Application of Optimization Modeling to Examine the Benefits of
Expanding the Sacramento River Watershed Bypass System

By

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ABSTRACT

The existing Sacramento River basin bypass system is the backbone of the Sacramento River Flood Control Project, as it conveys peak flood flows through the Sacramento Valley and to the Sacramento-San Joaquin River Delta. The bypass system currently includes the Sutter and Yolo bypasses and their primary control features – the Moulton, Colusa, Tisdale, Fremont, and Sacramento weirs/bypasses. The State of California is beginning to look at expanding portions of the bypass system, to increase its capacity and subsequently decrease peak flow likelihoods in mainstem rivers that run through communities in the Sacramento Valley and Delta regions. This is particularly important with the uncertainty of future flood frequencies, in part due to climate change. This study creates a pre-reconnaissance model of the Sacramento Valley flood management system to provide rapid preliminary modeling, conceptual understanding, and proof of concept regarding how critical components of this system interact during major storms to protect different parts of the Sacramento Valley, and how expansions of various elements of the system may reduce flood damage at various locations. The expansions included in the model increase the overall capacity and flexibility of the bypass system to deal with higher flood flows in a range that have a significant probability of future occurrence. In addition, the expansions reduce the cumulative flood damages expected during large floods. The software used in this study is HEC-ResFloodOpt (Hydrologic Engineering Center's Reservoir Flood Control Optimization Program). The improvements examined include widening of the Sutter Bypass, Fremont Weir, Yolo Bypass, Sacramento Weir/Bypass, the addition of Cherokee Bypass, and several combinations of those expansions. It was found that, of all the expansions to the system, the Fremont Weir is the "bottleneck" of the Sacramento River Flood Control Project and the widening of this feature has potential to greatly reduce expected flood damages from extreme events.

ACKNOWLEDGEMENTS

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CHAPTER 1
INTRODUCTION

1.1 Objectives of Study

This study seeks to quantify potential flood damage reduction benefits of several incremental and cumulative improvements to the Sacramento Bypass System. The study uses an optimization modeling approach that coordinates operations of existing flood control reservoirs in the Sacramento River watershed.

This study complements the 2012 Central Valley Flood Protection Plan (CVFPP), developed by California Department of Water Resources (CA DWR). CA DWR has done extensive research and surveys of many agencies with interest in the Sacramento River basin to identify weak points in the system. These studies were done to frame a State Systemwide Investment Approach (SSIA) to improve the overall flood management system for the Sacramento River basin (CA Department of Water Resources, 2011a).

This study creates a pre-reconnaissance model of the Sacramento Valley flood management system to provide rapid preliminary modeling, conceptual understanding, and proof of concept regarding how critical components of this system interact in major storms to protect different parts of the Sacramento Valley, and how expansions of various elements of the system may change flood damage at various locations. This study was performed using HEC-ResFloodOpt (Hydrologic Engineering Center's Reservoir Flood Control Optimization Program), a mixed integer linear programming optimization software. The objective function of the optimization software is formulated to minimize total damage and operational penalties from flood flows. The reservoirs are operated as an integrated system, with a focus on global, rather than local, damage reduction. The improvements examined include widening of the Sutter Bypass, Fremont Weir, Yolo Bypass, Sacramento Weir/Bypass, the addition of Cherokee Bypass, and several combinations of those expansions.

1.2 Overview of the Sacramento River Watershed

The Sacramento Valley, a large geologic feature in northern California that drains the Sacramento River watershed, is particularly vulnerable to flooding. Following the California Gold Rush of the middle to late 19th century, and prior to the construction of multi-purpose reservoirs, levees, and bypasses, winter and spring storm events resulted in repeated and widespread inundation of much of the Sacramento Valley. It wasn't until the floods of the early 20th century that basin-wide flood management was undertaken in the Sacramento River Watershed (Kelley, 1989). These actions consisted of building relatively large reservoirs with flood control space appropriated, levees along the mainstem of the Sacramento River and its primary tributaries, and bypasses with weirs to divert water from the mainstem of the river into engineered bypass channels (CA Department of Water Resources, 2003). Figure 1 is a map of the Sacramento River watershed and its flood control system.

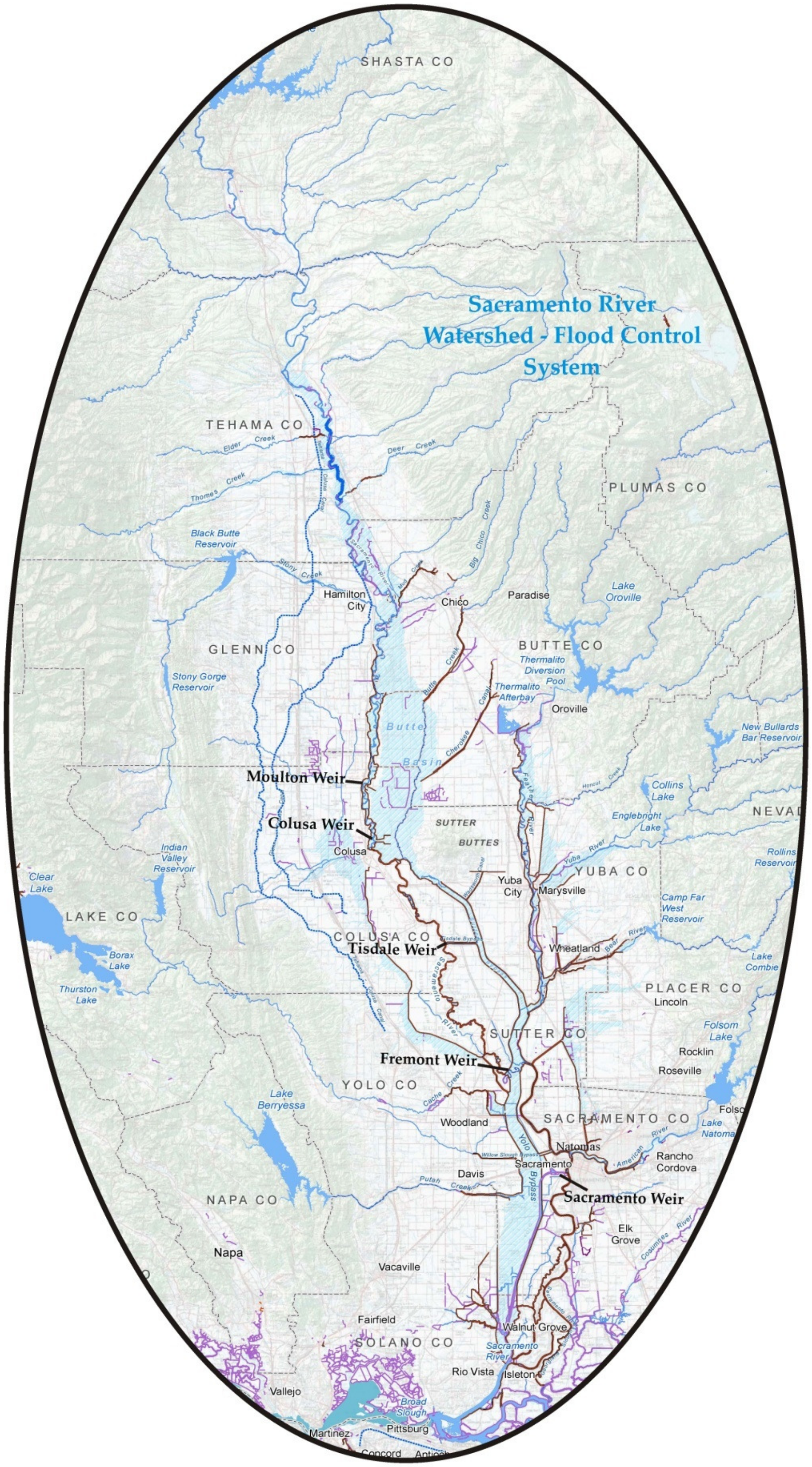


Figure 1. Map of the Sacramento River watershed flood control system. Seasonally-inundated bypass lands are shown in blue hatched shading.

Five primary flood control reservoirs operate within the Sacramento River watershed. They are multi-purpose reservoirs each with established seasonal flood storage allocations. The US Army Corps of Engineers (USACE) has been and is currently responsible for establishing flood storage and rules for operation during the flood season. Throughout high water periods, reservoir operators coordinate with CA DWR and USACE to determine reservoir operations likely to improve overall system operation (FloodSAFE, 2010). The flood and non-flood storage allocations for each reservoir are shown in Table 1.

Table 1. Flood reservation and remaining storage capacity for each flood control reservoir in the Sacramento River watershed

Reservoir	Total Reservoir Capacity (ac-ft)	Flood Reservation Capacity (ac-ft)	Remaining Capacity (ac-ft)
Shasta Lake	4,550,000	1,300,000	3,250,000
Black Butte Lake	160,000	137,000	23,000
Folsom Lake	973,000	400,000	573,000
Lake Oroville	3,540,000	750,000	2,790,000
New Bullards Bar Reservoir	960,000	170,000	790,000
TOTAL	**10,183,000**	**2,757,000**	**7,426,000**

There are four relief bypasses in the Sacramento River watershed; the Sutter, Tisdale, Sacramento, and Yolo bypasses. This study focuses on changes to the Sutter and Yolo bypasses, which are the two main bypasses of the Sacramento River System. The bypass channels are intended to reduce the magnitude and duration of flood flows in the Sacramento River (Russo, 2010).

(1) Sutter Bypass – The northern-most primary bypass in the Sacramento Valley. Flow enters through three weirs (Moulton, Colusa, and Tisdale) and four other relief structures. The design capacity of the bypass is about 185,000 cfs at the upstream end and 216,500 cfs at its confluence with the Feather River.

(2) Yolo Bypass – The largest contiguous floodplain area of the lower Sacramento Valley. This bypass conveys floodwaters from the Sacramento, Feather, and American rivers through the Fremont and Sacramento weirs. The downstream design capacity of the bypass is nearly 500,000 cfs (CA Department of Water Resources, 2009).

The Sacramento River watershed bypass system includes five major lateral weirs. These weirs are lowered and hardened sections of levees that allow flood flows into the bypass channels to decrease the flow in the main river channel below design capacity. All weirs include a fixed-level, concrete sill; a concrete, energy-dissipating stilling basin; an erosion blanket across the channel beyond the stilling basin; and a pair of training levees that define the weir-flow escape channel. All of the weirs, except the Sacramento Weir, pass flood flows by gravity once the river reaches the overflow water surface elevation. The Sacramento Weir is the only weir with control structures, consisting of 48 wooden flashboard sections which can be removed (Russo, 2010). Table 2 lists some pertinent information on each weir.

Table 2. Sacramento River Watershed weir characteristics (Russo, 2010; "Sacramento River / Sacramento River Atlas," n.d.)

Weir Name	Completed Date	River Mile	Lateral Length (ft)	Crest Elevation (ft above msl)	Design Capacity (cfs)
Moulton	1932	158	500	76.75	25,000
Colusa	1933	146	1,650	61.80	70,000
Tisdale	1932	119	1,150	45.45	38,000
Fremont	1924	184	10,560	33.50	343,000
Sacramento	1916	163	1,920	24.75	112,000

1.3 Overview of the Central Valley Flood Protection Plan

The Central Valley of California is susceptible to devastating floods. Residual flood risk to life, property, and economic prosperity in the Central Valley remains one of the highest in the country (CA Department of Water Resources, 2011a). Because of this high flood risk there has been extensive focus on improving flood management in the Central Valley. CA DWR has created and managed several programs such as the Central Valley Flood Management Program (CVFMP). Several documents are being prepared under the CVFMP in response to flood legislation passed in 2007 and the Central Valley Flood Protection Act of 2008. One of these documents was the 2012 Central Valley Protection Plan (CVFPP) (CA Department of Water Resources, 2011b).

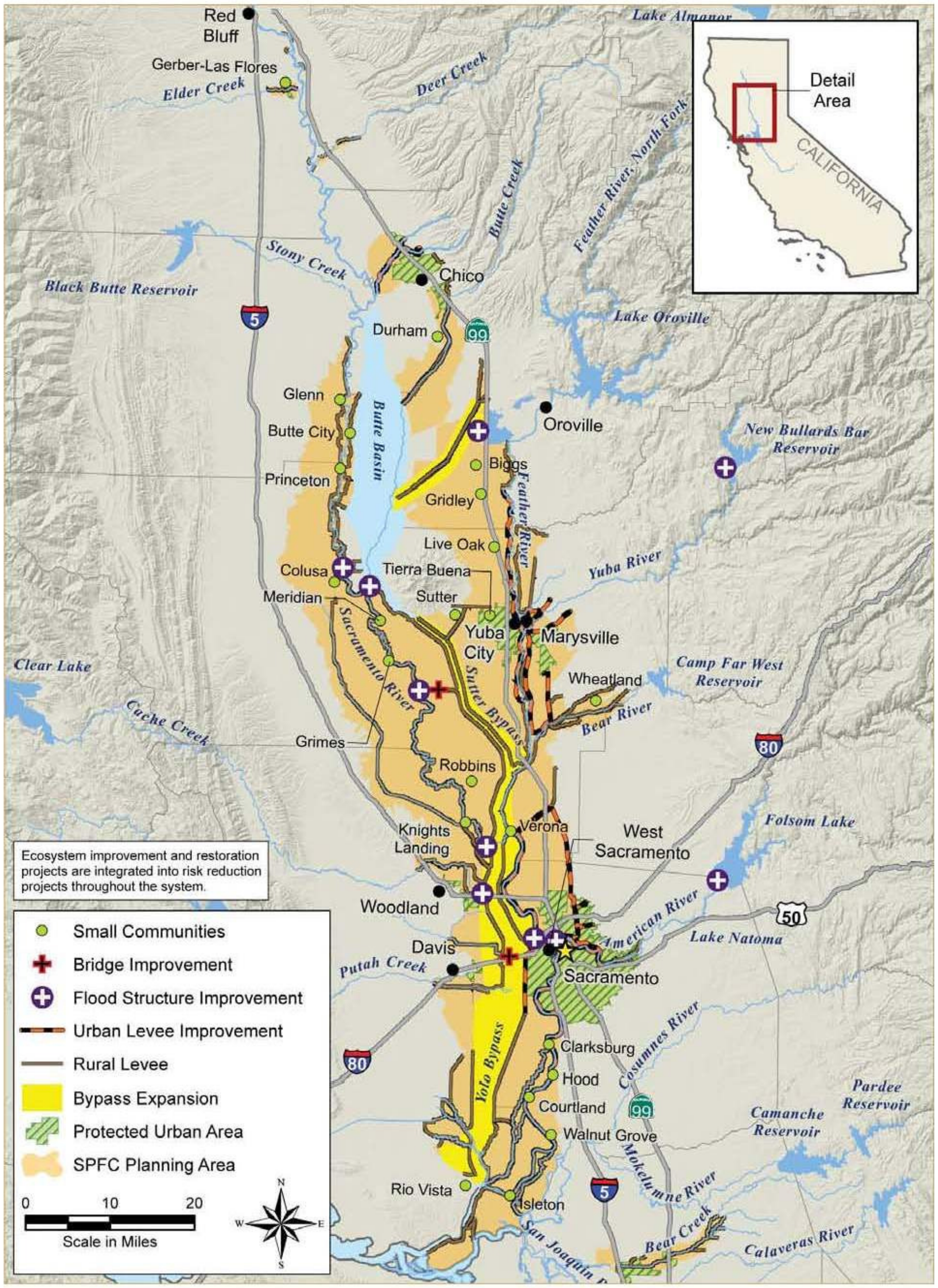
In January 2005, CA DWR published a white paper entitled "Flood Warnings: Responding to California's Flood Crisis," which described the challenges of mitigating flood risk and the deteriorating flood protection system. Some of its major recommendations were:

"…Evaluate the integrity and capability of existing flood control project facilities and prepare an economically viable rehabilitation plan.
…
Where feasible, implement a multi-objective management approach for floodplains that would include, but not be limited to, increased flood protection, ecosystem restoration, and farmland protection. …"

Since that paper, catastrophic flooding from Hurricane Katrina in New Orleans (August 2005) forced a new focus on flood risk management in California. In November 2006, California voters passed two bond measures: Proposition 1E and 84. Proposition 1E allocated $3 billion "To evaluate, repair, and restore existing levees in the state's Central Valley flood control system; to improve or add facilities in order to increase flood protection for urban areas in the state's Central Valley flood control system; and to reduce the risk of levee failure in the Delta region through grants to local agencies and direct spending by the state." Proposition 84 authorized the State of California to sell $5.4 billion in general obligation bonds for water and flood control projects. Because the voters passed the propositions, the recommendations from the 2005 white paper were now being used to guide spending the money that has now been authorized. In the 2007 Legislative Session, a cooperative effort involving the State of California, members of Legislature, local governments and planning agencies, landowners and developers was undertaken to implement recommendations from the 2005 white paper. Towards the end of 2007, the California Legislature passed and the Governor signed five flood bills that addressed flood protection and liability and directed the use of the bond funds approved in 2006. One of these bills enacted the Central Valley Flood Protection Act of 2008 which directed the CA DWR and the Board to prepare and adopt the CVFPP by mid-2012 (CA Department of Water Resources, 2007; “California Proposition 1E, Flood Control and Drinking Water Structures (2006) - Ballotpedia,” n.d., “California Proposition 84, Bonds for Flood Control and Water Supply Improvements (2006) - Ballotpedia,” n.d.; State of California The Resources Agency Department of Water Resources, 2005).

The public draft of the CVFPP was delivered to the Central Valley Flood Protection Board (Board) in December 2011. In February 2012, the Board invited the public to make comments and recommendations on the focus of the CVFPP before the July 1, 2012 acceptance deadline of the Plan as a final document (“Central Valley Flood Protection Plan,” n.d.). Public comments from the CVFPP have questioned the need for expanded bypasses as compared to construction of new flood control storage in reservoirs. Though the CVFPP looked at basic storage needs both for reservoirs and expanded bypasses, these approaches were based largely on observation of system performance under historical events. The CVFPP did not identify specific physical characteristics needed to accomplish this incremental capacity, but multiple ways that the amount of capacity could be achieved (i.e. raised levees or setback levees, widening weirs, etc.). More in depth studies will be done in the upcoming years to identify the most beneficial way to achieve the needed expansion of flood bypass capacity or reservoir flood control storage (Michael Mierzwa, 2012, personal communication).

The CVFPP was written as a descriptive document to address the flood management challenges as part of a sustainable, integrated flood management approach. According to the Central Valley Flood Protection Act of 2008, “The Plan (CVFPP) shall include…an evaluation of the structural improvements and repairs necessary to bring each of the facilities of the State Plan of Flood Control within its design standard.” In this evaluation, the CVFPP focuses on the existing bypass system of the Sacramento River Flood Control Project and discusses the benefits of expanding it as part of their SSIA. See Figure 2.



Key: SPFC = State Plan of Flood Control

Figure 2. Sacramento River basin improvements from the State Systemwide Investment Approach (SSIA) in the CVFPP (CA Department of Water Resources, 2011a)

1.4 Major Historical Flood Events and Hydrology of Interest in Study

In the previous three decades, the Valley has experienced several devastating flood events. The most notable floods occurred in February 1986 and January 1997. These floods were triggered by a "Pineapple Express", a meteorological phenomenon in which warm and plentiful moisture from the southwestern Pacific is channeled into the west coast of North America by a series of large low pressure systems that originate in the Gulf of Alaska. When these types of storms strike the Sierra Nevada during the winter, they can have unusual precipitation intensity, mostly as rain, and have the potential to melt massive amounts of snowpack, resulting in impressive peak streamflows and total storm runoff for the tributaries and mainstem of the Sacramento River (Dettinger et al., 2011).

A post flood assessment, performed by the USACE in 1999, found that near catastrophic damages were narrowly avoided in the 1986 and 1997 storms. The flood control system was pushed to its limits with both of these storms, resulting in numerous moderate failures in the system. Some conclusions from this assessment were that the existing flood management system functioned but was overtaxed, and that another flood like the 1986 or 1997 event would likely result in similar or greater devastation. Additionally, storms larger than 1997 are likely in the future and the resulting flooding could be catastrophic, and the flood control system is in need of upgrade and additional management (US Army Corps of Engineers, Sacramento District, 1999). Because of the extreme nature and magnitude of these storms, they are appropriate events to be analyzed in the optimization model used for this study. In recognition that more extreme floods should also be evaluated, the 1986 and 1997 storm hydrographs were scaled upward in 20 percent increments to generate synthetic storms that were 120 to 200 percent of the historically-measured values; the expected return periods associated with these synthetic events were also estimated as part of this study.

Currently, CA DWR and USACE are involved in the Central Valley Hydrology Study (CVHS). The purpose of this study is to estimate peak flows and hydrographs for various annual exceedence probabilities to characterize potential flood damage and hazards throughout the Central Valley. To produce those peak flows and hydrographs, the first thing done in the CVHS was to collect and process all historical gage data. To develop the unregulated flow time series, the historical gage records and models of the Sacramento and San Joaquin River basins were used to create a consistent flow record.

The last systemwide hydrologic analysis completed for the Central Valley was the Sacramento-San Joaquin Comprehensive Study (Comp Study) in 2002. For the 2012 Central Valley Protection Plan, CA DWR used the hydrology from the Comp Study to accomplish its initial evaluation on how to improve the systemwide flood management. The CVHS builds upon the Comp Study work to produce a more up to date and improved dataset (David Ford Consulting Engineers, Inc. and U.S. Army Corps of Engineers, Sacramento District, 2008).

The CVHS has created a combination of real and synthetic hydrology for local flows back to 1891. It has also created unregulated hydrographs into each of the five flood control reservoirs in the Sacramento River Watershed. This thesis uses hydrology for the 1986 and 1997 events from this study, with the understanding that this hydrology is classified as “preliminary” as of spring 2013.

1.5 Report Organization

Chapter 2 of this report includes a discussion of optimization, why it is used in this study, and how benefits of this study will be measured. Chapter 3 provides an overview of the analysis approach and optimization model formulation. Chapter 4 includes results from the optimization model and a discussion of them. Conclusions and thoughts for improvement and future studies are included in Chapter 5.

CHAPTER 2
METHODS OF OPTIMIZATION MODEL APPLICATION

2.1 Optimization

Optimization involves finding the best (or optimal) solution for a problem. Formal optimization is part of a branch of mathematics called "operations research" concerned with applying scientific methods to decision-making problems and establishing the best or optimal solution. The roots of mathematical optimization methods trace back to notable scientists including Isaac Newton, Augustin-Louis Cauchy, and Joseph Louis Lagrange. Newton contributed differential calculus methods of optimization and Cauchy created the first application of the steepest descent method to solve unconstrained minimization problems. Lagrange invented a method of optimization for constrained problems that produced a metric known as a "shadow price". Shadow prices relate to each constraint in an optimization problem and show the sensitivity of how changes in that constraint will change the optimal solution. Despite these early beginnings, operations research didn't really take hold until early in World War II. The British and U.S. military employed many scientists and mathematicians to help allocate scarce resources to various military and logistical operations and activities in an effective manner. Methods such as linear programming were developed as a result of their research and were instrumental in helping the Allied Forces win the Air Battle of Britain. Since World War II, high-speed digital computers have allowed major advances in optimization methods and applications (Hillier and Lieberman, 2005; Rao, 2009).

