     1       1
C
C  =====   Junction of Feather and Yuba   =====
CP    37
ID Feather Yuba
LQ300000  310000  320000
L$  0.00    0.01    0.02    0.03
RT    37      38     2.2    0.35       5
C
C  =====   Nicolaus, Sacramento River   =====
C  (Oroville Reservoir OM, Russ SPK)
CP    38
ID Nicolaus
LQ320000  493900  504000
L$  0.00     0.5     1.0     1.5
MQ320000  493900  504000
M$  0.00    0.01    2.99     3.0
RT    38      40     1.2     0.2       4
C
C  =====   Fremont Weir/Verona, Sacramento River   =====
CP    40                                                                      10
ID Fremont-Ver
LQ100000  104500  106700
L$  0.00     0.1     0.2     0.3
MQ100000  104500  106700
M$   0.0    0.01  559.77     560
RT    40      48     1.2     0.2       8
DR    40      50     1.2     0.2       6              -1
C  The first set of cards starts diverting at the origin.  The second set
C   forms a convex function.
C QS     1  460000
C QD     1  275000
C Revised 4/16/99:
QS     2   61000  460000
QD     2       0  355000
C QD     2       0       0
C
C  ------------------------------------------------------------------
C                             American River
```

```
C  -----------------------------------------------------------------
C  =====   Folsom Dam   =====
C  (Operating levels and S-O from manual, SPK)
C  Level 1: Match point
C        2: Top of Conservation (for 1996-1997 event)
C        3: Listed Top of Conservation
C        4: Gross Pool
C        5: Spillway Design Pool
C        6: Top of Dam
C 1997:
C RL    42  486000  440000  486000  610000 1010000 1130000 1300000
C 1995:
RL    42  559600  440000  486000  610000 1010000 1130000 1300000
S$                 -0.15   -0.10    0.02    0.04    1.50    2.00
RO     3      44      46      48
RS     6  440000  486000  610000 1010000 1130000 1300000
RQ     6   36000   39000   43000  444000  564000  733000
R2  7500    5000
P$     1       1
CP    42                                                                  1
ID Folsom Dam
RT    42      44     1.9
CR     1       1
C
C  =====   Fair Oaks, American River   =====
CP    44
ID Fair Oaks
LQ  7720  115000  194500
L$  -100   0.00     0.02    0.04
MQ  7720  115000  194500
M$  -100    0.00   89.32      90
RT    44      46     1.2     0.2       4
C
C  =====   H St, American River   =====
CP    46
ID H Street
LQ 75000  197000  201000
L$  0.00    0.02    0.03    0.04
MQ 75000  197000  201000
M$  0.00    0.02 4658.68    4659
RT    46      48     1.9
CR     1       1
C
C  =====   Sacramento Weir   =====
CP    48  110000                                                          6
ID Sac Weir
LQ 75000  260900  266200
L$  0.00    0.01    0.03    0.04
MQ 75000  260900  266200
M$  0.00    0.01 2703.92    2704
RT    48      56     1.2     0.2       5
DR    48      53     1.9       0       0              -1
CR     1       1
C  The first set of cards starts diverting at the origin.  The second set
C   forms a convex function.
C QS     1  225000
C QD     1       0
C QD     1   95000
QS     5   75000  170000  190000  210000  221600
QD     5       0   96000  111000  121000  123000
C
C  -----------------------------------------------------------------
C                             Yolo Bypass
```

```
C  ------------------------------------------------------------------
C  =====   Colusa Drain, Yolo Bypass   =====
CP    50
ID Colusa Drain
LQ343000  480000  485000
L$  0.00    0.02    0.03    0.04
RT    50      52     1.9
CR     1       1
C
C  =====   Woodland, Yolo Bypass   =====
CP    52
ID Woodland
LQ377000  573900  585600
L$  0.00    0.01    0.02    0.03
MQ377000  573900  585600
M$  0.00    0.01    0.06     0.1
RT    52      53     1.9
CR     1       1
C
C  =====   I-80, Yolo Bypass   =====
CP    53
ID I-80
LQ480000  573900  585600
L$  0.00    0.02    0.03    0.04
RT    53      54     1.2     0.2       6
C
C  =====   Lisbon, Yolo Bypass   =====
CP    54
ID Lisbon
LQ490000  772800  788600
L$  0.00    0.02    0.03    0.04
MQ490000  772800  788600
M$  0.00    0.02    0.92    0.95
RT    54      58     2.2     0.2       8
C
C -------------------------------------------------------------------
C               Sacramento River below Sacramento Weir
C -------------------------------------------------------------------
C  =====   Freeport, Sacramento River   =====
CP    56
ID Freeport
LQ110000  131200  133800
L$  0.00    0.02    0.03    0.04
MQ110000  131200  133800
M$  0.00    0.02   63.78      64
RT    56      58     1.2     0.2       8
C
C  =====   Rio Vista, Sacramento River   =====
CP    58
ID Rio Vista
LQ560000  568400  580000
L$  0.00    0.02    0.03    0.04
MQ560000  568400  580000
M$  0.00    0.02    0.44     0.5
RT    58       0
C
C  Solver option: 0 - XMP; 1 - OSL (MIP); 2 - Write MPS; 3 - OSL (RBE)
SO     1
ED
C  Choose one time period
C HEC-5 starts at the beginning of the hour and FCMIP starts at the end
C of the hour.