The ultimate goal of most optimization problems is to either minimize costs or to maximize benefits. Formal optimization seeks the maximum or minimum of an objective function which depends on a finite number of decision variables. The decisions can be independent of one another or related and limited through one or more constraints. An optimization formulation has mathematical equations which include an objective function and constraints given as mathematical functions of the form:

Max or Min: $Z = f(x_1, x_2, \dots, x_n)$

Subject to:

$$\left.\begin{matrix} g_1(x_1, x_2, \dots, x_n) \\ g_2(x_1, x_2, \dots, x_n) \\ \dots\dots\dots\dots\dots.. \\ g_m(x_1, x_2, \dots, x_n) \end{matrix}\right\} \begin{matrix} \le \\ = \\ \ge \end{matrix} \left\{\begin{matrix} b_1 \\ b_2 \\ \dots.. \\ b_m \end{matrix}\right. \qquad \text{(A)}$$

$$x_1, x_2, \dots, x_n \ge 0$$

where $f(x) =$ an objective function, $x_i =$ the decision variables (n in number), and $g_j(x) =$ the constraints (m in number). A mathematical program is linear if $f(x_1, x_2, \dots, x_n)$ and each

$g_i(x_1, x_2, \ldots, x_n)$ are linear in their arguments; otherwise it is considered a non-linear program. A mathematical program is a mixed integer program if it has the added restriction that some decision variables are integers (Bronson and Naadimuthu, 1997; Rao, 2009; USACE Hydrologic Engineering Center, 2000).

Modeling reservoirs and their downstream watersheds can be mathematically complicated. A simulation model (or descriptive model) simulates reservoir and system operations with a user specified operation policy. Each simulation model scenario analyzes only one alternative. Changes can then be made and the simulation model can be run again under a new scenario. However, this often requires multiple, iterative runs to find the most promising solution. Optimization (or prescriptive) models, on the other hand, suggest optimal solutions and results using an embedded simulation model to evaluate the results based on defined objectives, goals, and constraints for the system, and an efficient search method (Needham and Watkins, 1999; USACE Hydrologic Engineering Center, 1999).

Flood control operations in the Sacramento River System have greatly reduced flood damages during several historical flood events. However, in the absence of a fundamental optimization approach to flood operations, there could be room for improvement. If each reservoir were operated independently, without looking at what other reservoirs in the system are releasing, each reservoir would release to its ability and the combined releases could overwhelm the downstream channel capacity. Currently, there is no such wording in the reservoir water control manuals for such optimized, coordinated operations to be carried out. Fortunately, relations between the agencies that run the Sacramento River Watershed flood control reservoirs are good and “informal coordination” of reservoir releases does occur during flood events. There is still some room for improvement in those operations due to the nature of how far downstream in the system each reservoir operator looks (i.e. Oroville and New Bullards Bar both operate for the Feather-Yuba confluence, Shasta for Bend Bridge, and Black Butte and Folsom both operate to a maximum release rule). Studying historical flood events (in hindsight), in addition to hypothetical events, with optimization modeling can help identify improved reservoir release schedules for given inflows into the system. The optimal release schedule minimizes flood damages throughout the entire system while satisfying operational goals and constraints. Once the optimization model is calibrated to historical flood events and optimal release operations have been determined, hypothetical floods can be studied as well. It is assumed that historical flood damage information can be attained and compared with the resulting optimization model output to estimate the potential incremental benefits from operating flood control facilities in a coordinated manner rather than individually. If the damage computed from the optimization model nearly equals the historical damages, then it supports the notion that the current operating procedure is near-optimal. If the optimization model damages exceed historical damages then the input data and/or the model likely contain inaccuracies and more analysis will be needed. However, if the computed damage is substantially less than the historical damage, then the operational procedures from the optimization model should be strongly considered and given further scrutiny to assess validity and feasibility. If the optimal operation can be made feasible, then the optimization model could help assess the value of adding or upgrading facilities within the Sacramento River System. This project extends earlier work by examining the physical changes listed in Section 2.4 below and assessing how those changes might benefit the system as a whole (Jones, 1999; Needham and Watkins, 1999).

2.2 Introduction to HEC-ResFloodOpt

The software used for this study is the Hydrologic Engineering Center's Reservoir Flood Control Optimization Program (HEC-ResFloodOpt, formerly known as Flood Control Mixed Integer Program [FCMIP]). This software calculates the time series of releases from each reservoir which minimizes cumulative downstream damages. HEC-ResFloodOpt uses the "simplex" method to solve the flood operations problem as a linear program (LP). The simplex method finds the solution to LP problems by finding an initial feasible solution that satisfies all constraints, using that solution to compute a value for the objective function, comparing that value with the best value found so far, and then repeating the process until the best solution is found. For non-linear problems (such as most practical reservoir-operation problems where the cost is not necessarily proportional to the flood damage caused), the function must be modeled approximately using a piecewise linear cost function, in which the non-linear function is approximated by a series of connected linear segments. Figure 3 shows an example where an original decision variable x in the non-linear objective function would be replaced by four linear decision variables $x_1, x_2, x_3, \text{and } x_4$.

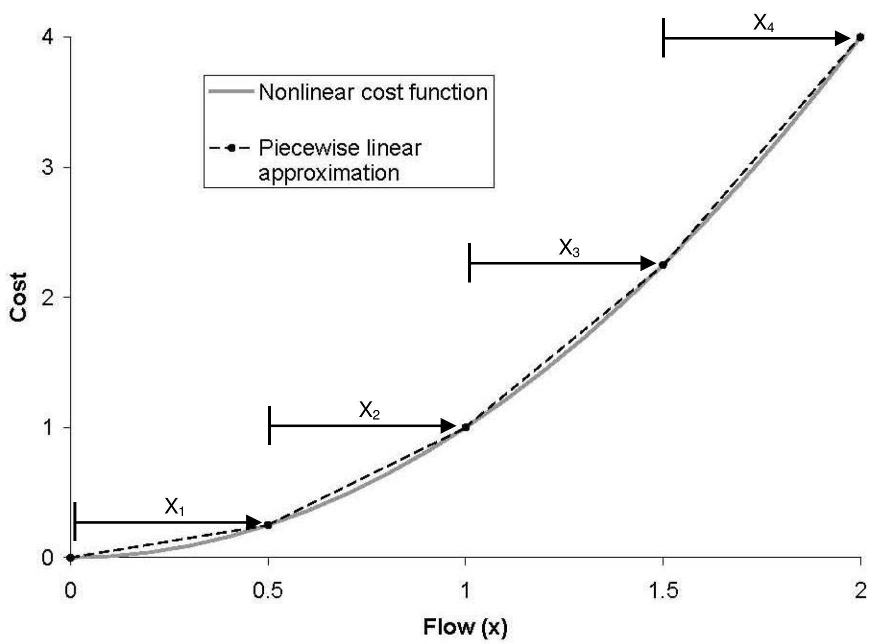


Figure 3. Example of a non-linear function being approximated by a piecewise linear function (USACE Hydrologic Engineering Center, 2000)

The type of piecewise linear cost function approximated in Figure 3 is amenable to linear programming because the cost function to be minimized is convex. This means that the solution will increase from zero in the correct order. However, when looking at a non-convex function such as shown in Figure 4, linear programming alone would not be enough to minimize the objective function because the variables would be filled in the wrong order (i.e. allowing x_4 to fill up before x_3 because of its lower unit cost). Therefore, using conventional LP could underestimate the cost.

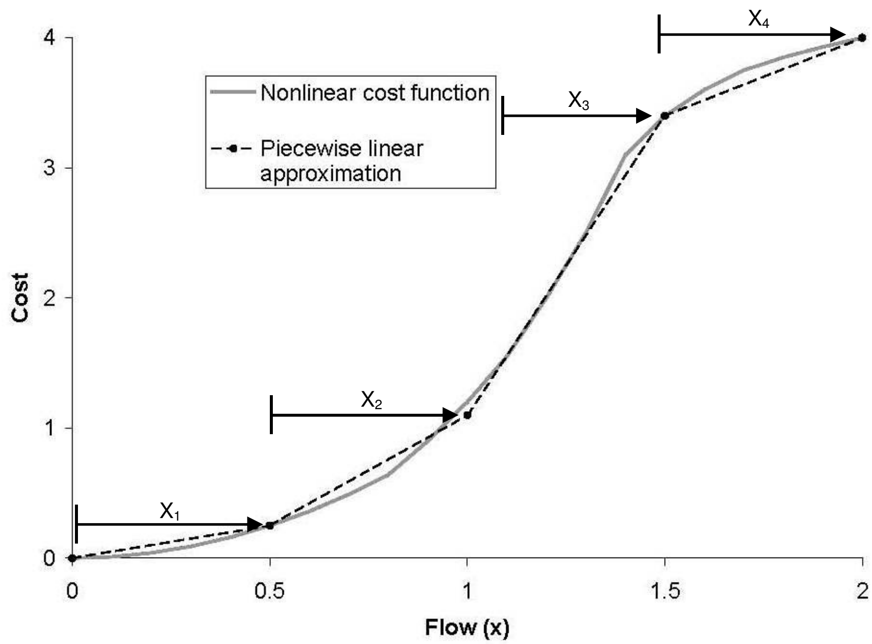


Figure 4. Example of a non-linear function being approximated by a piecewise linear function (USACE Hydrologic Engineering Center, 2000)

To help correct this issue of underestimating the cost function when modeling a non-convex objective function, HEC-ResFloodOpt uses an extension to LP called a mixed integer program (MIP). In this type of program some decision variables are further limited to integer values. It has the same formulation as equation (A) shown in the introduction with the added constraint of:

$$x_i \ integer, i = 1, \dots, p \qquad \text{(B)}$$

The reservoir-operation MIP can be solved using a branch-and-bound algorithm. The branch-and-bound method finds a solution to the MIP by iteratively fixing each integer decision variable at some feasible value and solving the resulting reduced math problem. It then takes the current values calculated from those feasible values and evaluates the objective function, repeating that until the best solution is found (USACE Hydrologic Engineering Center, 2000).

2.3 Development of Objective Function and Constraints

The objective function for this thesis study is to minimize the system penalties downstream of the reservoirs, focusing on the Sacramento Bypass System. System penalties are based on: flow-damage relationships, exceeding reservoir storage levels, and change-in-release constraints (to ensure the program does not increase or decrease the reservoir releases too rapidly) (Needham and Watkins, 1999).

Dustin Jones (1999) formulated a FCMIP model which provides most of the necessary modeling framework for this study. His thesis also focused on the Sacramento River Basin and included all reservoirs and river reaches down through the Yolo Bypass. See Figure 5 for a simplified schematic of the system.

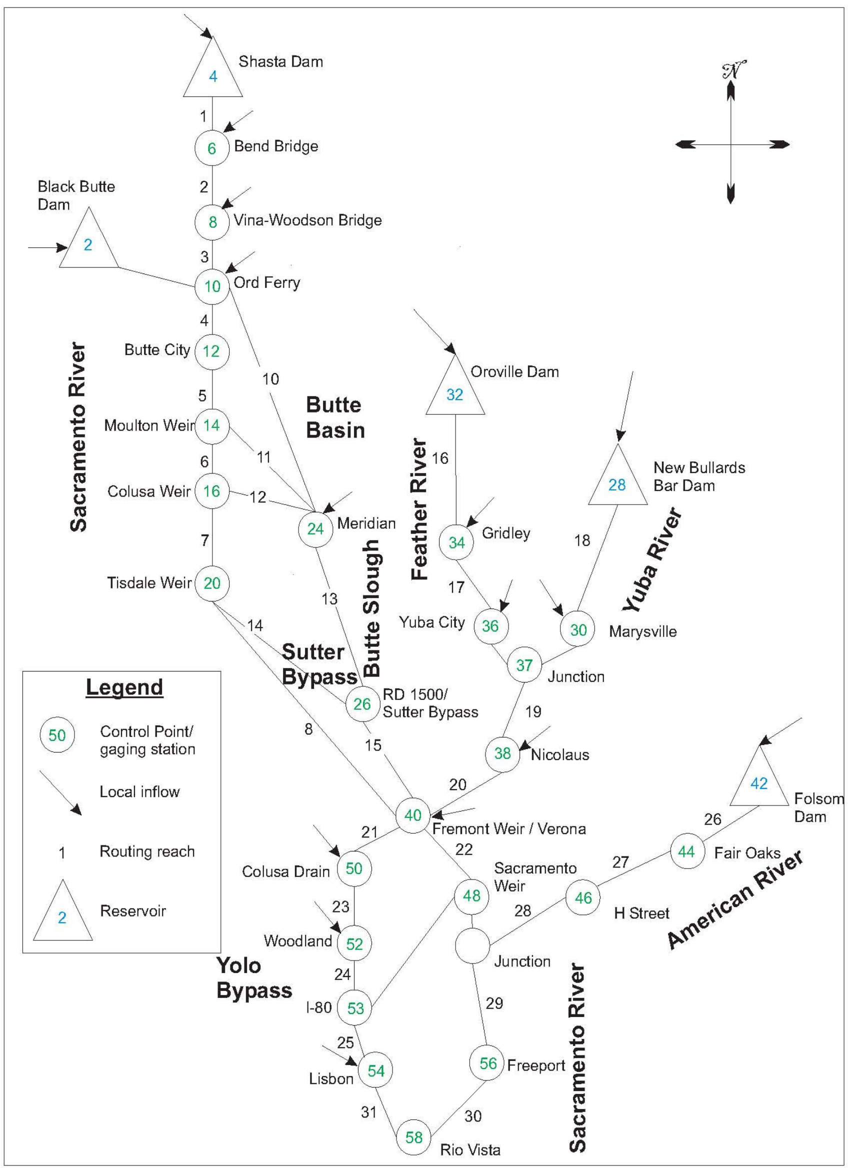


Figure 5. Schematic of the Sacramento River Watershed being modeled

D. Jones' 1999 optimization model was compared against the existing system to make sure that no changes have been made to the Sacramento system in the 12 years since his thesis. The constraints used in his study (FCMIP) are used in this study (HEC-ResFloodOpt) (Watkins et al., 1999). The constraints were originally formulated by Dr. David Ford in his doctoral dissertation (Ford, 1978). The following formulation components, for all channels, reservoirs, and weirs, constitute the overall optimization formulation.

Flow Capacity Constraints

The objective function penalizes higher channel flows, for each time step and each channel.

$$Min\ \sum_{l=1}^{3} c_l f_l \tag{1}$$

Subject to:

$$\sum_{l=1}^{2} f_l \ \geq Y \sum_{l=1}^{2} f_l^{max} \tag{2}$$

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

$$0 \ \leq \ f_l \ \leq \ f_l^{max} \qquad l = 1,2,3 \tag{4}$$

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

where l is a flow zone, f_l is the flow in zone l, f_l^{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.

Reservoir Outlet Constraints

$$R \ \leq \ \sum_{l=1}^{3} B_l S_l \tag{6}$$

$$\sum_{l=1}^{2} S_l \ \geq Y \sum_{l=1}^{2} S_l^{max} \tag{7}$$

$$S_3 \ \leq Y(S_3^{max}) \tag{8}$$

$$0 \ \leq \ S_l \ \leq \ S_l^{max} \qquad l = 1,2,3 \tag{9}$$

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

Here R is the release from the reservoir, S_l is the storage in zone l, and S_l^{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.

Flow Over a Weir

$$D = \sum_{l=1}^{3} \alpha_l f_l \tag{11}$$

$$f_1 \geq Y(f_1^{max}) \tag{12}$$

$$\sum_{l=2}^{3} f_l \leq Y \sum_{l=2}^{3} f_l^{max} \tag{13}$$

$$0 \leq f_l \leq f_l^{max} \qquad l = 1,2,3 \tag{14}$$

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

Here D is the flow over the weir, f_l is flow in zone l of the main channel, and f_l^{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 reduces flood damages overall, these constraints will ensure that the flow zones fill in the proper order.

Reservoir Continuity and Capacity Constraints

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

$$\frac{1}{\Delta t}\left[S_{i,j} - S_{i,j-1}\right] + R_{i,j} - \sum_{k,k\in\omega} \sum_{t=1}^{j} \gamma_{t,k} f_{t,k} = I_{i,j} \tag{16}$$

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 rate at control point k in period t; $\gamma_{t,k}$= linear coefficient to route period t flow from control point k to control point i for period j; $I_{i,j}$ = unregulated inflow rate to the reservoir during period j. Linear routing coefficients may be input directly or HEC-ResFloodOpt can compute them from given Muskingum coefficients.

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:

$$S_{i,j} = \sum_{l=1}^{NLF} S'_{i,j,l} \tag{17}$$

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

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

where the storage in each zone l is constrained as:

$$S'_{i,j,l} \leq SMAX_{i,l} \quad (19)$$

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:

$$f_{i,j} - \sum_{k,k\in\omega}\sum_{t=1}^{j}\gamma_{t,k} f_{t,k} = I_{i,j} \quad (20)$$

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

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:

$$\sum_{l=1}^{NF} f_{i,j,l} - \sum_{k,k\in\omega}\sum_{t=1}^{j}\gamma_{t,k} f_{t,k} = I_{i,j} \quad (21)$$

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

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 piece-wise linear function of the volume of water stored in the reservoir during the period. The total penalty for storage, SP, is defined as:

$$SP_i = \sum_{j=1}^{T}\sum_{l=1}^{NLF} c_{i,l}^{S} S_{i,j,l} \quad (22)$$

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

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 can cause bank damage downstream or 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 $\mathrm{R}_{\mathrm{i,j}}^{+}$ = the total increase in release from period *j-1* to period *j*; and $\mathrm{R}_{\mathrm{i,j}}^{-}$ = the total decrease in release from period *j-1* to period *j*. If $\mathrm{R}_{\mathrm{i,j}}$ ³$\mathrm{R}_{\mathrm{i,j-1}}$, then $\mathrm{R}_{\mathrm{i,j}}^{+}$ is positive and $\mathrm{R}_{\mathrm{i,j}}^{-}$ is zero. If $\mathrm{R}_{\mathrm{i,j}}$ £ $\mathrm{R}_{\mathrm{i,j-1}}$, then $\mathrm{R}_{\mathrm{i,j}}^{-}$ is positive and $\mathrm{R}_{\mathrm{i,j}}^{+}$ is zero. If $\mathrm{R}_{\mathrm{i,j}} = \mathrm{R}_{\mathrm{i,j-1}}$, then both $\mathrm{R}_{\mathrm{i,j}}^{+}$ and $\mathrm{R}_{\mathrm{i,j}}^{-}$ are zero.

To define allowable increases and decreases, $\mathrm{R}_{\mathrm{i,j}}^{+}$ and $\mathrm{R}_{\mathrm{i,j}}^{-}$ 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 $\mathrm{Ra}_{\mathrm{i,j}}^{+}$, $\mathrm{Re}_{\mathrm{i,j}}^{+}$ are the acceptable and excessive release increase, respectively; and $\mathrm{Ra}_{\mathrm{i,j}}^{-}$, $\mathrm{Re}_{\mathrm{i,j}}^{-}$ are the acceptable and excessive release decrease, respectively. Thus, the current release can be defined as:

$$R_{i,j} = R_{i,j-1} + \left[Ra_{i,j}^{+} + Re_{i,j}^{+}\right] - \left[Ra_{i,j}^{-} + Re_{i,j}^{-}\right] \tag{24}$$

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

$$RP_i = \sum_{j=1}^{T} c_i^{r+} Re_{i,j}^{+} + \sum_{j=1}^{T} c_i^{r-} Re_{i,j}^{-} \tag{25}$$

where $\mathrm{c}_{\mathrm{i}}^{\mathrm{r+}}$ is the penalty per unit flow rate for an excessive increase in release rate and $\mathrm{c}_{\mathrm{i}}^{\mathrm{r-}}$ is the penalty per unit flow rate for an excessive decrease in release rate.

Penalty for too Much or too Little Flow at Control Points

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:

$$QP_i = \sum_{j=1}^{T} \sum_{l=1}^{NF} c_{i,l}^{f} f_{i,j,l} \tag{26}$$

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

Peak Flow Penalty

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

$$\overline{QP_l} = \sum_{l=1}^{NF} c_{i,l}^{\bar{f}} \overline{f_{l,l}} \tag{27}$$

$$\sum_{l=1}^{NF} \overline{f_{l,l}} \geq \sum_{l=1}^{NF} f_{i,j,l} \qquad \forall\, i,j \tag{28}$$

where $c_{i,l}^{\bar{f}}$ is the slope of the peak flow penalty function in flow zone l at control point i. This peak flow usually represents most flood damage.

Overall 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:

$$\min TP = \left[\sum_{i,j\in\varphi}(QP_i + \overline{QP_l}) + \sum_{i,j\in\Phi}(RP_i + SP_i)\right] \tag{29}$$

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.

In D. Jones' 1999 discussion he suggests that his optimization model could be used in the future for a study of structural enhancements to the system (Jones, 1999). That is the topic of this thesis, which focuses on enhancements to the Sacramento Bypass System.

2.4 System Modifications of Interest for Study

This thesis focuses on the following potential system changes:

1) 01 Current – Changing D. Jones' 1999 optimization model to include changes in estimated capacity on the mainstem rivers and the bypasses in the system. Some reservoir storage-outflow relationships were also changed to represent the existing water control manuals more closely. This will be discussed in Chapter 3.

2) 02_SBWiden - Expanding the Sutter Bypass capacity to include an additional 4,000 acres as described in Attachment 8J of the CVFPP. To utilize the additional 4,000 acres, it was calculated that the Sutter Bypass would have to be widened by 1,000 feet, and would require 15 miles of new levee along one side of the bypass. Currently the bypass is about 4,000 feet wide, and the capacity of the bypass would be increased by about 25%.

3) 03_FWWiden - Widening the Fremont Weir by a mile. Currently the Fremont Weir is about 2 miles long, and flow capacity over the weir would therefore be increased by about 50%.

4) 04_SWWiden - Widening the Sacramento Weir and bypass. The Sacramento Weir would be increased by approximately 1,000 feet and would require 2 sets of 8 gates according to Attachment 8C in the CVFPP. The bypass will be expanded by 1,300 acres. Currently the Sacramento Weir is 1,920 feet wide, and an expansion of 1,000 feet would therefore increase the flow capacity by about 50%.

5) 05_YBWiden - Expanding the Yolo Bypass to increase its capacity by 40,000 cfs.