BF     2      60              95030806              6                     12
```

```
CBF     2      64                96122600             6
CBF     2      60                96122606             6                    12
C
C  ******   INFLOW RECORDS   ******
ZR=IN2   A=STONY CR     B=BLACK BUTTE       C=FLOW-RES IN   E=6HOUR  F=HEC
ZR=IN4   A=SACRAMENTO   B=SHASTA            C=FLOW-RES IN   E=6HOUR  F=HEC
ZR=IN6   A=SACRAMENTO   B=BEND BRIDGE       C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN8   A=SACRAMENTO   B=VINA-WOODSON BR   C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN10  A=SACRAMENTO   B=AT ORD FERRY      C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN24  A=BUTTE SLOUGH B=NR MERIDIAN       C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN28  A=NORTH YUBA   B=NEW BULLARDS BAR  C=FLOW-RES IN   E=6HOUR  F=HEC
ZR=IN30  A=YUBA         B=NR MARYSVILLE     C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN32  A=FEATHER      B=OROVILLE          C=FLOW-RES IN   E=6HOUR  F=HEC
ZR=IN34  A=FEATHER      B=NR GRIDLEY        C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN36  A=FEATHER      B=AT YUBA CITY      C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN38  A=FEATHER      B=AT NICOLAUS       C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN40  A=SACRAMENTO   B=FREMONT_VERONA    C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN42  A=AMERICAN     B=FOLSOM            C=FLOW-RES IN   E=6HOUR  F=HEC
ZR=IN50  A=YOLO BYPASS  B=COLUSA DRAIN      C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN52  A=YOLO BYPASS  B=NR WOODLAND       C=FLOW-INC      E=6HOUR  F=HEC
ZR=IN54  A=YOLO BYPASS  B=AT LISBON         C=FLOW-INC      E=6HOUR  F=HEC
C
C ** Historical releases and flows
ZR=QA2   A=STONY CR     B=BLACK BUTTE       C=FLOW-RES OUT  E=6HOUR  F=LOOKBACK
ZR=QA4   A=SACRAMENTO   B=SHASTA            C=FLOW-RES OUT  E=6HOUR  F=LOOKBACK
ZR=QA6   A=SACRAMENTO   B=BEND BRIDGE       C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA8   A=SACRAMENTO   B=VINA-WOODSON BR   C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA10  A=SACRAMENTO   B=AT ORD FERRY      C=FLOW          E=6HOUR  F=LOOKBACK
ZR=DA10  A=SACRAMENTO   B=ORD FERRY OVERFLOW  C=FLOW        E=6HOUR  F=LOOKBACK
ZR=QA12  A=SACRAMENTO   B=AT BUTTE CITY     C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA14  A=SACRAMENTO   B=AT MOULTON WEIR   C=FLOW          E=6HOUR  F=LOOKBACK
ZR=DA14  A=SACRAMENTO   B=MOULTON WEIR SPILL  C=FLOW        E=6HOUR  F=LOOKBACK
ZR=QA16  A=SACRAMENTO   B=AT COLUSA WEIR    C=FLOW          E=6HOUR  F=LOOKBACK
ZR=DA16  A=SACRAMENTO   B=COLUSA WEIR SPILL C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA20  A=SACRAMENTO   B=AT TISDALE WEIR   C=FLOW          E=6HOUR  F=LOOKBACK
ZR=DA20  A=SACRAMENTO   B=TISDALE WEIR SPILL  C=FLOW        E=6HOUR  F=LOOKBACK
ZR=QA24  A=BUTTE SLOUGH B=NR MERIDIAN       C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA28  A=NORTH YUBA   B=NEW BULLARDS BAR  C=FLOW-RES OUT  E=6HOUR  F=LOOKBACK
ZR=QA30  A=YUBA         B=NR MARYSVILLE     C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA32  A=FEATHER      B=OROVILLE          C=FLOW-RES OUT  E=6HOUR  F=LOOKBACK
ZR=QA34  A=FEATHER      B=NR GRIDLEY        C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA36  A=FEATHER      B=AT YUBA CITY      C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA38  A=FEATHER      B=AT NICOLAUS       C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA40  A=SACRAMENTO   B=FREMONT_VERONA    C=FLOW          E=6HOUR  F=LOOKBACK
ZR=DA40  A=SACRAMENTO   B=FREMONT WEIR SPILL  C=FLOW        E=6HOUR  F=LOOKBACK
ZR=QA42  A=AMERICAN     B=FOLSOM            C=FLOW-RES OUT  E=6HOUR  F=LOOKBACK
ZR=QA44  A=AMERICAN     B=AT FAIR OAKS      C=FLOW          E=6HOUR  F=LOOKBACK
ZR=DA48  A=SACRAMENTO   B=SAC WEIR SPILL    C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA52  A=YOLO BYPASS  B=NR WOODLAND       C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA54  A=YOLO BYPASS  B=AT LISBON         C=FLOW          E=6HOUR  F=LOOKBACK
ZR=QA56  A=SACRAMENTO   B=FREEPORT          C=FLOW          E=6HOUR  F=LOOKBACK
C
ZW   A=SAC_BASIN F=MIP
EJ
ER
```

Appendix B

INCREMENTAL INFLOW & MUSKINGUM ROUTING DETERMINATION

The following information was provided by David Ford Consulting Engineers.