6) 06_CBAdd - Establishing a 32,000 cfs capacity Cherokee Bypass from just below Oroville Dam off the Feather River to the Butte Basin.

7) 07_SBFWWiden – Expanding both the Sutter Bypass and the Fremont Weir in combination with the changes as described above.

8) 08_SBFWYBWiden – Expanding the Sutter Bypass, Fremont Weir, and Yolo Bypass in combination with the changes as described above.

9) 09_SBFWYBSWWiden – Expanding the Sutter Bypass, Fremont Weir, Yolo Bypass, and Sacramento Weir and bypass in combination with the changes as described above.

10) 10_FWYBWiden – Expanding the Fremont Weir and Yolo Bypass in combination with the changes as described above.

Appendix A shows the changes made in the optimization model to reflect the capacity changes. The next Chapter includes a discussion of how the "01_Current" HEC-ResFloodOpt run compares to the equivalent FCMIP run.

CHAPTER 3
INITIAL MODEL SENSITIVITIES AND INPUTS

Since D. Jones' thesis was completed in 1999, many advances have been made in computers and some changes have also been made to the software program, HEC-ResFloodOpt, itself. This chapter focuses on the changes between the 1999 version of this program and today's version of this program, the changes in the base optimization model used for this study and D. Jones' version of the Sacramento River Watershed model (including channel capacities and reservoir storage-outflow relationships), and how the different hydrologic data sets available for use in HEC-ResFloodOpt affect the solution.

3.1 Hardware/Software Impacts

As mentioned in Section 2.1, studying historical flood events with optimization can help identify reservoir release schedules that result in reduced cumulative damages. D. Jones' 1999 thesis calibrated the Sacramento River Watershed FCMIP model to the January 1997 event. This was accomplished by adjusting the storage and minimum-flow penalties until the optimization model operation matched the historical operation reasonably well (Jones, 1999). His calibrated optimization model is the starting point for the HEC-ResFloodOpt model used in this thesis.

In 1999, personal computers were slower than today. For D. Jones' 1999 thesis a computer with a 400 MHz Pentium II processor and 128 MB of RAM was used. Each FCMIP run took approximately 30 minutes (Jones, 1999). For this thesis, a Dell Latitude E6500 laptop personal computer with a 2.26GHz Intel Core 2 Duo processor and 2 GB of RAM was used. The HEC-ResFloodOpt runs each took less than one minute. An analysis of the results from HEC-ResFloodOpt on the new computer was done to compare to the results from D. Jones' 1999 original results.

To first compare the differences in how the optimization solver behaved in 1999 to how it behaves with today's version, a HEC-ResFloodOpt run was made using identical inputs to D.Jones' version. The following figures show some comparisons of the results between the 1999 computer and 2013 computer results. The 1999 computer results were kindly made available from D. Jones and were for the March 1995 and January 1997 historical storms. Overall, the programs appear to function similarly. Some changes have been made since D. Jones' version of the program to today's version of the program. The main differences appear in releases from the reservoirs and flow over the weirs. These differences primarily originate from HEC changing some of the optimization constraint formulations to improve the program. Also, the program solution can have multiple potential local optima to choose from so the same results are emerging for the overall solution, but due to the different formulations, the reservoirs make slightly different releases to reach a similar solution. Figure 6 through Figure 14 show some of the main points in the system and their differences.

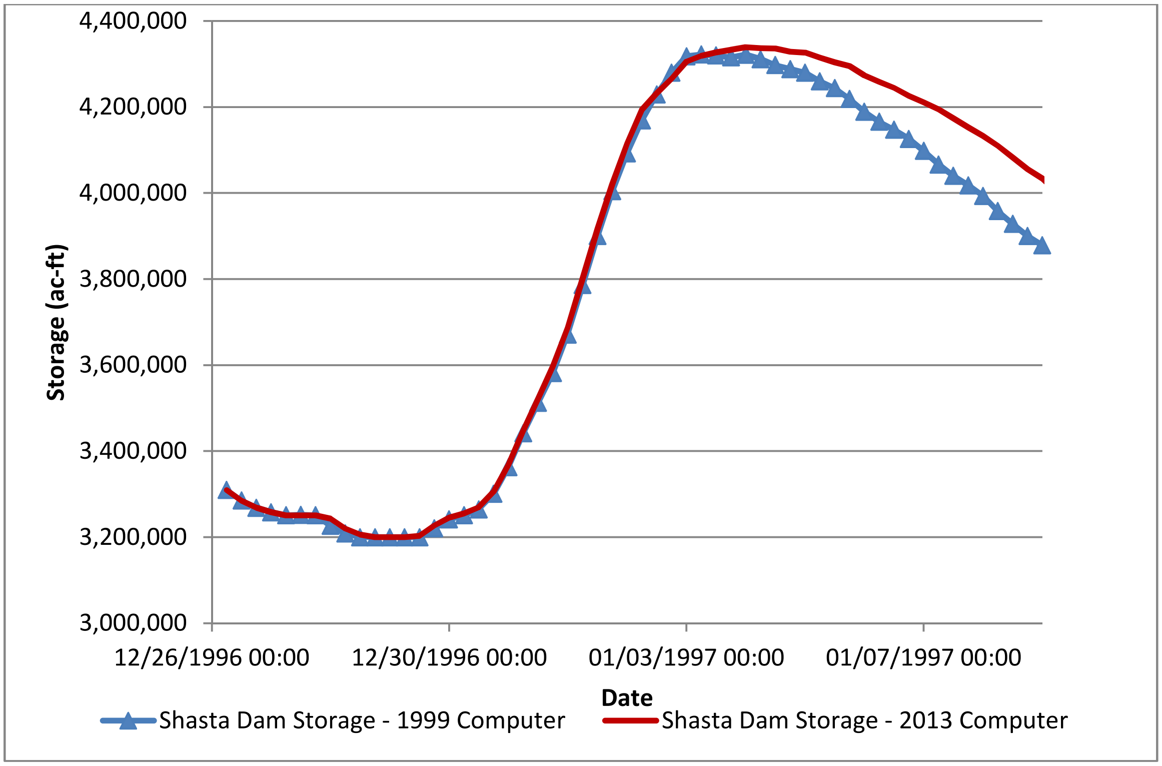


Figure 6. 1999 and 2013 computer results for Shasta Dam storage level (January 1997 event)

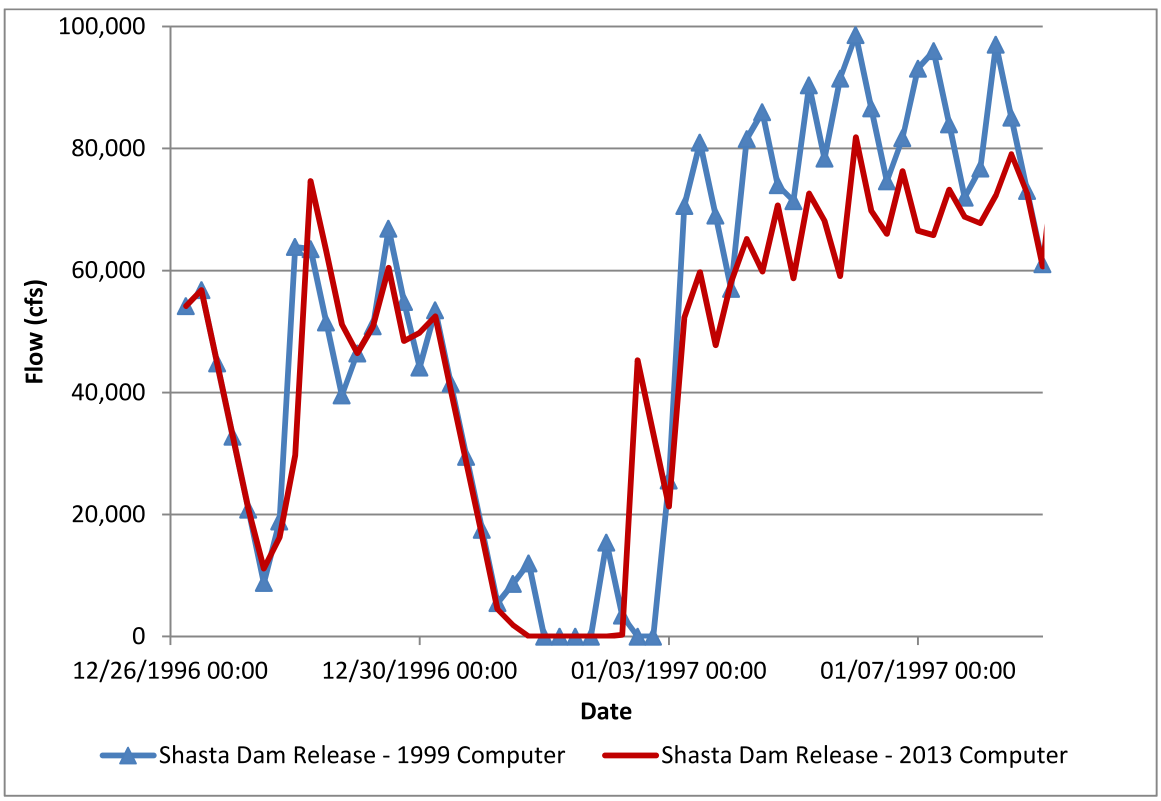


Figure 7. 1999 and 2013 computer results for Shasta Dam release (January 1997 event)

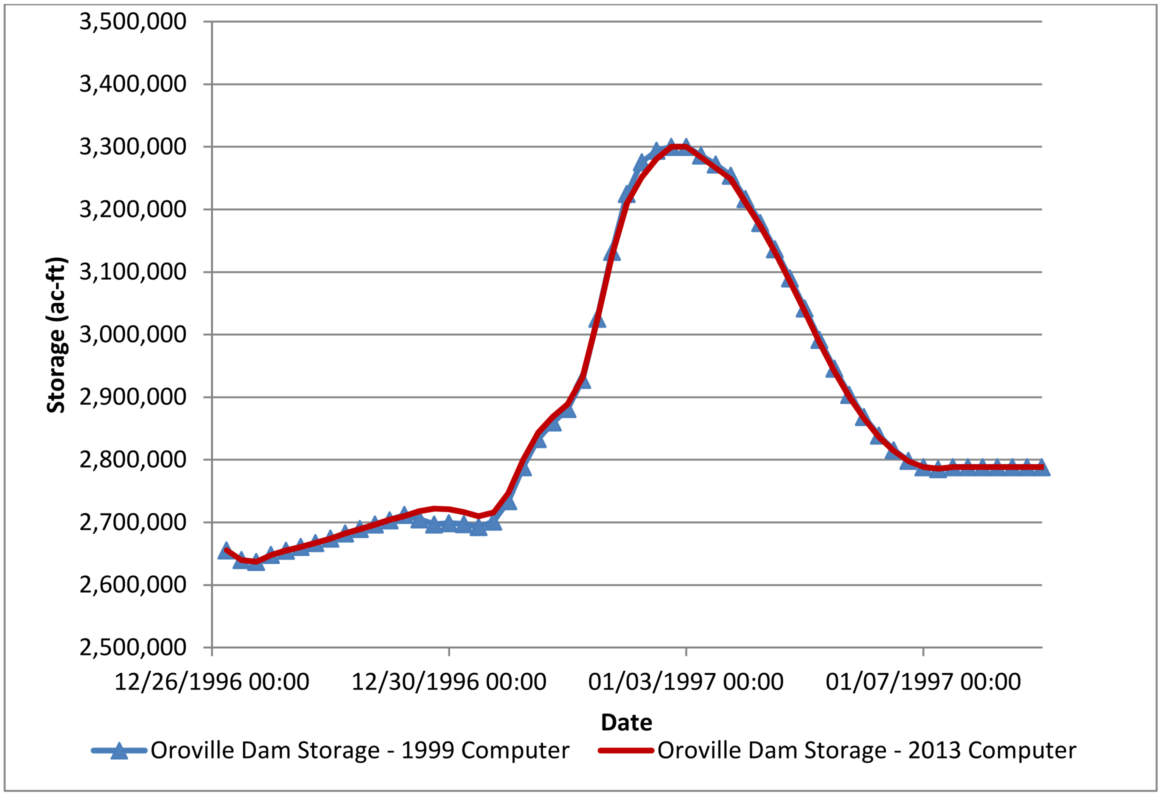


Figure 8. 1999 and 2013 computer results for Oroville Dam storage (January 1997 event)

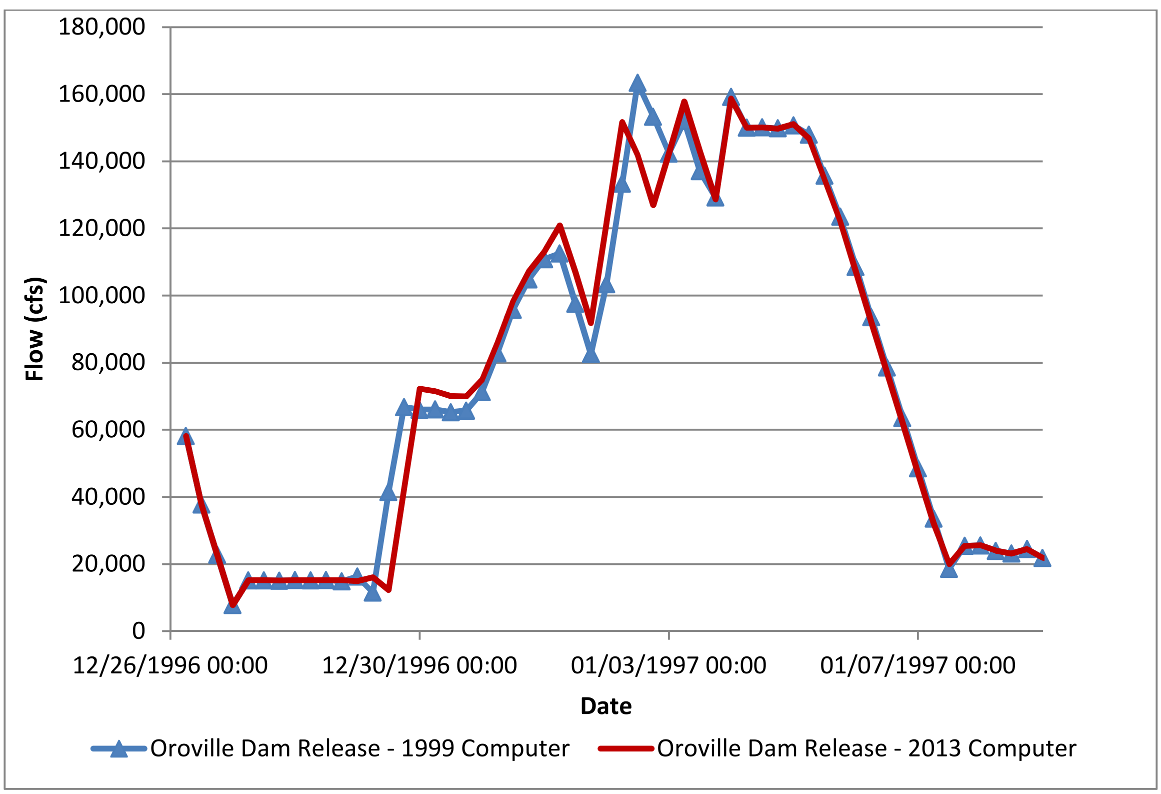


Figure 9. 1999 and 2013 computer results for Oroville Dam release (January 1997 event)

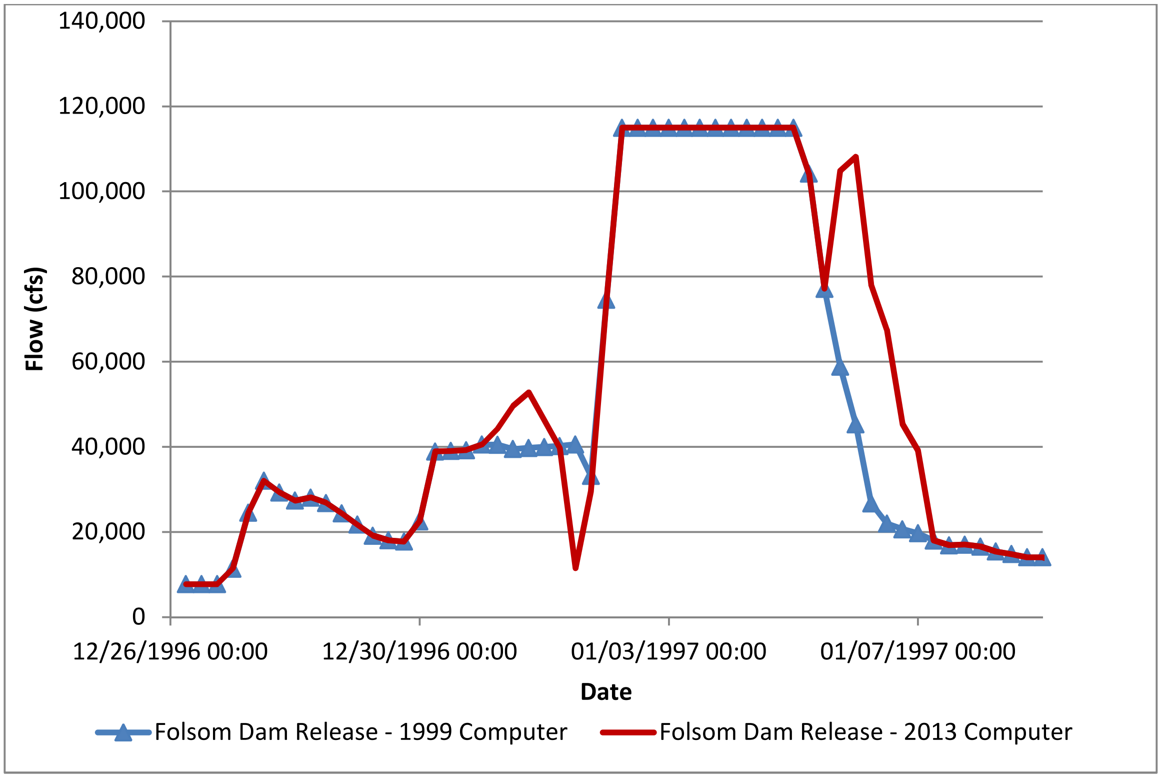


Figure 10. 1999 and 2013 computer results for Folsom Dam release (January 1997 event)

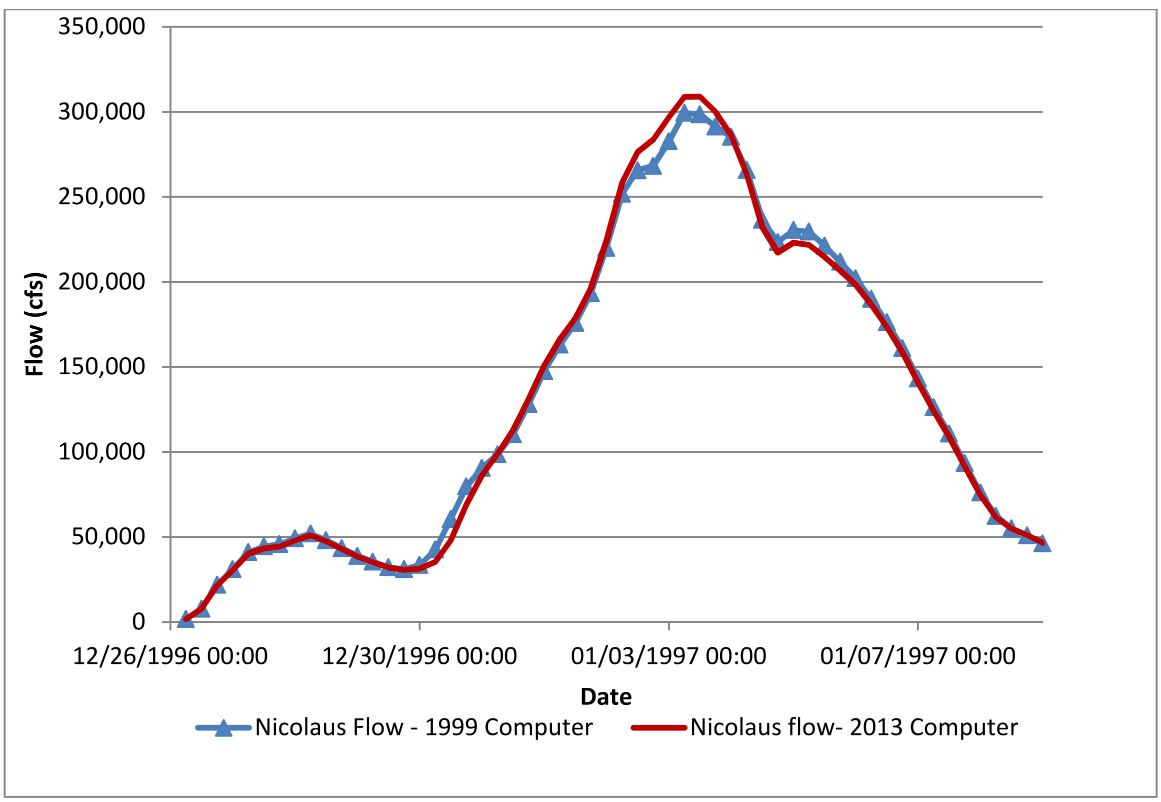


Figure 11. 1999 and 2013 computer results for the flow at Nicolaus (January 1997 event)

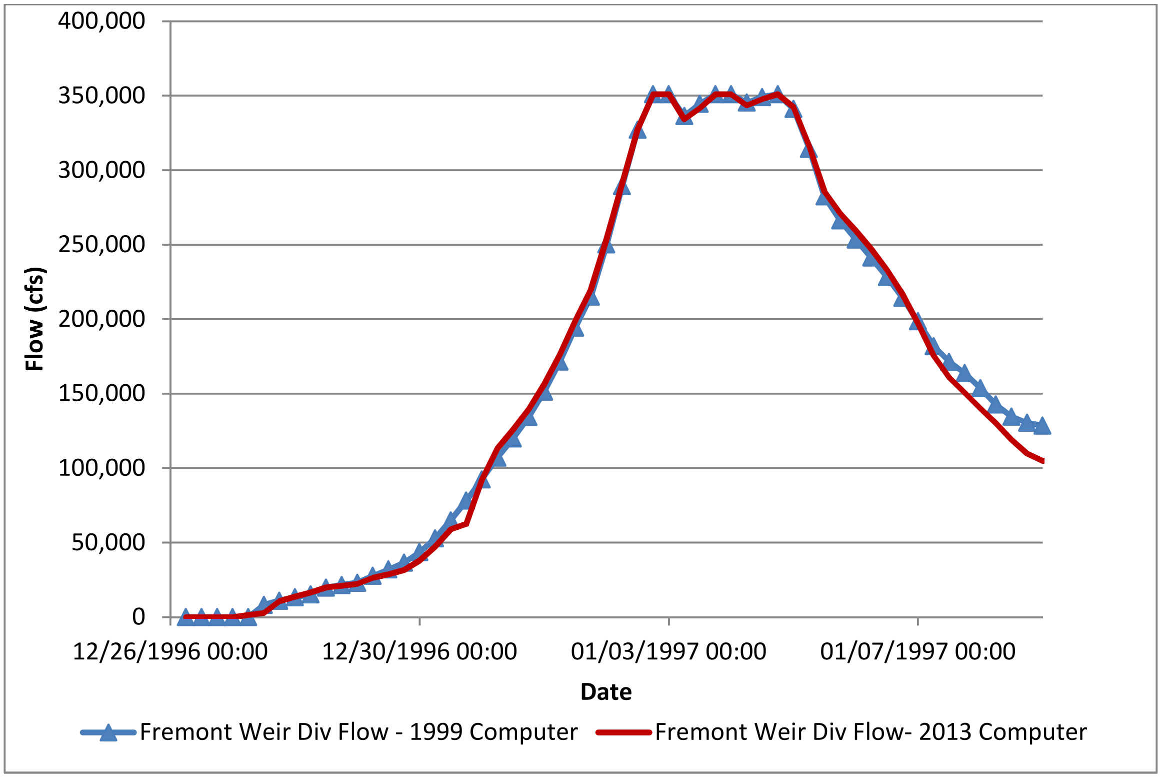


Figure 12. 1999 and 2013 computer results for flow over the Fremont Weir (January 1997 event)

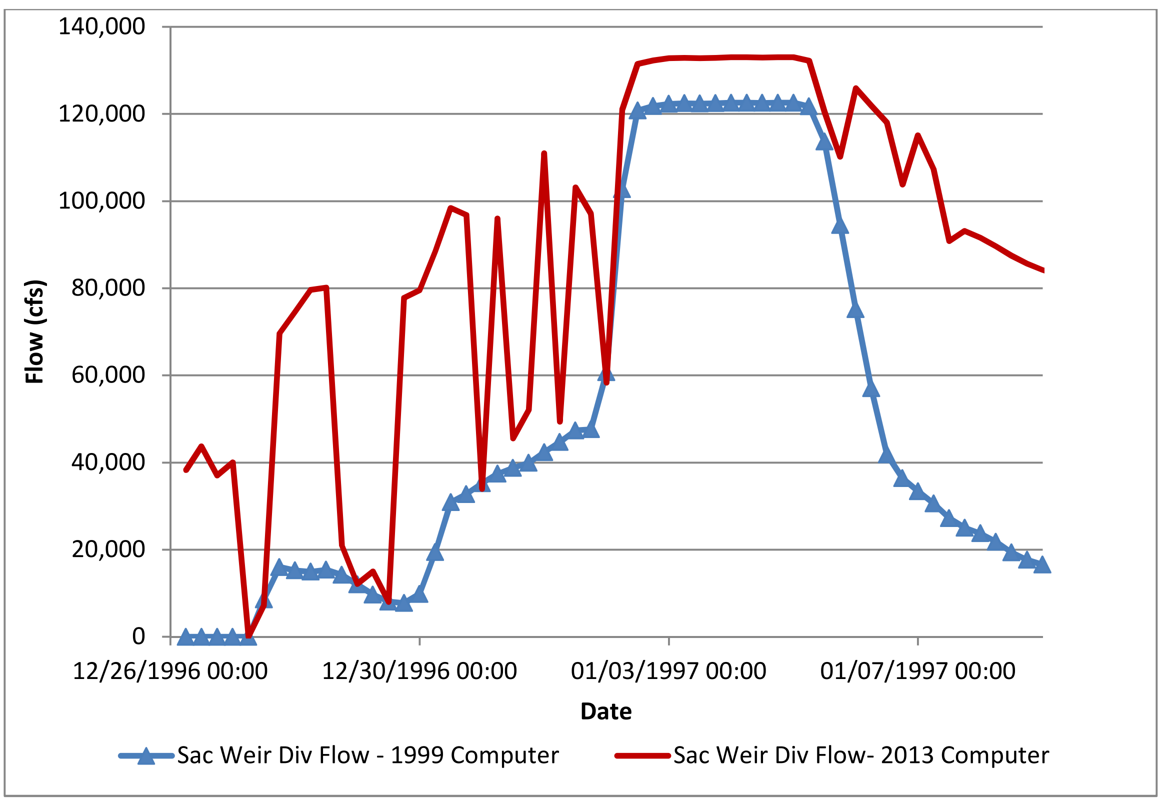


Figure 13. 1999 and 2013 computer results for flow over the Sacramento Weir (January 1997 event)

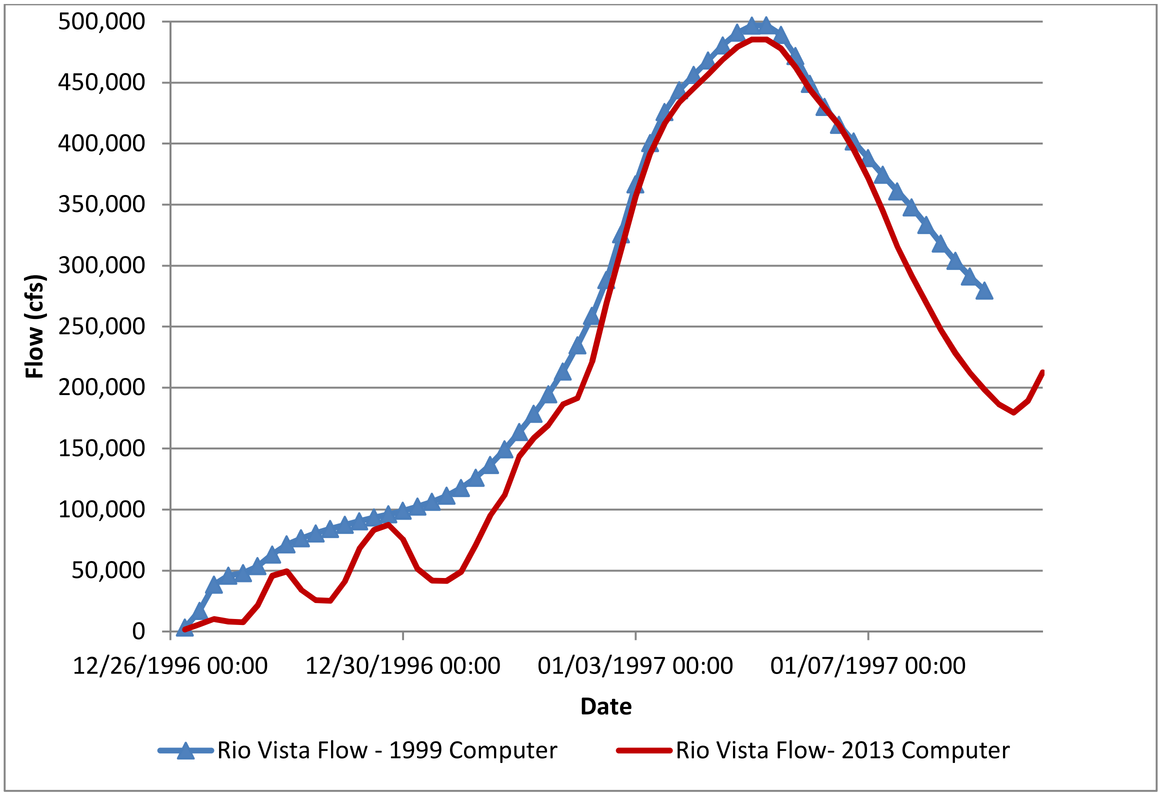


Figure 14. 1999 and 2013 computer results for Rio Vista flow (January 1997 event)

3.2 Weir Flows

As can be seen in Figure 13, there is a fundamental difference in how the FCMIP software calculates bypass weir flow as compared to the HEC-ResFloodOpt software. To go into more depth on the calculation of the weir flow and the continuity of the control point as was shown in in Equation (20) of Chapter 2, a control point must have the continuity constraint of having the total inflow equal the total outflow. To take into account a control point where a diversion is included, the new continuity constraint for each control point is:

$$f_{i,j} + D_{i,j} - \sum_{k,k\in\omega} \sum_{t=1}^{j} \gamma_{t,k} f_{t,k} = I_{i,j} \qquad (30)$$

Where $D_{i,j}$ is the average diversion flow leaving the control point i in period j.

Weir flows are represented in HEC-ResFloodOpt as a function of the flow in the main channel. Most relationships representing weir flows as a function of the main channel flow include convex segments, owing to the increasing rate of flow increase over a weir with increase in depth. As was shown in Equations (11)-(15) in Chapter 2, flow over the weir is constrained by a binary variable which ensures that the flow zones fill in the proper order. Figure 15 shows the Sacramento Weir flow relationship (taken from the HEC-ResSim model built for the Central

Valley Hydrology Study) with multiple points as well as the simplified version with only one convex segment that is allowed in HEC-ResFloodOpt.

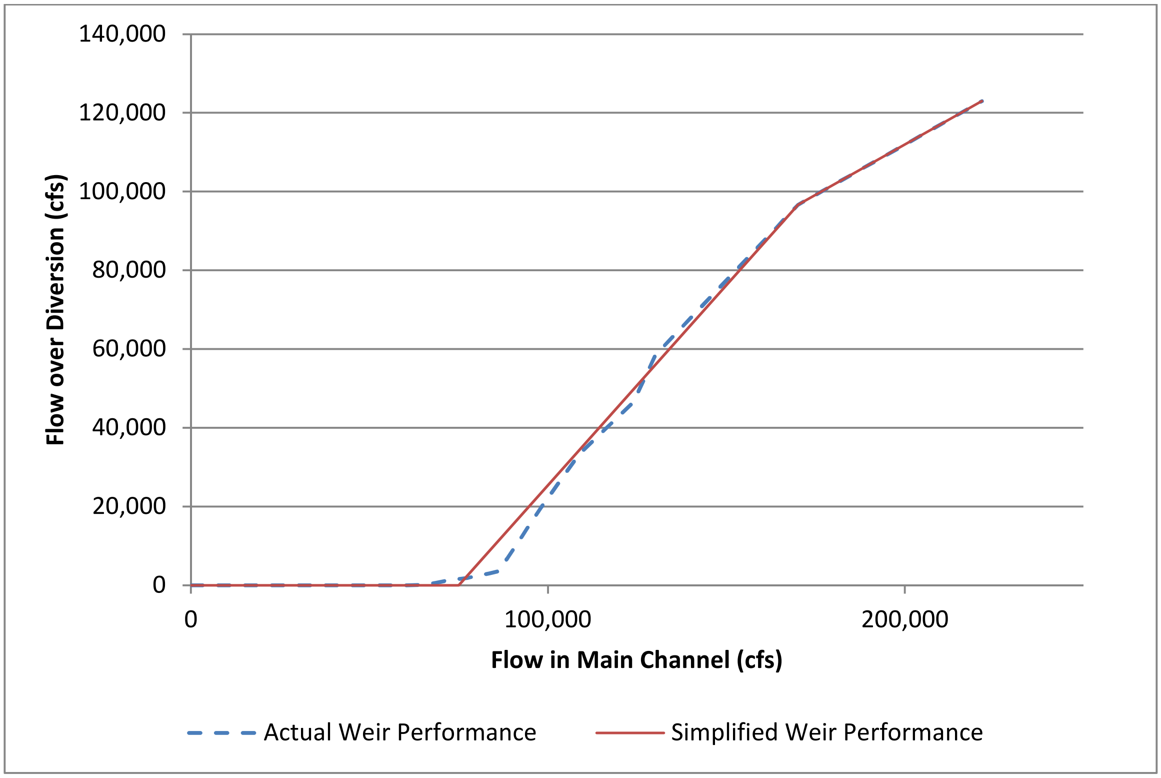


Figure 15. Actual weir performance curve vs. simplified weir performance curve

After further research and multiple troubleshooting attempts, it was determined that there was a repeatable instability issue of the weir calculations in HEC-ResFloodOpt. A senior engineer at HEC was contacted to follow up on what more could be done to smooth the weir curve. It was found that changes were made to the constraint equations within the code to improve how the calculations were solved. Due to those changes, a mathematical error seems to have been introduced into the constraint equations for weirs which is causing the instability in the new software's weir flow equations. No more improvements are being made to this software as of spring 2013, and there does not appear to be any thought to pursue further improvement to this particular optimization software.

Even with these instabilities in the weir function, the modeled weirs generally attempt to function as expected from the given rating curve. The crucial downstream control points in the Sacramento River System are proportionally much less affected and follow expected results. This can be seen in Section 3.5 where the "01_Current" HEC-ResFloodOpt run is compared against D. Jones' 1999 FCMIP run and the observed January 1997 flows. With these comparisons providing relatively consistent results, the HEC-ResFloodOpt software is sufficient to continue for the purposes of this study.

3.3 Reservoir Outlet Rating Curves

In D. Jones' 1999 thesis, he modeled two reservoir storage-outflow relationships, Black Butte Dam and Oroville Dam, as simplified concave functions. This was to simplify computation for those reservoirs. Integer variables are used to model spills from reservoirs in FCMIP and HEC-ResFloodOpt. Since these reservoirs did not spill during historical January 1997 operations, D. Jones felt that they could be simplified in the modeling to keep the amount of computations the computer had to do to a minimum. In his thesis he mentions that the more accurate concave-convex reservoir outlet rating curves could be used if the computer computation time is not a concern and if more severe flood events are to be analyzed. This study's computation times were so short that computation time was not an issue. This thesis is also studying much more severe floods than the January 1997 event. Therefore, the more complex reservoir outlet rating curves were input to HEC-ResFloodOpt. An example of the different reservoir outlet rating curves between D. Jones' 1999 thesis and this thesis is shown in Figure 16. The concave-convex curves effectively remove unrealistic conservatism from reservoir operating flexibility, as they allow for much higher releases when approaching full pool.

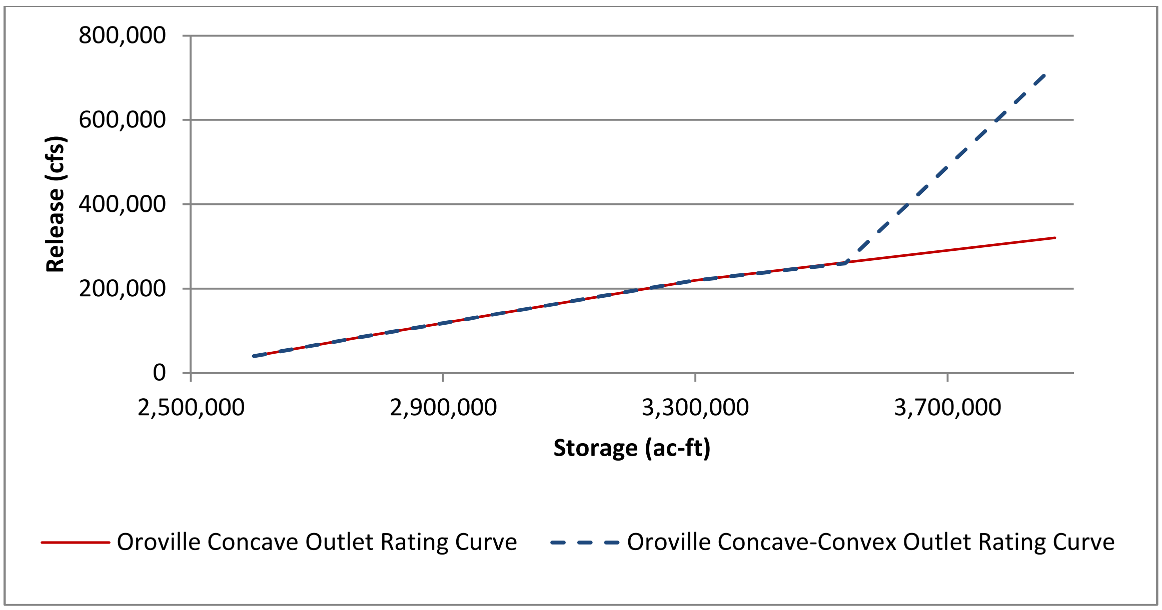


Figure 16. Difference between concave reservoir outlet rating curve and more complicated concave-convex rating curve

3.4 Channel Capacities

Since 1999, several studies have looked at the whole of the Sacramento River Watershed system. One of these studies done as part of the CVFMP, of which the CVFPP falls under, was the State Plan of Flood Control (SPFC) Descriptive Document. This document serves as the most up-to-date inventory and physical description of the Sacramento River System. This document was used to identify any large channel capacity changes in the system since D. Jones' thesis in 1999. Two discrepancies were found in the channel capacities along the

Sacramento River at Moulton and Colusa Weir, and two discrepancies found in the Sutter Bypass near Meridian and below Tisdale Bypass. Table 3 below compares channel capacities of the two studies with the four discrepancies highlighted. The four channel capacities from the SPFC Descriptive Document, which differed from D. Jones' 1999 study, were put into the "01_Current" HEC-ResFloodOpt run as the base model for this study. The effects of these changes along with the different hydrology being used for this study are shown in the next section.

Table 3. Channel capacities comparison table between D. Jones' 1999 thesis and SPFC Descriptive Document. Differences in values are highlighted below.

Location	Design Flow (cfs) from D. Jones' 1999 Study	Design Flow (cfs) from SPFC Descriptive Document
Sacramento River below		
Bend Bridge (just above Red Bluff)	100,000	100,000
Vina-Woodson (just below Red Bluff)	260,000	260,000
Ord Ferry	160,000	160,000
Butte City	160,000	160,000
Moulton Weir	160,000	135,000
Colusa Weir	60,000	65,000
Tisdale Weir	30,000	30,000
Verona	107,000	107,000
Sacramento Bypass	107,000	107,000
Sacramento (I street)	110,000	110,000
Freeport	110,000	110,000
Rio Vista	579,000	579,000
Sutter Bypass		
Below Butte Slough (nr Meridian)	130,000	178,000
Downstream of Tisdale Bypass	180,000	216,500
Downstream of Feather River	380,000	380,000
At confluence w/ Sac River	380,000	380,000
Feather River		
At Gridley	150,000	150,000
Above Yuba River (at Yuba City)	210,000	210,000
At Nicolaus	320,000	320,000
Yuba River at Feather River (Marysville)	120,000	120,000
American River at H Street Bridge	115,000	115,000
Sacramento-Feather River Confluence	410,000	410,000
Yolo Bypass below		
Fremont Weir	343,000	343,000
Woodland	377,000	377,000
Sacramento Bypass	480,000	480,000
Lisbon	490,000	490,000

3.5 Comp Study Data vs. CVHS Data

This thesis uses the updated hydrology data sets from the CVHS currently being completed. In D. Jones' 1999 thesis, he used hydrology data provided from David Ford Consulting Engineers, Inc. and the US Army Corps of Engineers, Sacramento District. It is believed that his hydrology data came from the initial draft deliverables of the Comp Study. Since the comparison of old and new results showed that the program was responding appropriately on a new computer, the next evaluation was the comparison between the "01_Current" HEC-ResFloodOpt run with the CVHS hydrology versus D. Jones' FCMIP model, which utilized hydrology from the Comp study. Figure 5 shows the 17 inflow locations included in the representation of the Sacramento River Watershed used in this study. The CVHS local flows were matched up to the equivalent points used in D. Jones' 1999 thesis. The CVHS hydrology created some differences in this run, but overall, the system ran almost the same and was able to be adequately calibrated to the January 1997 observed flows.

There was no observed data at Rio Vista for the January 1997 flood event due to tidal influences. The main reason that the "01_Current" HEC-ResFloodOpt run is so much higher at Rio Vista than D. Jones' 1999 FCMIP model is due to a limitation found in the routing of the weir flows in HEC-ResFloodOpt. HEC-ResFloodOpt uses two types of routing: 1) user specified linear routing coefficients and 2) Muskingum method. When using a user specified linear routing coefficient of the Sacramento Weir flow, the downstream control point (I-80) did not seem to account for the additional diversion flow. The "01_Current" HEC-ResFloodOpt run replaces the user specified linear routing coefficients routing with the Muskingum method for the Sacramento Weir diversion flow and this resolved the missing flow in the Yolo Bypass. The Lisbon Flow (below I-80) matches the observed peak slightly better than in D. Jones' 1999 study. Perhaps the most important lesson is that both models kept Rio Vista below its capacity of 579,000 cfs, an important check in the overall efficacy of the optimization solution.

Figure 17 through Figure 26 show the results of those two runs for the January 1997 flood event, compared against the observed historical data. Appendix B shows the difference in the Comp Study flows versus CVHS flows that were input into each model.