Table 12. Sacramento River incremental inflow determination methods.

Location	Method
Shasta Reservoir	Hourly data were provided by the Sacramento District for the ’97 event and most of the ’95, we derived hourly inflow from daily inflows (provided by district) using the DSSMATH TTSR function for 3/8/95 0100 to 3/9/95 2400.
Black Butte Reservoir	Provided by Sacramento District
Oroville Reservoir	Provided by Sacramento District
New Bullards Bar Reservoir	Hourly data were provided by the Sacramento District for the ’97 event, we derived hourly inflow from daily inflows (provided by district) using the DSSMATH TTSR function for the ’95 event.
Folsom Reservoir	Provided by Sacramento District
Bend Bridge	We subtracted the routed Shasta outflow hydrograph from the observed hydrograph at Bend Bridge.
Vina-Woodson Bridge	We subtracted the routed Bend Bridge hydrograph from the observed hydrograph at Vina-Woodson Bridge.
Ord Ferry	The East Bank Overflow (EBO) at Ord Ferry was determined using the Sacramento River flow at Ord Ferry vs. EBO relationship provided by HEC. We determined the total flow at Ord Ferry by summing the routed hydrograph from Vina Woodson, the routed Black Butte Dam release, and an estimated incremental inflow. The EBO was calculated using this flow. Then we subtracted the EBO from the total flow at Ord Ferry and routed the resulting hydrograph down to Butte City. The estimated incremental inflow was adjusted until these two hydrographs matched. Some small negative incrementals were changed to zero.
Butte Slough at Meridian	We subtracted the routed weir spill hydrographs (East Bank Overflow, Moulton Weir spill, and Colusa Weir spill) from the observed hydrograph at Meridian.
Gridley	We subtracted the routed Oroville reservoir release hydrograph from the observed hydrograph at Gridley. Many negative incrementals were found, but the sum of the computed incrementals at Gridley was approximatley zero. Therefore, there will be no incremental inflows at Gridley.

Yuba City	The only observed data available at Yuba City was stage data. Therefore, we developed a rating curve at Yuba City using stage and flow output from the Sacramento District's UNET model. We determined the incremental flow at Yuba City by subracting the routed Gridley hydrograph from the "computed" observed hydrograph for Yuba City. Small negative incrementals were changed to zero.
Marysville	We subtracted the routed New Bullards Bar Reservoir outflow hydrograph from the observed hydrograph at Marysville. Hourly data was provided for the '97 event, we derived the hourly flow hydrograph at Marysville from daily flows using the DSSMATH TTSR function for the '95 event.
Nicolaus	We will use an incremental inflow at Nicolaus equal to the flow on the Bear River at Wheatland routed down to the confluence with the Feather River.
Fremont/Verona	We determined the total flow in the system at Fremont/Verona by adding the observed Fremont Weir spill hydrograph to the observed hydrograph at Verona. We determined the incremental flow by subtracting the routed Wilken's Slough, Tisdale Weir spill, Meridian, and Nicolaus hydrographs from this total flow hydrograph at Fremont/Verona. Significant negative incrementals were change to zero for the '95 event and minor negative incrementals were change to zero for the '97 event.
Colusa Drain	First, we determined the total incremental inflow at Woodland by subtracting the routed Fremont Weir spill hydrograph from the observed hydrograph at Woodland. The Colusa Drain incremental inflow was determined by subtracting the observed Cache Creek hydrograph from the total incremental inflow at Woodland.