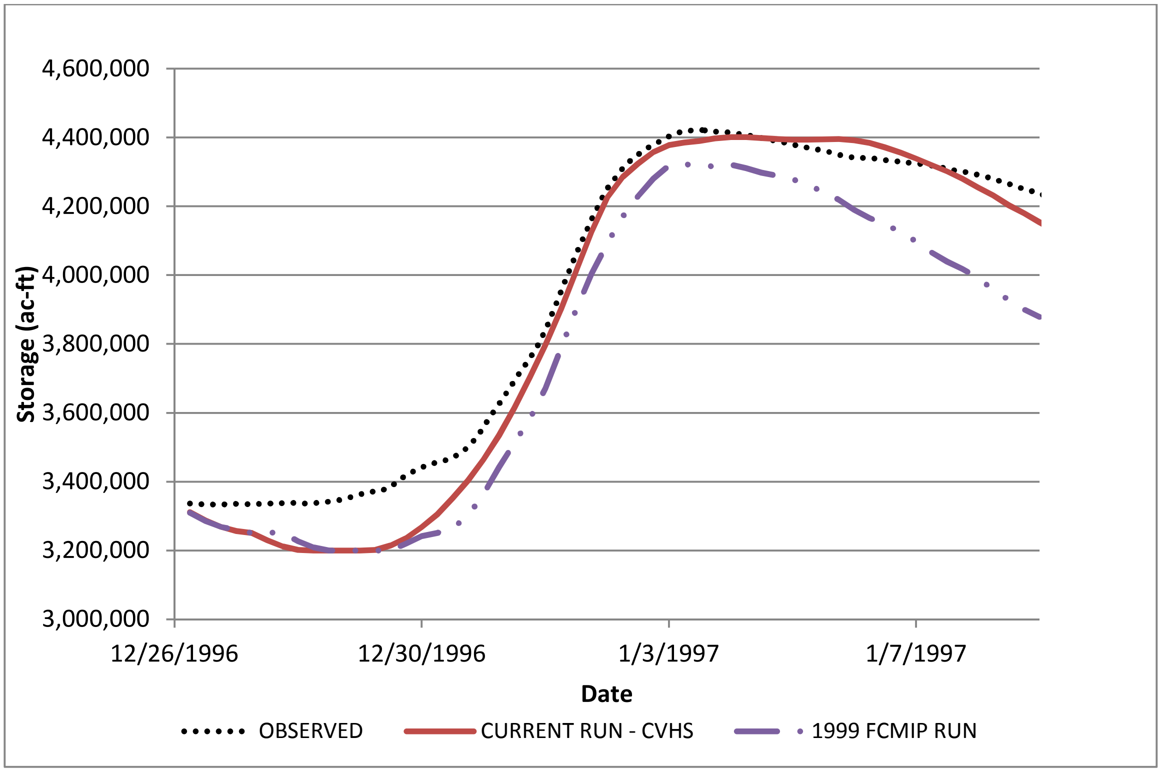


Figure 17. Shasta Dam storage results using the Comp Study data and CVHS data (January 1997 event) versus observed data

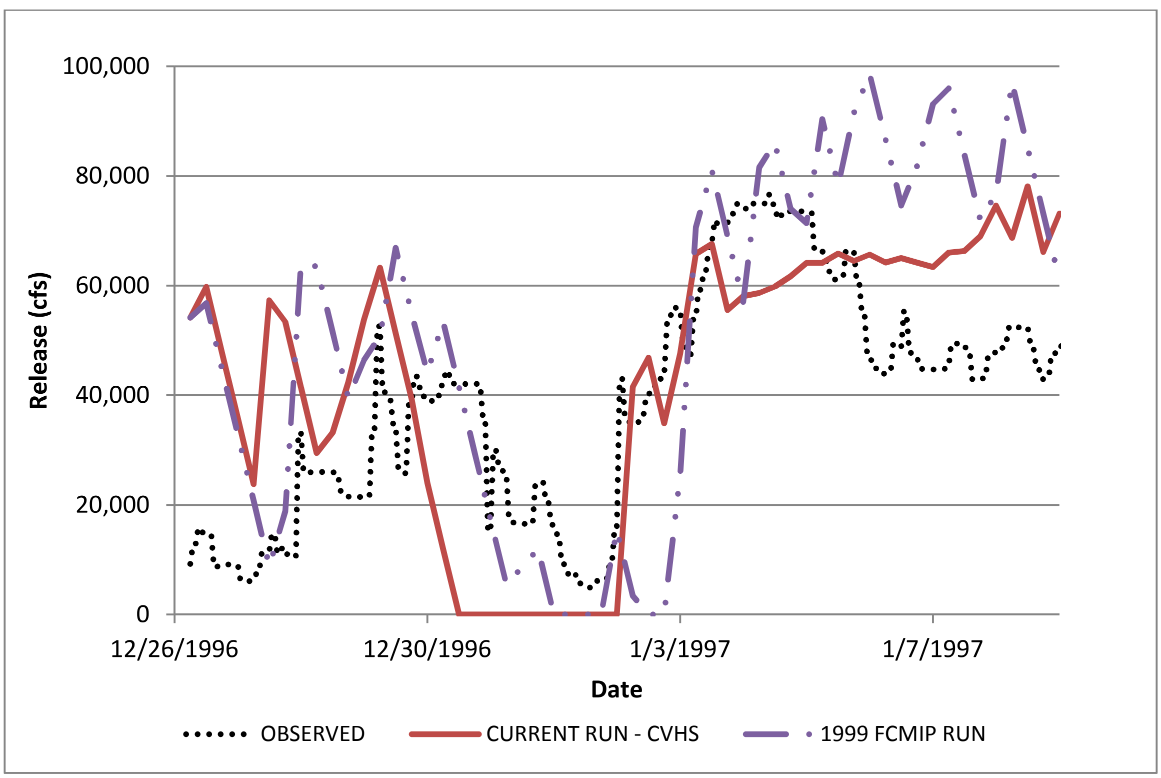


Figure 18. Shasta Dam release results using the Comp Study data and CVHS data (January 1997 event) versus observed data

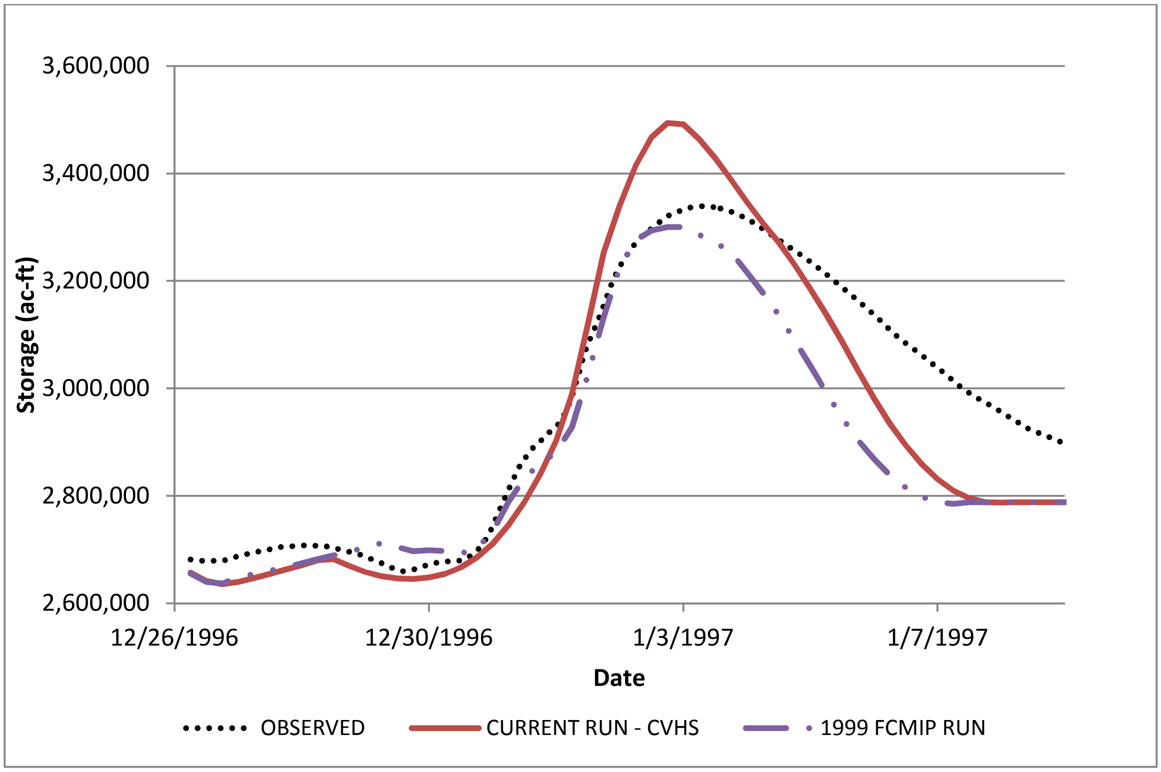


Figure 19. Oroville Dam storage results using the Comp Study data and CVHS data (January 1997 event) versus observed data

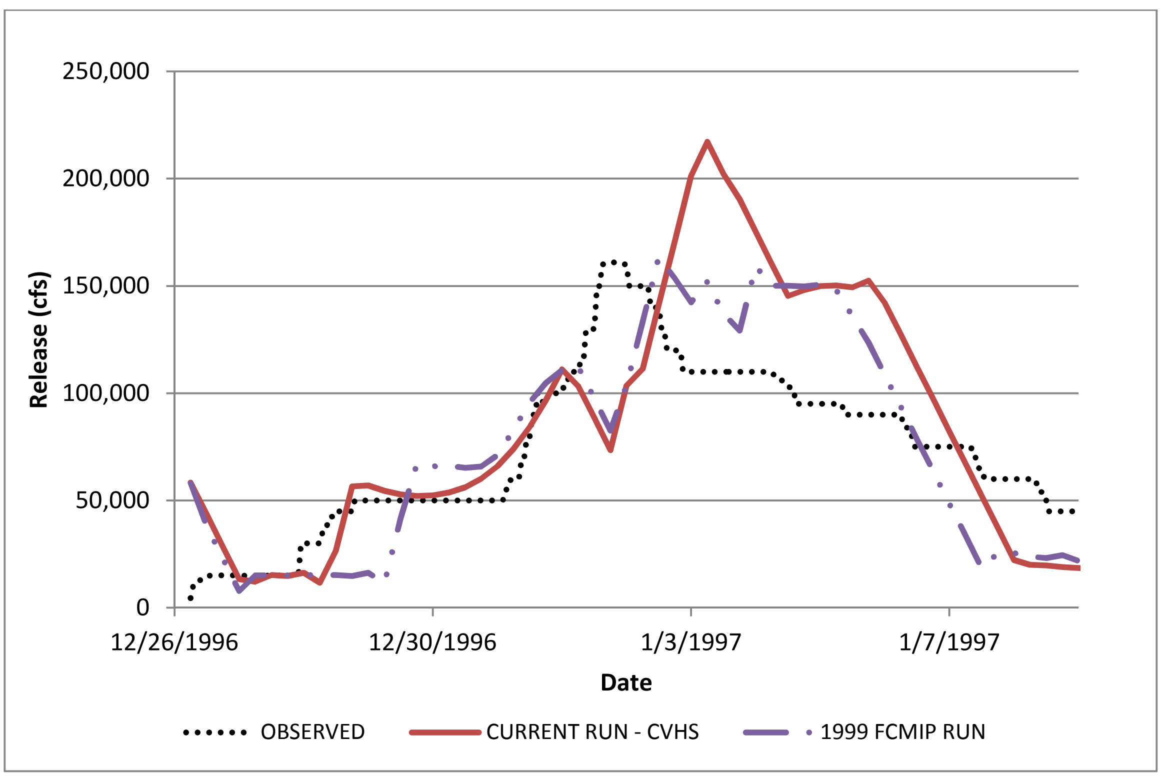


Figure 20. Oroville Dam release results using the Comp Study data and CVHS data (January 1997 event) versus observed data

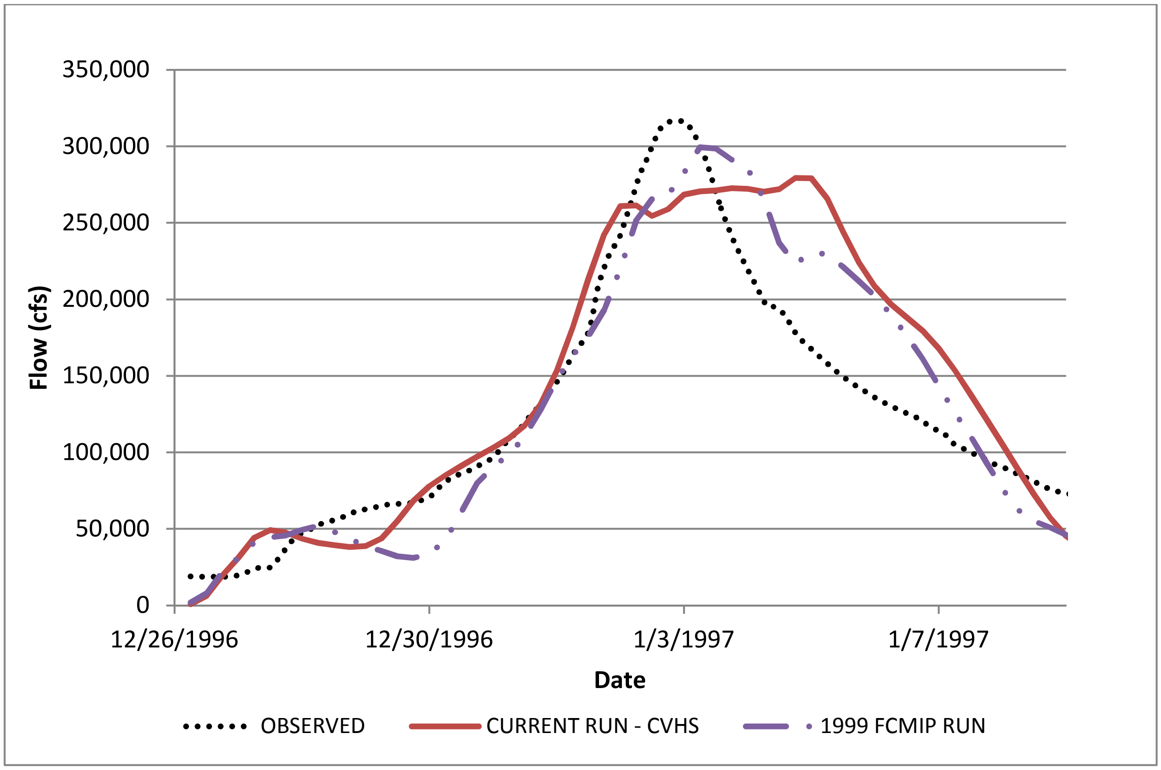


Figure 21. Nicolaus flow results using the Comp Study data and CVHS data (January 1997 event) versus observed data

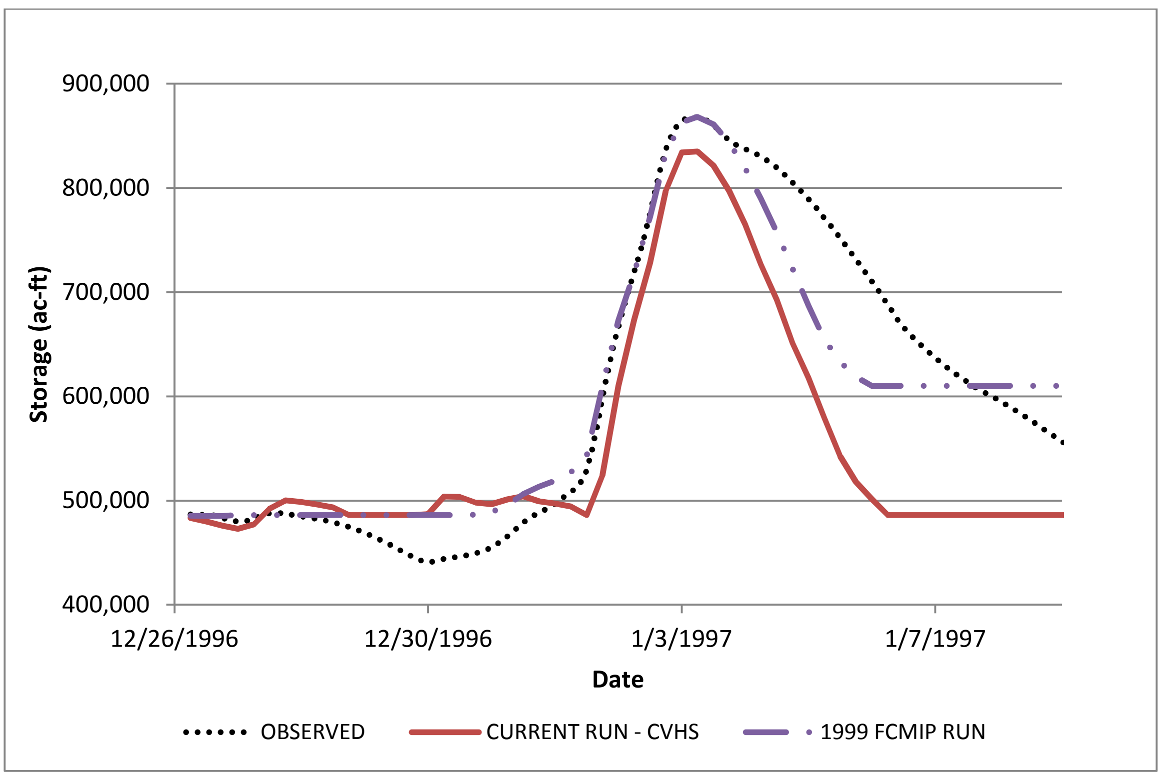


Figure 22. Folsom Dam storage results using the Comp Study data and CVHS data (January 1997 event) versus observed data

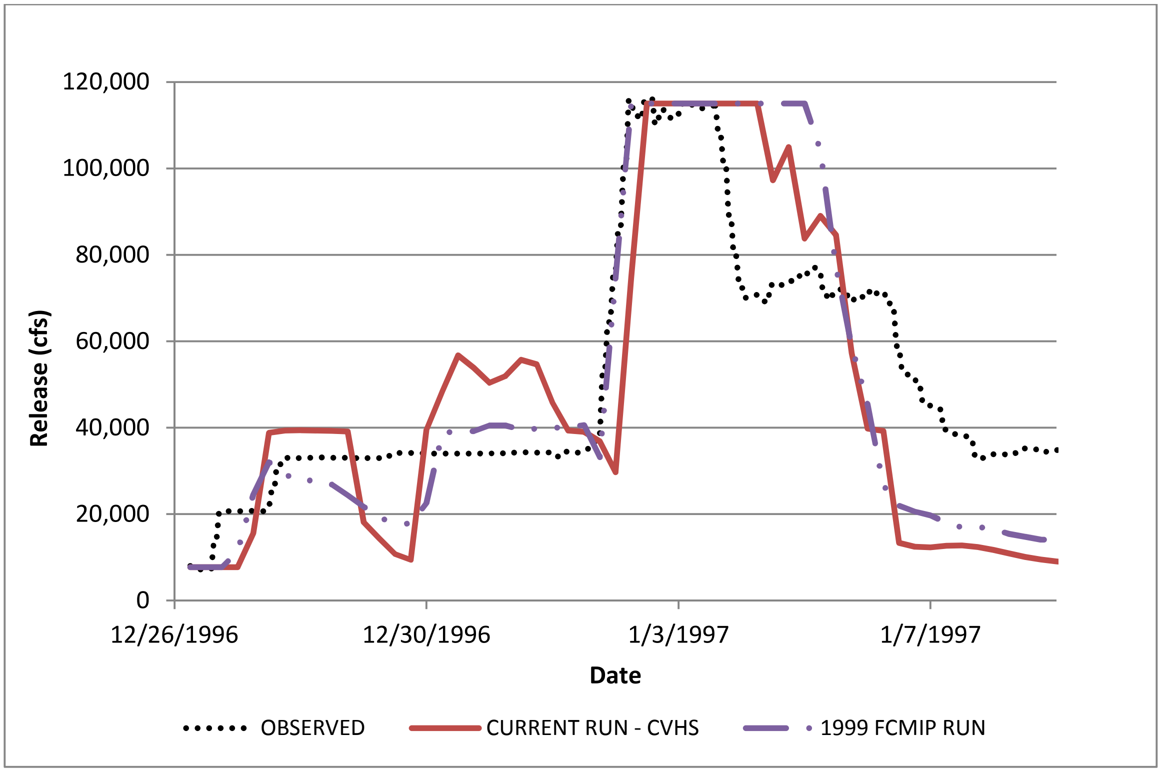


Figure 23. Folsom Dam release results using the Comp Study data and CVHS data (January 1997 event) versus observed data

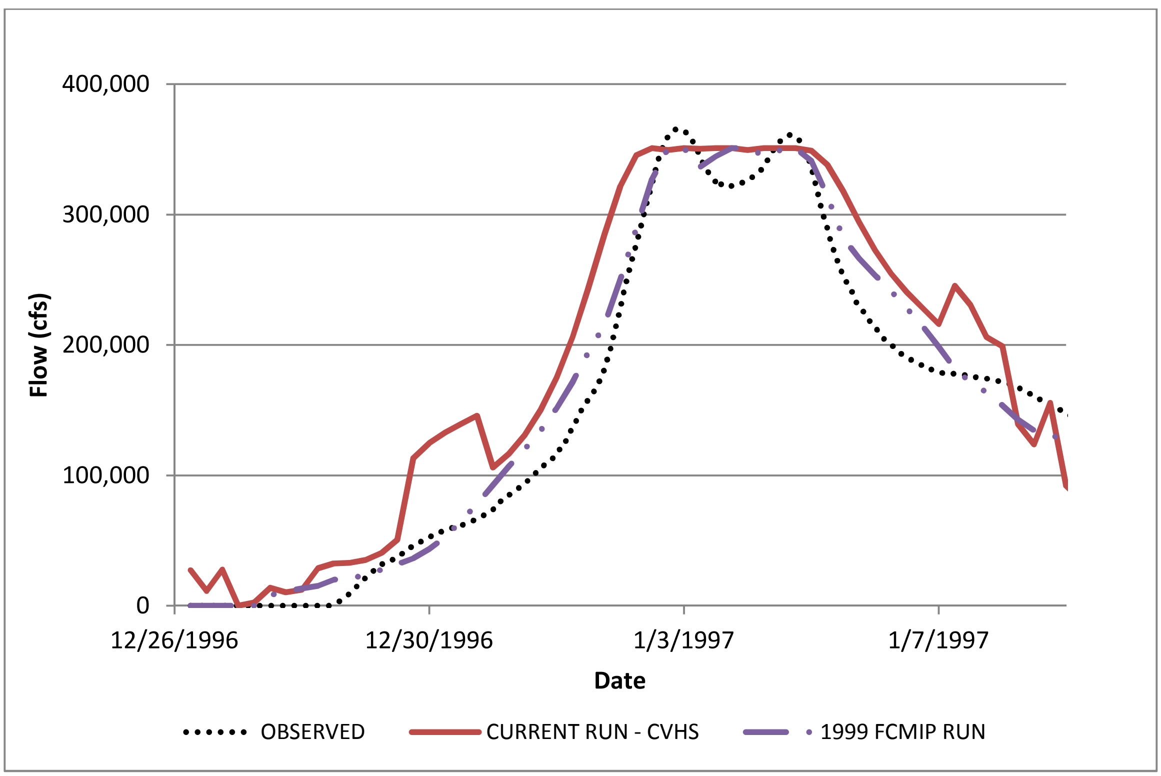


Figure 24. Fremont Weir diversion flow results using the Comp Study data and CVHS data (January 1997 event) versus observed data

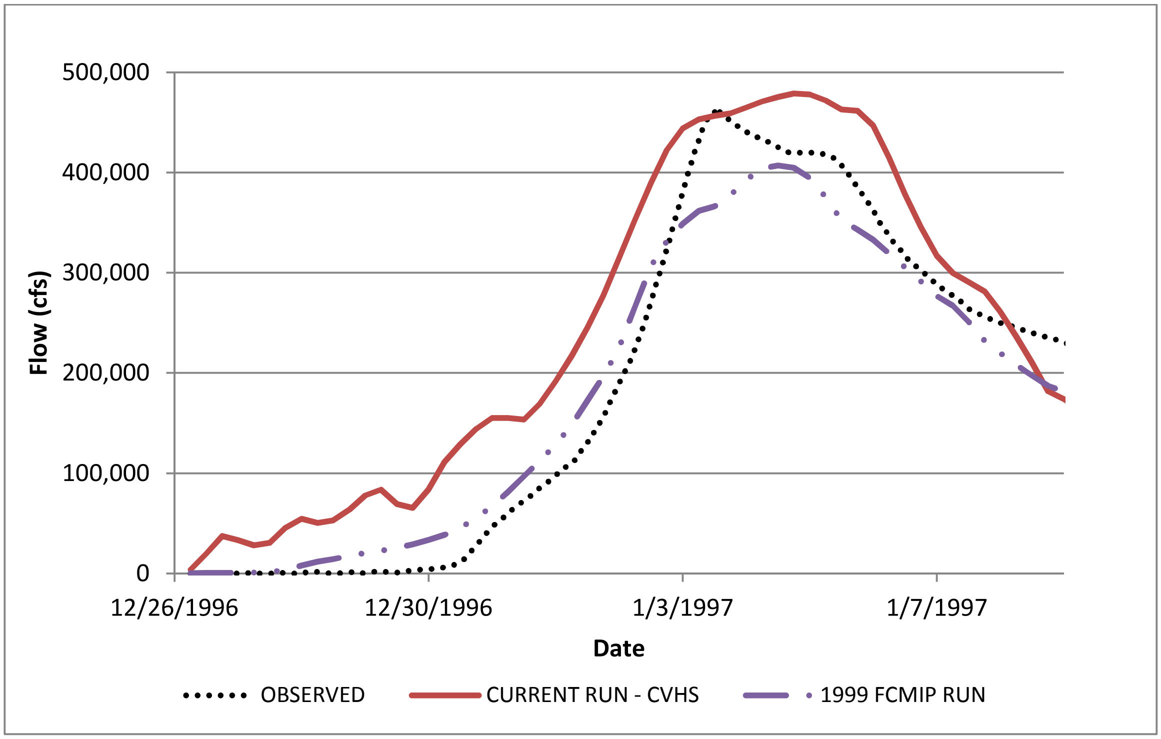


Figure 25. Lisbon flow results using the Comp Study data and CVHS data (January 1997 event) versus observed data

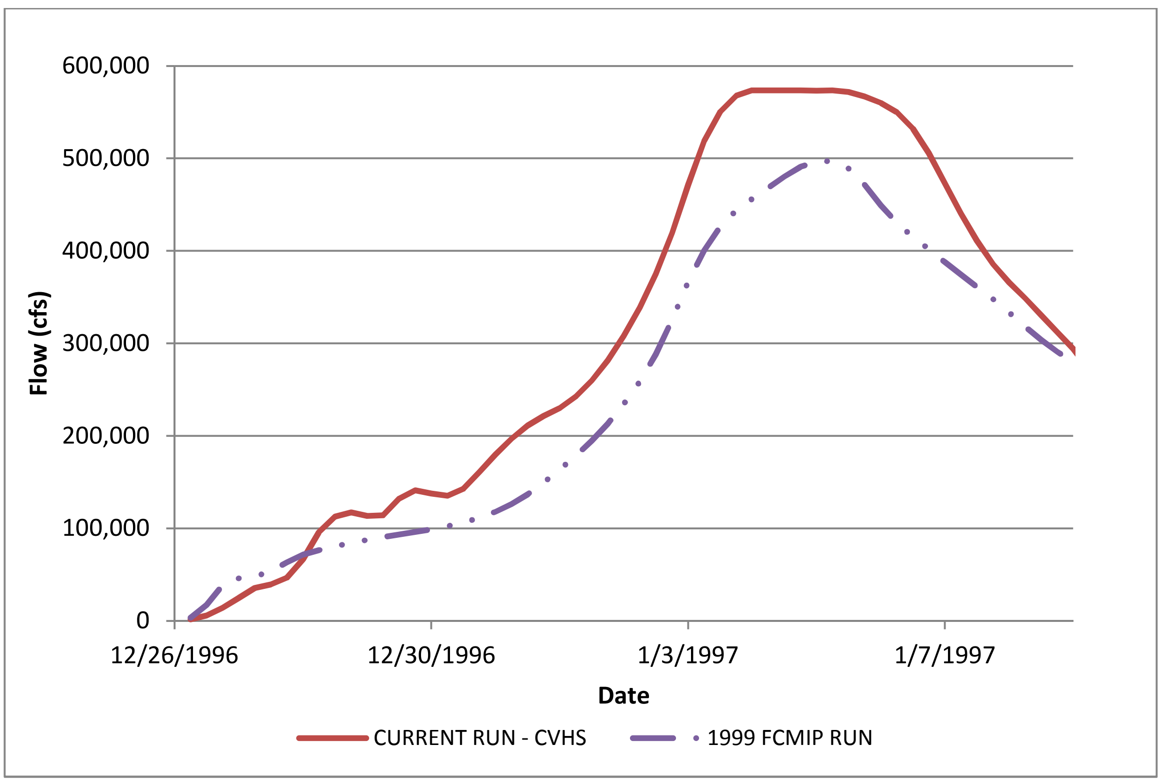


Figure 26. Rio Vista flow results using the Comp Study data and CVHS data (January 1997 event)

CHAPTER 4
RESULTS AND DISCUSSION

4.1 Systemwide Operations versus Individual Reservoir Operations

The overall results from this application of HEC-ResFloodOpt (reservoir storages, reservoir releases, and flows at the downstream control points) are consistent with how the system has operated in the past. There are many places for improvement in this model, as discussed in Chapter 5, but for the purposes of this study it was deemed an appropriate approximation of how the Sacramento River Watershed could function as a whole system when compared to individual operations.

To determine how the optimization model was performing to reduce systemwide penalties (when compared to the individual operation of each reservoir), a HEC-ResSim simulation model was run with the same January 1997 inflow hydrology from CVHS that was used in HEC-ResFloodOpt. Two main differences exist between the HEC-ResSim and HEC-ResFloodOpt modeling efforts. First, HEC-ResSim is a simulation model (i.e. it is incapable of performing optimization). HEC-ResSim has limited foresight to make release decisions, other than rules that implicitly take into account assumptions on future conditions. Each reservoir within HEC-ResSim acts on its own operation rule set without looking at other reservoir releases within the basin unless the two reservoirs operate for a common downstream control point (e.g. Oroville and New Bullards Bar at the Feather-Yuba confluence).Reservoirs within HEC-ResSim mostly act independently to make their releases. Shasta Reservoir in HEC-ResSim will only look as far as Bend Bridge, which is Shasta's furthest downstream control point. Shasta releases will be made based on many rules at the dam itself (i.e. amount of inflow, rate of decrease/increase, storage-outflow relationships, downstream control point rules, etc.), but it does not make decisions based on what Black Butte Dam is releasing into Sacramento River further downstream from Bend Bridge. It does not look at what is coming in from Feather River and Sutter Bypass to add to the Sacramento River at the Fremont Weir. In this regard, HEC-ResSim is limited in how it makes decisions for a systemwide operation.

HEC-ResFloodOpt, on the other hand, explicitly and optimally coordinates reservoirs' releases based on the penalties associated with each downstream point. HEC-ResFloodOpt provides for a much simpler representation of the physical and operational reservoir characteristics as compared to HEC-ResSim; for example, HEC-ResFloodOpt does not handle nearly as many reservoir operation rules. The only rules at each reservoir in HEC-ResFloodOpt include: the definition of storage zones, the storage-outflow curve and the penalties for each storage zone, and the penalties associated with the rate of increase or decrease of release from the reservoir. However, even though there is not a rule associated with specific downstream control points for each reservoir, the reservoirs' release decisions are being made by the program evaluating downstream control points at each time step to determine what flows are occurring and how best to minimize those penalties at each point. What one reservoir releases in a time step can affect what every other reservoir release at several time steps.

The second difference between the two modeling efforts is the explicit adherence to existing reservoir operating rules during a flood event. The HEC-ResSim modeling was performed as part of CVHS; this study sought to represent as accurately as possible the rules in each reservoir's existing water control manual. In real-time, reservoir operators do not necessarily follow these rules explicitly due to physical and/or operational constraints that are outside of their control. This difference in "operating philosophy" can result in significant differences in the resulting reservoir pool elevation and outflow assumptions during a simulated flood event, when compared to observed data. HEC-ResFloodOpt, on the other hand, is calibrated to match observed operations, which inherently results in a closer match between modeled and observed data. In summary, the differences identified between HEC-ResSim and HEC-ResFloodOpt output should not be attributed solely to differences between optimization and simulation modeling approaches. That said, meaningful observations can be made through the direct comparison of these model outputs, as described below.

Table 4 below shows the difference in peak flows between the two modeling efforts against the historical observed peak in January 1997 and the overall channel capacity. Both modeling efforts, have periods when the flow exceeds the channel capacity, but this is to be expected based on what was observed in the actual 1997 event. What can be shown by this summary of flow peaks is that HEC-ResSim tried to meet most downstream objectives of each reservoir. However, for further downstream points such as Woodland, Lisbon, and Rio Vista, the model allowed an aggregated outflow that exceeded known capacities due to a lack of comprehensive rules to prevent this type of operation. HEC-ResFloodOpt, on the other hand, prioritized a minimization of capacity exceedances at the most downstream control points (with subsequent highest damage potential) while compromising at times with intermediate control point operations.

Table 4. Difference in peak flows between HEC-ResFloodOpt and HEC-ResSim for the January 1997 event

Control Point	Channel Capacity (cfs)	Observed Peak (cfs)	HEC-ResFloodOpt Peak (cfs)	HEC-ResSim Peak (cfs)
Bend Bridge	100,000	121,070	114,745	129,009
Vina-Woodson	260,000	154,000	155,319	170,038
Ord Ferry	160,000	118,332	107,747	135,625
Butte City	160,000	146,520	107,112	135,218
Moulton Weir	135,000	119,699	88,200	109,183
Colusa Weir	65,000	58,204	42,264	48,264
Tisdale Weir	30,000	40,882	25,433	28,153
Meridian	178,000	140,000	142,435	138,088
RD 1500	216,500	N/A	158,534	157,942
Yuba City	210,000	165,721	205,800	179,210
Marysville	120,000	143,880	128,865	170,359
Nicolaus	320,000	319,133	279,312	344,453
Fair Oaks	115,000	116,650	115,000	115,000
Sacramento (I St)	110,000	107,520	131,571	112,461
Freeport	110,000	114,900	131,129	111,847
Woodland	377,000	396,550	368,125	547,585
Lisbon	490,000	460,394	478,876	547,585
Rio Vista	579,000	N/A	573,406	654,359

The other reason for differences between the outcomes of the two modeling efforts described above is the relative lack of foresight in the HEC-ResSim model. Not only does HEC-ResSim have its reservoirs look only as far as their downstream control point, it also only has a limited foresight to look at a time series only as far out as the time it takes to route a release down to that specific control point (Joan Klipsch, 2013, personal communication). This limited foresight changes how a reservoir operates within the basin. Figure 27 through Figure 36 show that, for reservoirs that have downstream control points in their operation rule sets (Shasta, Oroville, and New Bullards Bar) in HEC-ResSim, the model results in similar storage outcomes to that of HEC-ResFloodOpt. New Bullards Bar Reservoir is the exception. This is largely because New Bullards Bar's furthest downstream point is the confluence of the Yuba and Feather rivers. Therefore, HEC-ResSim was releasing based on the maximum capacity at that confluence. HEC-ResFloodOpt was looking even further downstream at the Feather River at Nicolaus, which was under channel capacity within HEC-ResFloodOpt, but over channel capacity within HEC-ResSim during the peak flow period. For Black Butte Reservoir, HEC-ResFloodOpt held more water back early and released more water later in the storm to mitigate for flows coming from Shasta Reservoir into the upper Sacramento River at the beginning of the storm. On the other hand, at Folsom Reservoir, HEC-ResFloodOpt released more water in the beginning of the storm to evacuate more water in the reservoir to be able to handle the larger second peak apparent in the inflow hydrology.

With all of the contrasts in operation described above, each model nevertheless produced results that reasonably simulated observed operation for the January 1997 flood event. A primary purpose of HEC-ResFloodOpt is to look at the systemwide reservoir functions. A logical approach would be to take results from the optimization model and use them to guide modifications to the active simulation model, to assess how those modifications function against current water control manual rules. This approach creates the potential for future in depth systemwide studies that could be performed by an agency such as CA DWR.

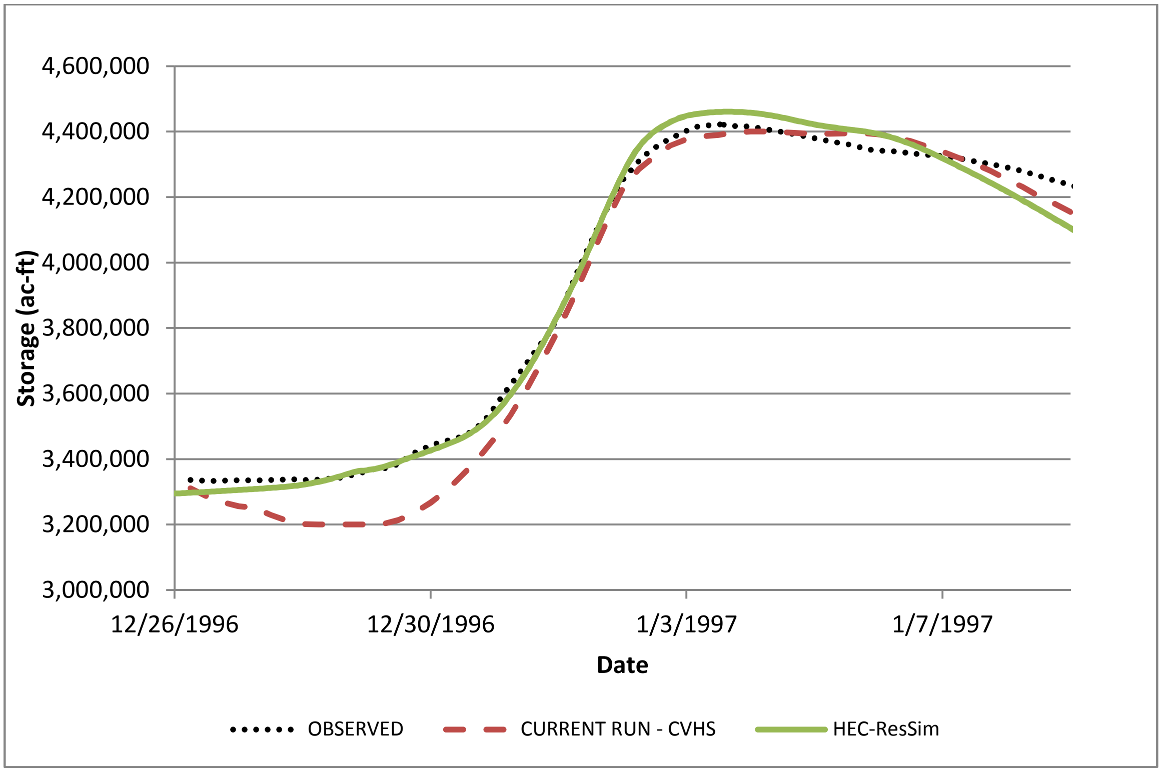


Figure 27. 1997 Shasta Dam storage for observed, HEC-ResSim, and HEC-ResFloodOpt results

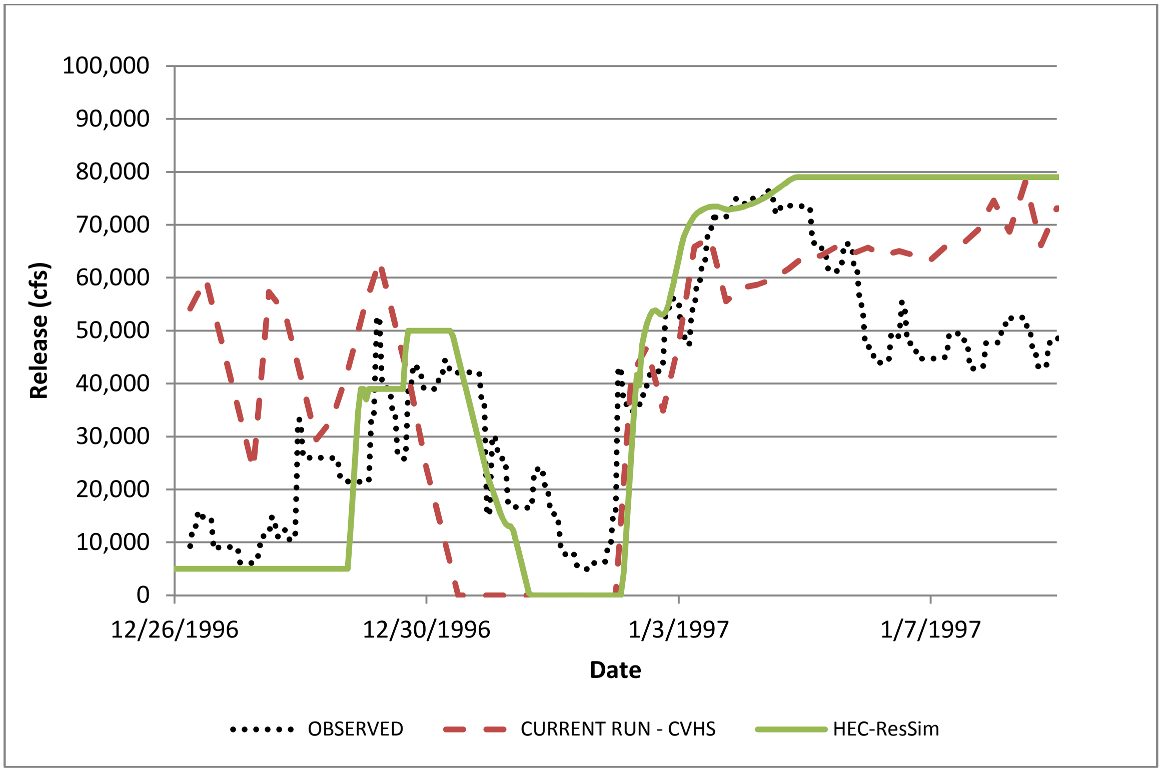


Figure 28. 1997 Shasta Dam release for observed, HEC-ResSim, and HEC-ResFloodOpt results

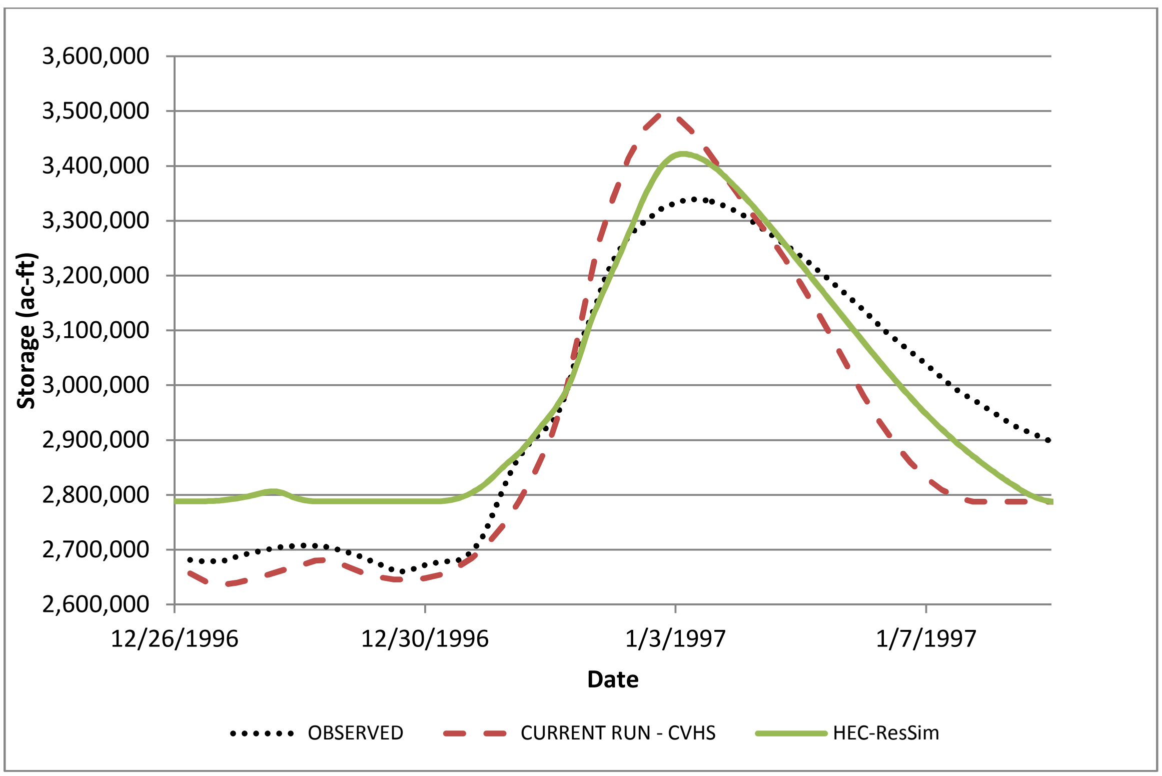


Figure 29. 1997 Oroville Dam storage for observed, HEC-ResSim, and HEC-ResFloodOpt results

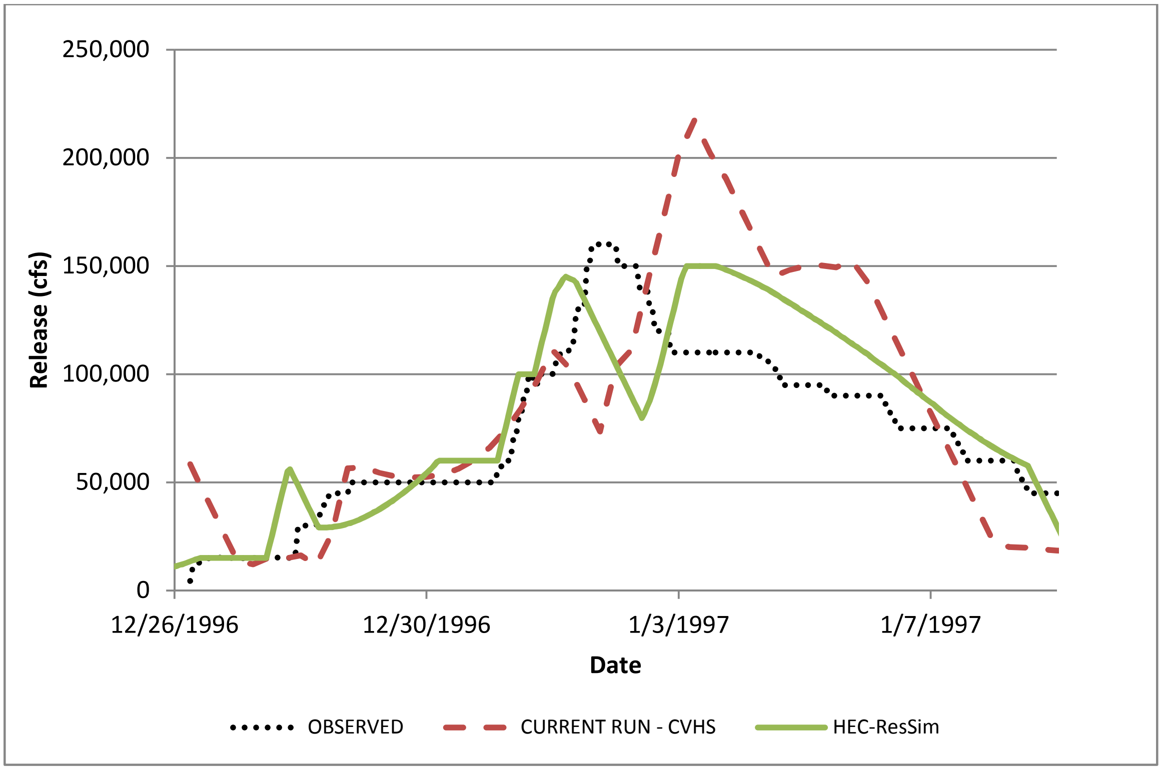


Figure 30. 1997 Oroville Dam release for observed, HEC-ResSim, and HEC-ResFloodOpt results