Woodland	The Woodland incremental inflow is equal to the routed hydrograph for Cache Creek at Yolo.
Lisbon	Only stage data was provided at Lisbon. We developed a rating at Lisbon using stage and flow output from the Sacramento District's UNET model. We determined the incremental flow at Woodland by subtracting the routed Woodland and Sacramento Weir spill hydrographs from the "computed" observed hydrograph for Lisbon. Small negative incrementals were changed to zero for both events.

Table 13 lists the Muskingum routing parameters and identifies the method in which they were determined. The routing parameters developed are based on a one-hour time-step.

We used one of the following methods to determine K and X:

1. K and X were determined by following the method outlined in EM 1110-1417. In this method, K is estimated as the interval between similar points on the inflow and outflow hydrographs. Then, X is obtained through trial and error.

2. In cases where the incremental inflow was too large to allow a reasonable estimation of K and X using method 1, and where appropriate information was available from the Sacramento District's UNET model, K was estimated as

$$K = \frac{L}{V_w}$$

where L = length of reach and V_w = flood wave velocity.

X was estimated using the equation

$$X = \frac{1}{2}\left(1 - \frac{Q_o}{BS_o c \Delta x}\right)$$

where Q_o = reference flow from the inflow hydrograph, c = flood wave speed, S_o = friction slope or bed slope, B = top width of the flow area, and Dx = length of the routing subreach (EM 1110-1417).

If only velocity information was available from the UNET model, X was estimated as 0.2.

3. In cases where the incremental inflow was too large to allow a reasonable estimation of K and X using method 1 and appropriate data were not available from the UNET model, X was estimated as 0.2 and K was estimated using an assumed flood wave velocity between 3-5 ft/s.

To ensure that the Muskingum routing coefficients were positive for each reach, the number of steps was calculated using the equation:

$$\#STEPS = \frac{K}{\Delta t}$$

Since Dt = 1 hour, # STEPS = K.

Table 13. Muskingum routing parameters for the Sacramento River system.

Reach	From	To	X	K (hrs)	# Steps	Method
1	Shasta Reservoir	Bend Bridge	0.1	12	12	1
2	Bend Bridge	Vina-Woodson	0.2	9	9	1
3	Vina-Woodson	Ord Ferry	0.15	17.5	17	1
4	Ord Ferry	Butte City	0.2	1.5	1	1
5	Butte City	Moulton Weir	0.2	5	5	1
6	Moulton Weir	Colusa Weir	0.2	14	14	1
7	Colusa Weir	Colusa City	0	0	0	1
8	Colusa City	Tisdale Weir	0.25	6	6	2
9	Tisdale Weir	Wilkins Slough	0	0	0	1
10	Wilkins Slough	Fremont Weir / Verona	0.38	13	13	2
11	Black Butte Reservoir	Ord Ferry	0.2	11	11	1
12	Ord Ferry	Butte Slough nr Meridian	0.1	28	28	1
13	Moulton Weir	Butte Slough nr Meridian	0.1	19	19	1
14	Colusa Weir	Butte Slough nr Meridian	0.1	16	16	1
15	Butte Slough nr Meridian	Sutter Bypass RD 1500	0.2	16	16	3
16	Tisdale Weir	Sutter Bypass RD 1500	0.2	11	11	3
17	Sutter Bypass RD 1500	Fremont Weir / Verona	0.2	1	1	3
18	Oroville Reservoir	Gridley	0.2	8	8	1
19	Gridley	Yuba City	0.17	18	18	2
20	Yuba City	Nicolaus	0.34	4	4	2
21	New Bullards Bar Reservoir	Marysville	0.15	5	5	1
22	Marysville	Nicolaus	0.37	4	4	2
23	Nicolaus	Fremont Weir / Verona	0.2	4	4	2
24	Fremont Weir / Verona	Colusa Drain	0.2	4	4	2
25	Fremont Weir / Verona	Sacramento Weir	0.2	6	6	2
26	Colusa Drain	Woodland	0.2	2	2	2
27	Woodland	Lisbon	0.2	7	7	2
28	Folsom Reservoir	Fair Oaks	0.4	1	1	1
29	Fair Oaks	H Street	0.2	3	3	2
30	H Street	Sacramento Weir	0.2	2	2	2

Reach	From	To	X	K (hrs)	# Steps	Method
31	Sacramento Weir	Freeport	0.2	5	5	2
32	Sacramento Weir	Lisbon	0.2	6	6	2
33	Freeport	Rio Vista	0.2	8	8	2
34	Lisbon	Rio Vista	0.2	16	16	2