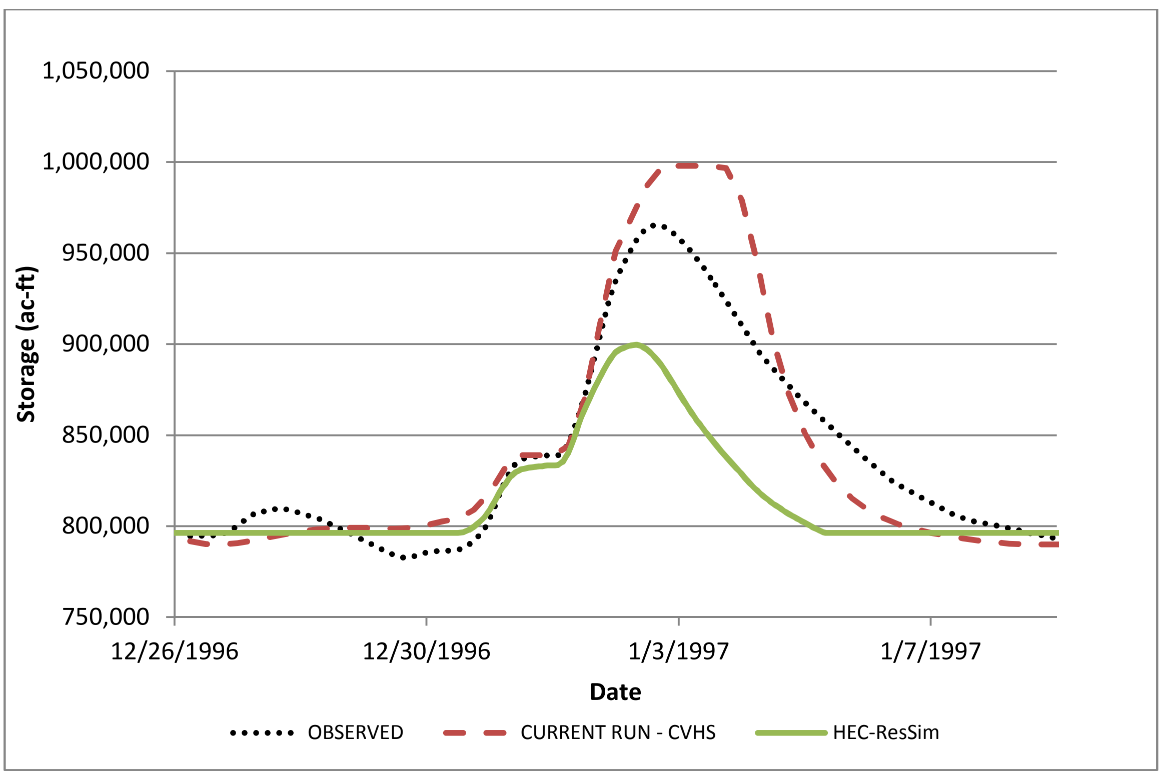


Figure 31. 1997 New Bullards Bar Dam storage for observed, HEC-ResSim, and HEC-ResFloodOpt results

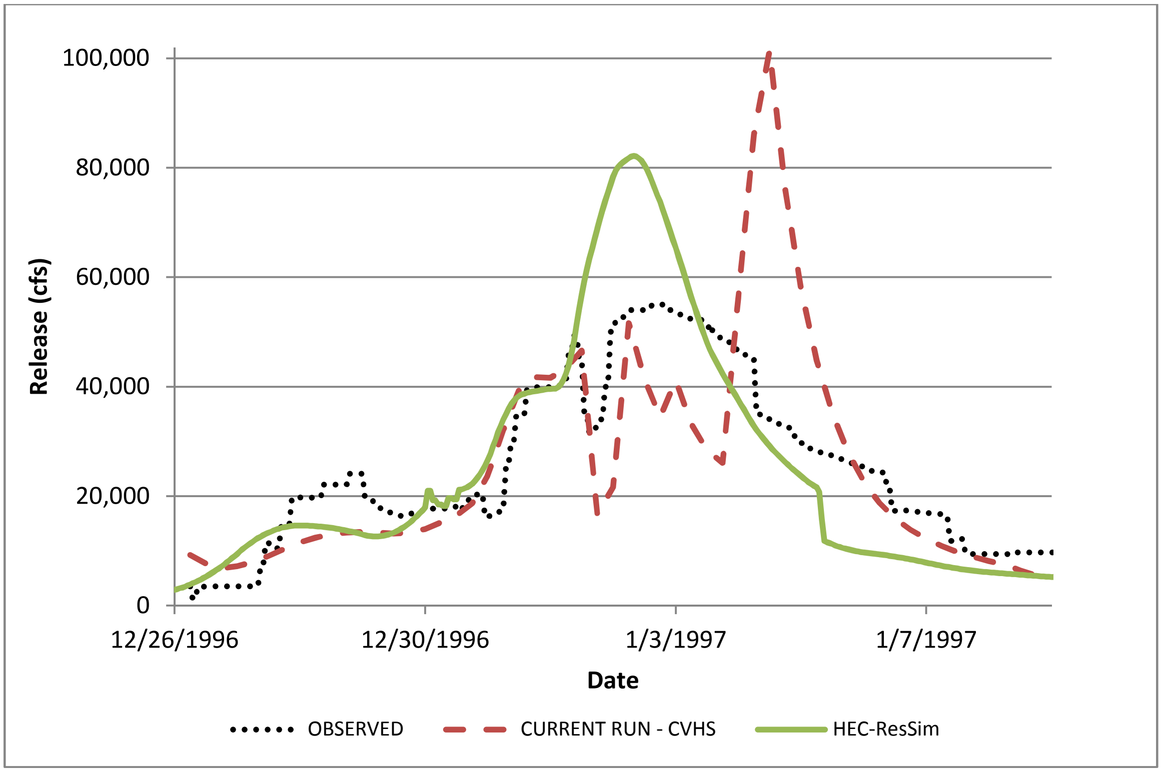


Figure 32. 1997 New Bullards Bar Dam release for observed, HEC-ResSim, and HEC-ResFloodOpt results

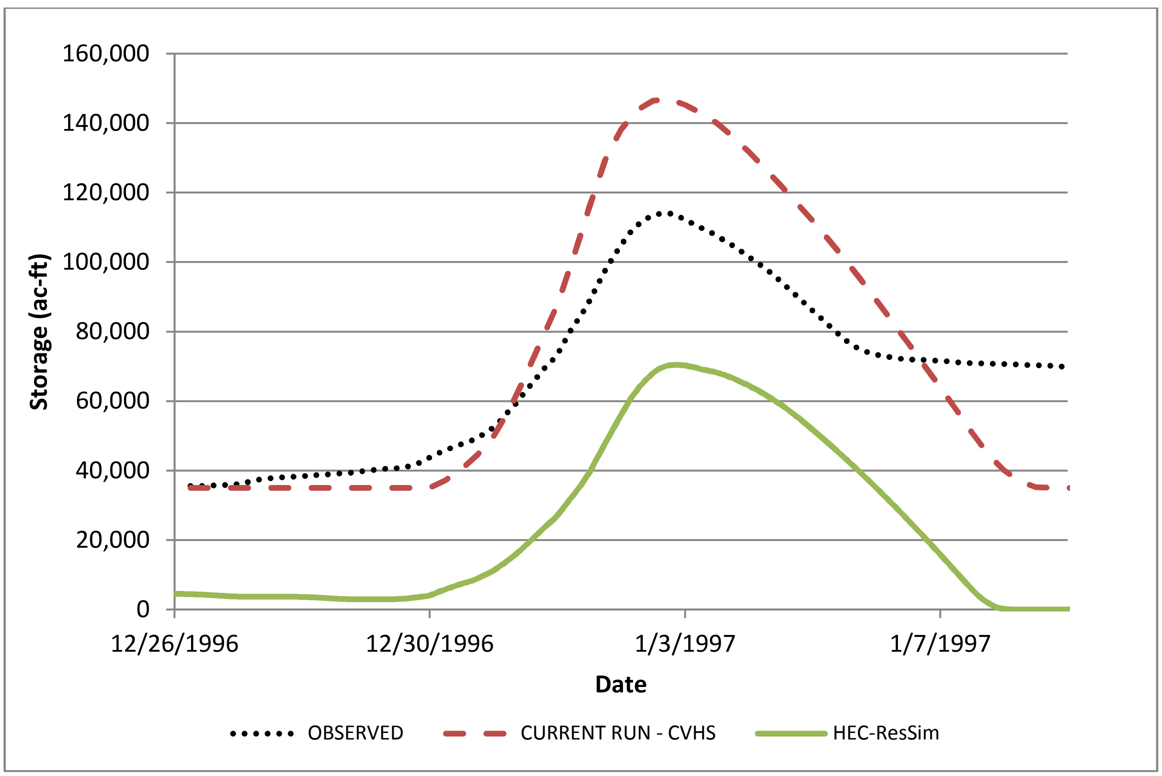


Figure 33. 1997 Black Butte Dam storage for observed, HEC-ResSim, and HEC-ResFloodOpt results

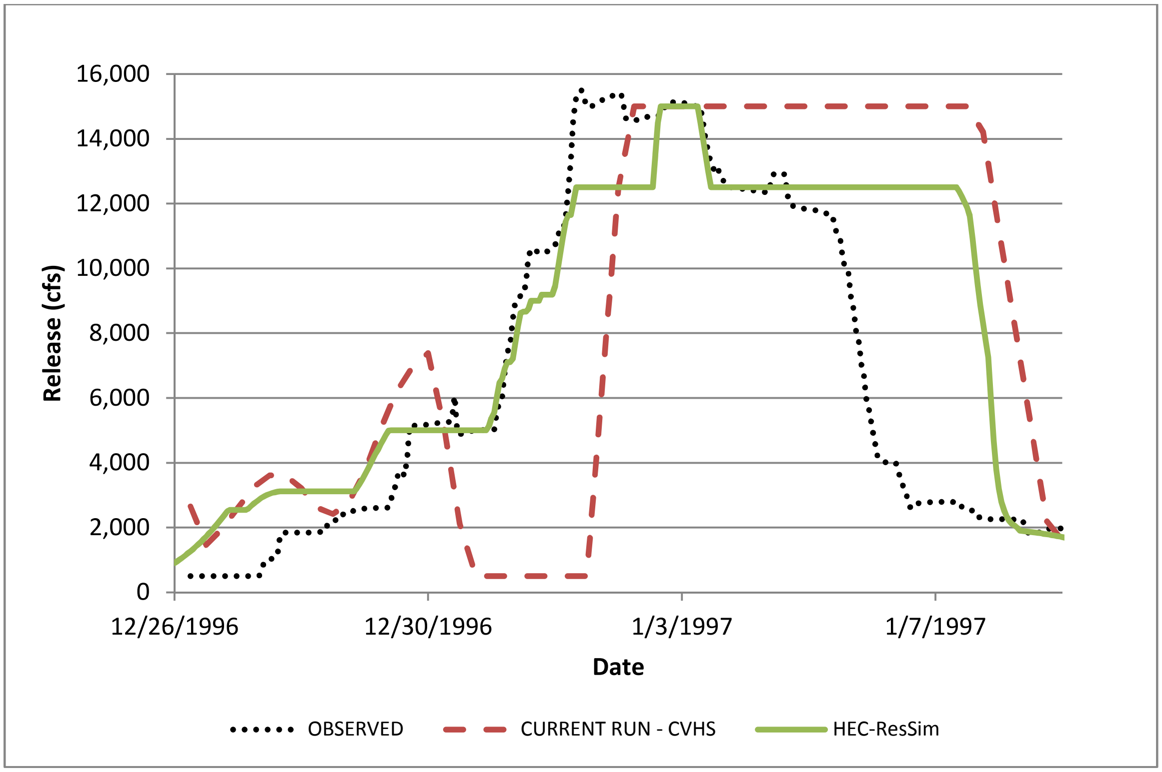


Figure 34. 1997 Black Butte Dam release for observed, HEC-ResSim, and HEC-ResFloodOpt results

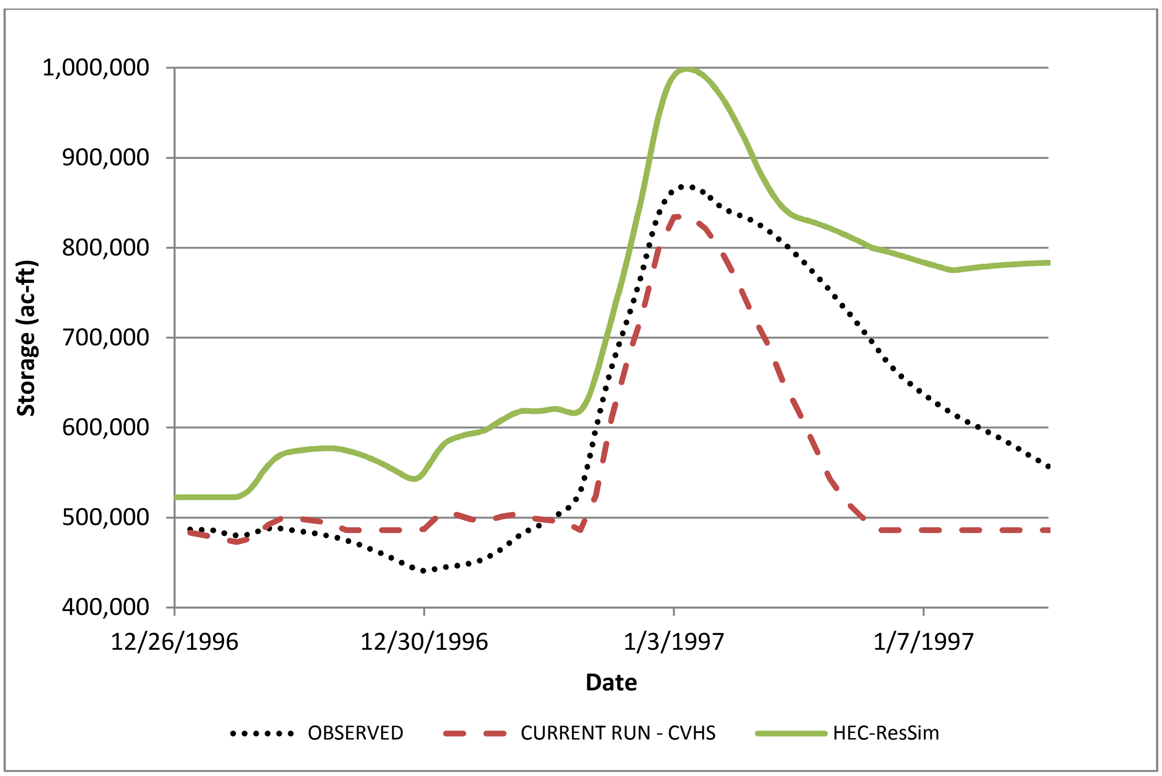


Figure 35. 1997 Folsom Dam storage for observed, HEC-ResSim, and HEC-ResFloodOpt results

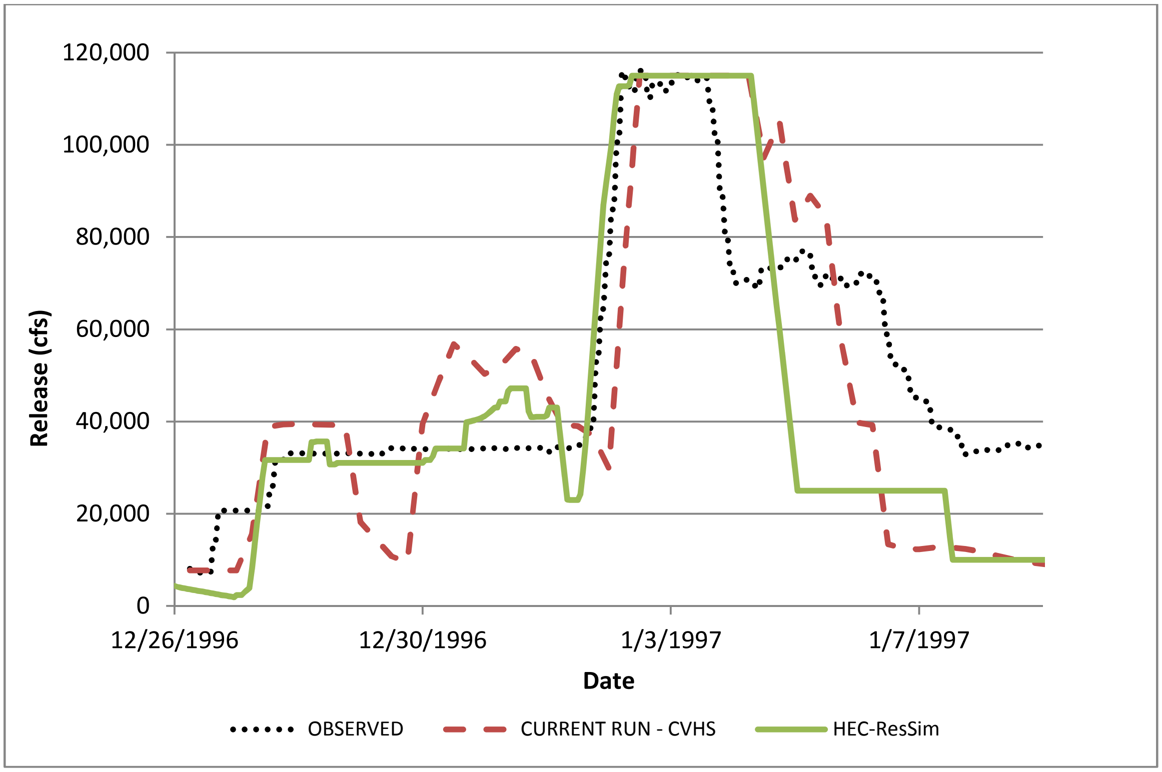


Figure 36. 1997 Folsom Dam release for observed, HEC-ResSim, and HEC-ResFloodOpt results

4.2 System Expansion Alternatives

As explained in Section 2.4, ten HEC-ResFloodOpt cases were run for the Sacramento River Watershed. The first run was “01_Current,” which is the base model for this study. It is meant to portray existing flood management infrastructure within the watershed without changes. The other nine cases have at least one infrastructure expansion added to the system or a combination of expansions. The purpose of adding each of those infrastructure expansions was to estimate how much flood damage and penalty reduction benefit could be achieved within the system.

The next element of study for the ten cases was to look at how the February 1986 and January 1997 storms influenced the amount of expected damage. The 1997 storm was chosen because it generally resulted in some of the highest recorded flows ever observed across the Sacramento River Watershed. The 1986 storm was chosen because it was an almost equally powerful storm as the 1997, but it had a double peak and it was not clear how that would be dealt with in HEC-ResFloodOpt (if indeed any differently than a storm with a single peak). A cursory frequency analysis was performed by the author on the reservoir inflow unregulated time series for each event to estimate a return period for each storm at each reservoir. To make that estimate, the author chose to analyze the 3-day average peak flows for each storm. Once the average peak flows were calculated, they were compared against their respective “Rain

Flood Frequency Curve for Unregulated Conditions" from the Comp Study to estimate the return period and annual exceedence probability (AEP). The results are shown in Table 5 and Table 6.

Table 5. Return periods and their associated annual exceedence probabilities (AEPs) for the February 1986 scaled floods run through the optimization model

Return Period in years (AEP) for February 1986 - 3-Day Average Peak Flood Flows						
Reservoir	**1986**	**1986*1.2**	**1986*1.4**	**1986*1.6**	**1986*1.8**	**1986*2.0**
Folsom	55 (0.018)	88 (0.011)	132 (0.008)	204 (0.005)	270 (0.004)	380 (0.003)
Oroville	39 (0.026)	65 (0.015)	110 (0.009)	175 (0.006)	257 (0.004)	390 (0.003)
New Bullards Bar	43 (0.023)	76 (0.013)	135 (0.007)	231 (0.004)	377 (0.003)	628 (0.002)
Shasta	18 (0.054)	39 (0.026)	78 (0.013)	161 (0.006)	313 (0.003)	657 (0.002)
Black Butte	17 (0.059)	28 (0.036)	46 (0.022)	69 (0.014)	110 (0.009)	165 (0.006)

****AFC - Above Frequency Curve**

Table 6. Return periods and their associated annual exceedence probabilities (AEPs) for the January 1997 scaled floods run through the optimization model

Return Period in years (AEP) for January 1997 - 3-Day Average Peak Flood Flows						
Reservoir	**1997**	**1997*1.2**	**1997*1.4**	**1997*1.6**	**1997*1.8**	**1997*2.0**
Folsom	31 (0.032)	49 (0.02)	70 (0.014)	104 (0.01)	140 (0.007)	200 (0.005)
Oroville	88 (0.011)	161 (0.006)	273 (0.004)	501 (0.002)	776 (0.001)	AFC**
New Bullards Bar	103 (0.01)	209 (0.005)	384 (0.003)	728 (0.001)	AFC**	AFC**
Shasta	110 (0.009)	300 (0.003)	911 (0.001)	AFC**	AFC**	AFC**
Black Butte	11 (0.092)	17 (0.06)	26 (0.039)	39 (0.026)	56 (0.018)	80 (0.013)

****AFC - Above Frequency Curve**

The 1986 storm was a much smaller event than 1997 on the Sacramento, Feather, and Yuba systems and relatively equal on the American River and Stony Creek systems. After the original 1986 and 1997 storms were run through HEC-ResFloodOpt, each storm was scaled up by 20%, 40%, 60%, 80%, and 100% to see the effects on the system as the storm increased. As mentioned earlier, HEC-ResFloodOpt calculates both persuasion penalties and peak flow damages, which also represent a penalty in the model. The overall sum of these two penalties are output by the program, but for the purpose of showing just the peak flow damage penalties, the output hydrographs were exported into Microsoft Excel and the peak flow penalties were post-processed, as described in Chapter 2 (Equations [27] and [28]). Table 7 summarizes the 1986 peak flow damage penalties and Table 8 summarizes the 1997 peak flow damage penalties with the persuasion penalties removed.

Table 7. 1986 total peak flow damages ($1,000)

Case Runs	1986 ($1,000)	1986*1.2 ($1,000)	1986*1.4 ($1,000)	1986*1.6 ($1,000)	1986*1.8 ($1,000)	1986*2.0 ($1,000)
01_Current	161,769	728,272	6,445,574	16,212,274	48,775,736	266,236,060
02_SBWiden	161,769	728,272	6,445,459	16,210,958	48,774,846	266,235,170
03_FWWiden	161,249	758,606	4,085,937	5,773,442	47,122,042	242,507,428
04_YBWiden	158,619	709,680	6,422,436	16,191,116	48,752,790	266,167,741
05_SWWiden	160,046	725,812	7,549,237	15,972,552	44,894,303	261,134,665
06_CBAdd	163,102	728,663	6,427,761	16,649,454	49,175,181	258,888,878
07_SBFWWiden	161,249	758,606	4,085,989	5,772,552	47,121,152	242,506,538
08_SBFWYBWiden	158,492	737,240	4,065,499	5,758,312	47,077,747	242,444,446
09_SBFWYBSWWiden	158,386	736,096	4,058,986	6,898,370	44,521,554	238,185,734
10_FWYBWiden	158,492	737,241	4,064,713	5,751,510	47,078,559	242,445,336

Table 8. 1997 total peak flow damages ($1,000)

Case Runs	1997 ($1,000)	1997*1.2 ($1,000)	1997*1.4 ($1,000)	1997*1.6 ($1,000)	1997*1.8 ($1,000)	1997*2.0 ($1,000)
01_Current	257,641	6,276,534	15,949,283	62,040,632	102,160,148	144,318,729
02_SBWiden	257,641	6,276,534	15,948,475	62,039,742	102,158,948	144,316,949
03_FWWiden	287,287	502,727	2,253,158	39,309,679	72,137,513	108,706,122
04_YBWiden	254,570	6,255,549	15,928,056	61,977,468	102,105,447	144,256,731
05_SWWiden	255,612	6,276,072	15,945,619	62,432,817	98,958,091	139,305,725
06_CBAdd	255,111	6,383,934	16,618,813	54,054,929	92,757,795	134,475,377
07_SBFWWiden	287,287	502,534	2,252,268	39,308,789	72,136,608	108,704,342
08_SBFWYBWiden	265,741	482,110	2,227,407	39,246,809	72,074,595	108,639,380
09_SBFWYBSWWiden	267,524	480,475	2,226,817	39,248,331	71,184,189	105,024,145
10_FWYBWiden	265,741	480,788	2,228,297	39,247,699	72,075,500	108,641,160

There was little difference with how HEC-ResFloodOpt dealt with a single peak storm versus a double peak storm due to the perfect foresight of the optimization. Since this study is more focused on how the system will react to the largest of historical storms, the following analysis concentrates mostly on the 1997 results. After looking at the frequencies and amount of damage incurred to the system above, the 140% scaled storm resulted in the most useful result from the standpoint of testing the system to its overall physical limits. Once the storm went beyond the 140% scale factor, the system capacities became overwhelmed and therefore the model did not produce results pertinent for this study. However, when the total penalties were calculated for each expansion for each storm scaling and then sorted from the smallest amount of damage to the most damage, it became apparent which expansions provided the most benefit. Table 9 through Table 14 shows the total penalties and the percent reduction in penalty units calculated for each run sorted on the 1997 event from smallest to largest.

Table 9. Total penalties and percent reduction in penalty units from the "01_Current" run for 1986 and 1997 base storm events, sorted by 1997 results smallest to largest

*1.0	1997	1997 %Diff from "01_Current"	1986	1986 %Diff from "01_Current"
10_FWYBWiden	4,671,405	22.53%	2,886,723	0.65%
08_SBFWYBWiden	4,671,405	22.53%	2,886,722	0.65%
09_SBFWYBSWWiden	4,678,518	22.41%	2,884,274	0.74%
07_SBFWWiden	4,743,878	21.33%	2,901,809	0.14%
03_FWWiden	4,743,878	21.33%	2,901,808	0.14%
06_CBAdd	5,973,512	0.94%	2,888,937	0.58%
04_YBWiden	6,018,809	0.18%	2,892,352	0.46%
05_SWWiden	6,026,902	0.05%	2,901,002	0.16%
01_Current	6,029,951	--	2,905,752	--
02_SBWiden	6,029,951	--	2,905,752	--

Table 10. Total penalties and percent reduction in penalty units from the "01_Current" run for 120% scaled 1986 and 1997 storm events, sorted by 1997 results smallest to largest

*1.2	1997	1997 %Diff from "01_Current"	1986	1986 %Diff from "01_Current"
08_SBFWYBWiden	9,005,282	48.06%	6,864,650	5.79%
09_SBFWYBSWWiden	9,008,169	48.04%	6,853,981	5.94%
10_FWYBWiden	9,008,209	48.04%	6,864,658	5.79%
07_SBFWWiden	9,097,527	47.53%	6,936,248	4.81%
03_FWWiden	9,099,295	47.52%	6,936,248	4.81%
06_CBAdd	16,815,130	3.01%	7,241,031	0.62%
04_YBWiden	17,254,630	0.48%	7,238,563	0.66%
01_Current	17,337,514	--	7,286,455	--
02_SBWiden	17,337,514	--	7,286,456	--
05_SWWiden	17,337,816	--	7,282,204	0.06%

Table 11. Total penalties and percent reduction in penalty units from the "01_Current" run for 140% scaled 1986 and 1997 storm events, sorted by 1997 results smallest to largest

*1.4	1997	1997 %Diff from "01_Current"	1986	1986 %Diff from "01_Current"
08_SBFWYBWiden	23,084,635	38.63%	13,444,586	30.06%
09_SBFWYBSWWiden	23,088,245	38.62%	13,442,258	30.08%
10_FWYBWiden	23,095,459	38.60%	13,448,864	30.04%
07_SBFWWiden	23,216,398	38.27%	13,562,620	29.45%
03_FWWiden	23,227,220	38.25%	13,567,204	29.43%
06_CBAdd	34,743,474	7.63%	19,139,272	0.44%
04_YBWiden	37,485,225	0.34%	19,115,721	0.56%
02_SBWiden	37,601,745	0.03%	19,223,845	--
05_SWWiden	37,611,420	--	18,908,880	1.64%
01_Current	37,612,458	--	19,224,323	--

Table 12. Total penalties and percent reduction in penalty units from the "01_Current" run for 160% scaled 1986 and 1997 storm events, sorted by 1997 results smallest to largest

*1.6	1997	1997 %Diff from "01_Current"	1986	1986 %Diff from "01_Current"
09_SBFWYBSWWiden	67,034,065	26.97%	23,293,759	34.96%
08_SBFWYBWiden	67,030,184	26.98%	24,363,359	31.97%
10_FWYBWiden	67,047,579	26.96%	24,381,141	31.92%
07_SBFWWiden	67,234,116	26.75%	24,523,479	31.53%
03_FWWiden	67,251,512	26.73%	24,537,998	31.49%
05_SWWiden	91,350,600	0.48%	33,319,585	6.97%
06_CBAdd	83,143,888	9.42%	35,032,150	2.18%
04_YBWiden	91,582,203	0.23%	35,668,434	0.41%
02_SBWiden	91,773,364	0.02%	35,805,884	0.02%
01_Current	91,791,218	--	35,814,662	--

Table 13. Total penalties and percent reduction in penalty units from the "01_Current" run for 180% scaled 1986 and 1997 storm events, sorted by 1997 results smallest to largest

*1.8	1997	1997 %Diff from "01_Current"	1986	1986 %Diff from "01_Current"
09_SBFWYBSWWiden	105,712,357	23.49%	74,759,096	6.84%
08_SBFWYBWiden	106,610,252	22.84%	77,493,823	3.43%
10_FWYBWiden	106,631,010	22.82%	77,515,388	3.40%
07_SBFWWiden	106,830,925	22.68%	77,719,528	3.15%
03_FWWiden	106,851,794	22.66%	77,740,951	3.12%
06_CBAdd	127,671,670	7.59%	75,458,939	5.97%
05_SWWiden	134,890,267	2.37%	76,318,229	4.90%
04_YBWiden	137,933,617	0.16%	80,064,498	0.23%
02_SBWiden	138,138,558	0.02%	80,224,806	0.03%
01_Current	138,160,697	--	80,246,875	--

Table 14. Total penalties and percent reduction in penalty units from the "01_Current" run for 200% scaled 1986 and 1997 storm events, sorted by 1997 results smallest to largest

*2.0	1997	1997 %Diff from "01_Current"	1986	1986 %Diff from "01_Current"
09_SBFWYBSWWiden	146,736,019	21.63%	273,032,807	9.73%
08_SBFWYBWiden	150,320,182	19.71%	277,292,825	8.32%
10_FWYBWiden	150,352,890	19.69%	277,320,308	8.32%
07_SBFWWiden	150,577,829	19.57%	277,543,748	8.24%
03_FWWiden	150,611,033	19.56%	277,570,717	8.23%
06_CBAdd	176,043,416	5.97%	294,406,981	2.67%
05_SWWiden	182,250,708	2.66%	297,295,300	1.71%
04_YBWiden	186,977,455	0.13%	302,219,044	0.08%
02_SBWiden	187,189,928	0.02%	302,443,965	0.01%
01_Current	187,223,068	--	302,471,737	--

After sorting all of the above total penalties, it becomes apparent that the "09_SBFWYBSWWiden" case minimizes penalties the most. The "01_Current" case incurs the most penalties, as expected since all other cases have greater capacity. For the base and the 120% scaled versions of each storm, when the "01_Current" case does not incur the most penalties, the system has not yet reached capacity at all control points. The Sutter Bypass expansion does not create a better solution because there isn't enough water running through the system in the base and 140% scaled storm to reach capacity in that portion of the system. Once the storm is scaled 160% and above, the results all become consistent.

The tables above and Figure 37, for the January 1997 event, show how the expansions don't help much for the base historical storm. As the bigger storms get routed through the system, the damage increases, but the incremental decrease in the penalty units also increases between the scenarios and shows how the expansions improve the system's capabilities.

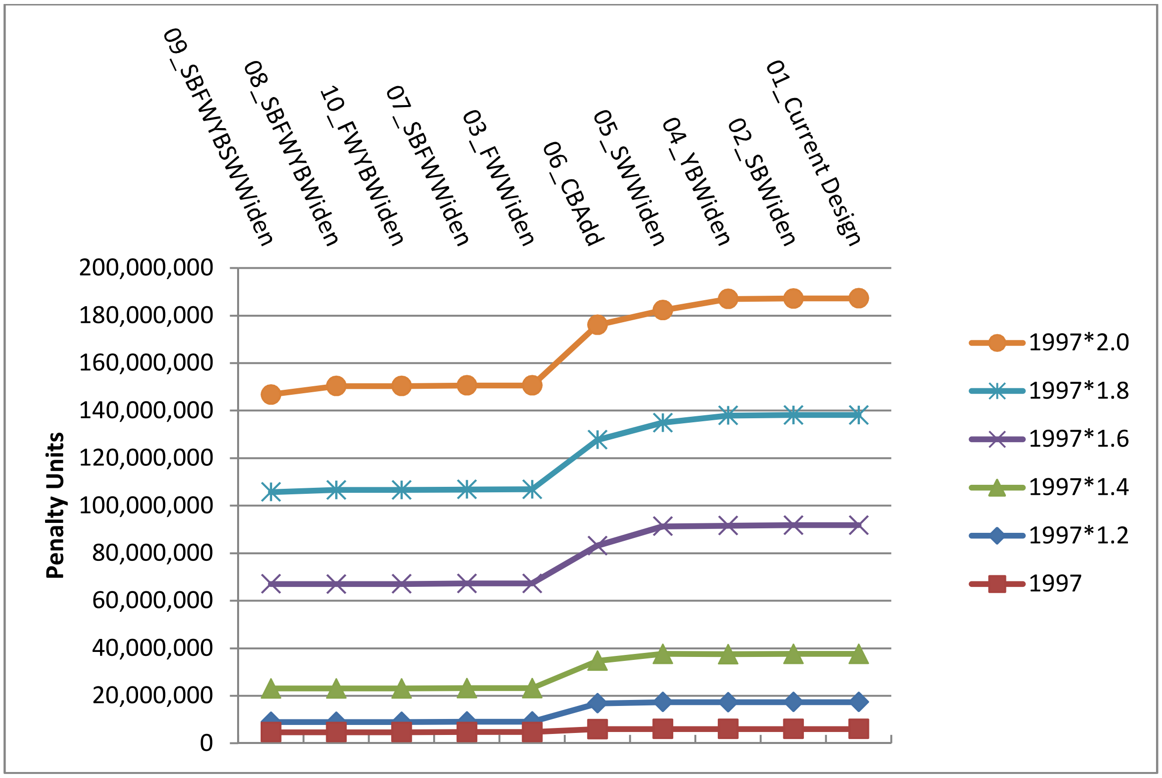


Figure 37. Magnitude of the improvement in the penalties due to different system expansions for the January 1997 event

If the flood management system were to have only one of the examined expansions, the most beneficial expansion would be the Fremont Weir. The Fremont Weir appears to be the major "bottleneck" of the system. As seen from the percent reduction in penalty units, expanding the Fremont Weir creates the largest incremental improvement compared to any other single expansion. Water flows into the Fremont Weir from the Feather/Yuba river system, the Sutter Bypass, and the Sacramento River. As all of this water accumulates at the Fremont Weir, it benefits the system to move these incremental flows into the Yolo bypass as quickly as possible to avoid the damages further downstream on the Sacramento River.

The relatively small percent changes in penalty units as the different expansions are combined do not seem to make it worthwhile to expand other elements of the system. Additionally, expansions in combination with the Fremont Weir expansion provide little incremental benefit. Figure 38 shows how the flows are reduced in the area of the Fremont Weir due to its expansion for the 140% scaled January 1997 event. The Fremont Weir expansion allows more water to be diverted into the Yolo Bypass faster, which helps to alleviate flows down the mainstem of the Sacramento River. Due to this increased diversion, the Sacramento River is kept below channel capacity in the vicinity of communities such as Natomas and West Sacramento, which explains the large decrease in expected damages.

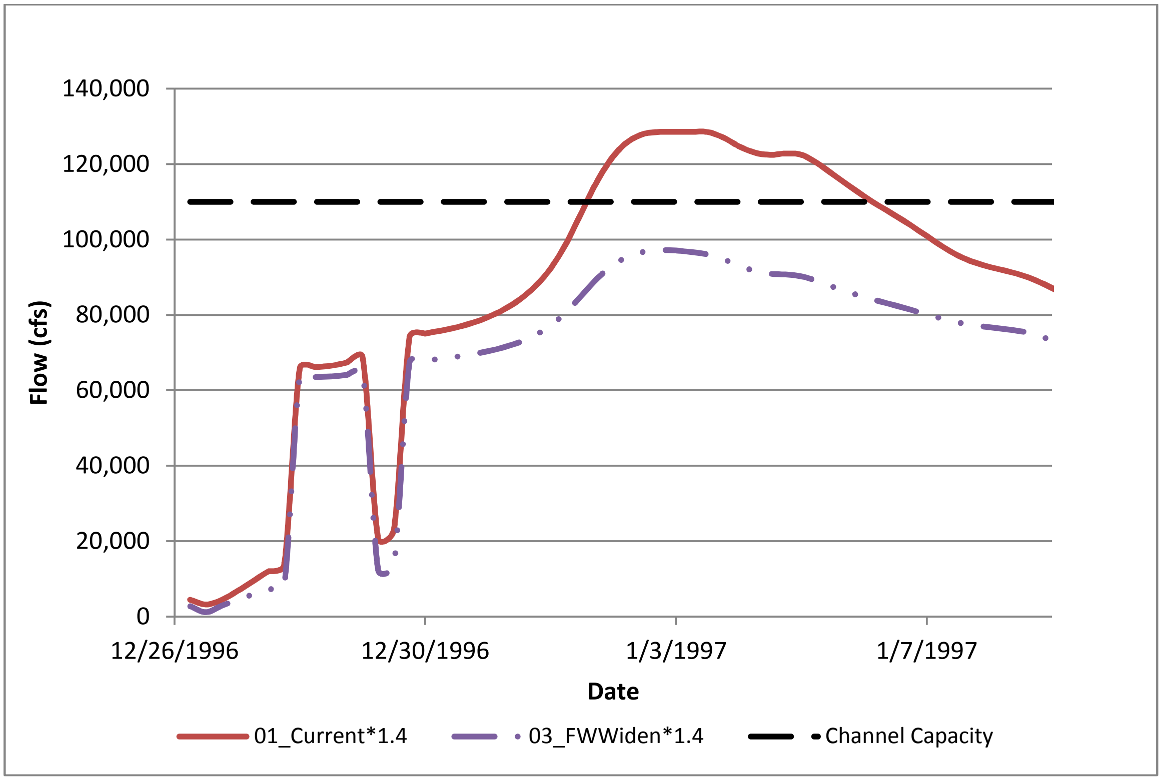


Figure 38. Sacramento River below Fremont Weir for the 140% scaled January 1997 event, as compared to channel capacity

4.3 Expected Annual Damages

To further study the benefits of these different expansions to the Sacramento River Watershed system, a highly simplified probabilistic approach was completed (Lund, 2002). These calculations provide a cursory estimate of expected flood damage reduction benefits of the proposed expansions within the system. Every flood event has a corresponding annual exceedence probability (AEP). The AEPs in Tables 5 and 6 were based on the estimated unregulated flow into each reservoir. To estimate the systemwide AEP for each flood, the regulated frequency curves were obtained from the draft CVHS. The control point used to estimate the systemwide AEP was the Yolo Bypass below Sacramento Bypass (near I-80). This point was chosen because all of the different expansions modeled affect this point and a regulated frequency curve exists as a final draft for this location. The channel capacity at the Yolo Bypass below Sacramento Weir is 480,000 cfs, however, damages can occur before the flow reaches channel capacity. To take that into account, the 1997 storm was also scaled down by 40% and 60% to approximate the 25-year and 10-year return period floods at this location, respectively (David Ford and Mike Imgarten, 2013, personal communication). Using the peak flows for each scaling of the 1997 storm for the "01_Current" model, Table 15 shows the estimated AEPs. Table 15 also shows the estimated annual probability of each storm occurring.

Table 15. Estimated AEPs and expected frequencies for each scaled 1997 storm

1997 Peak Storm Flows and Associated AEPs at Yolo Bypass below Sacramento Weir								
	1997*0.4	**1997*0.6**	**1997**	**1997*1.2**	**1997*1.4**	**1997*1.6**	**1997*1.8**	**1997*2.0**
Peak Flow (cfs)	285,628	396,959	480,000	567,592	684,149	831,670	977,206	1,126,767
AEP for Storm	0.107	0.043	0.018	0.005	0.002	0.001	**AFC	**AFC
Estimated Annual Frequency of Storm Interval (P_i)	0.0640	0.0245	0.0128	0.0033	0.0012	0.0009	~0	~0

**AFC - Above Frequency Curve

The expected value of annual flood damages (EAD) would be the sum of all the damages multiplied by the probability that the storm would occur. The total EAD, should storm i occur, would be:

$$EAD = \sum_i (P_i D_i) \quad (31)$$

where P_i is the probability that storm i would occur and D_i is the amount of damage that storm i creates. The D_i for each expansion and its associated storm were shown in Table 7 and Table 8. Table 16 shows damages calculated for the 40% and 60% down-scaled 1997 storms. The total EAD expected in the Sacramento River Watershed system is shown in Table 17.

Table 16. 40% and 60% scaled 1997 total peak flow damages ($1,000)

Case Runs	1997*0.4 ($1,000)	1997*0.6 ($1,000)
01_Current	192	1,826
02_SBWiden	192	1,826
03_FWWiden	165	1,784
04_YBWiden	192	1,826
05_SWWiden	118	1,732
06_CBAdd	192	1,826
07_SBFWWiden	165	1,784
08_SBFWYBWiden	165	1,784
09_SBFWYBSWWiden	100	1,703
10_FWYBWiden	165	1,784

Table 17. Total EAD in the Sacramento River Watershed system ($1,000)

Case Runs	EAD ($1,000)	EAD Reduction ($1,000)
01_Current	99,921	0
02_SBWiden	99,919	2
03_FWWiden	44,036	55,885
04_YBWiden	99,729	192
05_SWWiden	100,240	-319
06_CBAdd	93,751	6,170
07_SBFWWiden	44,033	55,888
08_SBFWYBWiden	43,603	56,318
09_SBFWYBSWWiden	43,615	56,306
10_FWYBWiden	43,600	56,321

This analysis is only a rough estimate of future expected annual damages in the Sacramento River Watershed intended to illustrate extending the model results into a more risk-based framework. An example illustrative of a more in-depth approach would be to split the system into multiple sub-systems (i.e., Oroville-New Bullards Bar system, Shasta-Black Butte system, and Folsom system), and calculate the estimated expected damages for each sub-system. The reason for this is because the storms aren't the same size in all parts of the Sacramento River Watershed; they have very different frequencies for the same time frame (see Table 5 and Table 6 as examples of how different the return periods are between reservoirs in the system).

However, even with this approximation of expected annual damages for the Sacramento River Watershed, the results help to further show how important the Fremont Weir is in this system. By widening the Fremont Weir alone, the systemwide EAD decreases by a little over $55 million. It is uncertain how much expansion of the Fremont Weir would cost, but further refined estimates of the EAD could show that the flood damage reduction benefits outweigh the overall construction costs.

CHAPTER 5
CONCLUSIONS

5.1 Key Findings

The February 1986 and January 1997 flood events are some of the largest storms that have historically tested the Sacramento River flood management system. Regional flood frequency analyses suggest that larger events can be expected in the future, and climate change has potential to exacerbate the situation. While the precise nature of future storms cannot be predicted, scaling the largest historical events is a common approach that provides a reasonable and understandable level of conservatism for system planning. Both the unadjusted and scaled versions of the historical events were modeled through HEC-ResFloodOpt in this study to evaluate the efficacy of system improvements, in isolation and in aggregate. The hydrologic input data set used for all cases came from the CVHS. Ten cases were represented and ranked by their expected system flood damage reduction benefits:

1. 09_SBFWYBSWWiden – Widening of the Sutter Bypass, Fremont Weir, Yolo Bypass and Sacramento Weir.
2. 08_SBFWYBWiden - Widening of the Sutter Bypass, Fremont Weir, and Yolo Bypass.
3. 10_FWYBWiden - Widening of the Fremont Weir and Yolo Bypass.
4. 07_SBFWWiden - Widening of the Sutter Bypass and Fremont Weir.
5. 03_FWWiden – Widening of the Fremont Weir.
6. 06_CBAdd – Addition of the Cherokee Bypass.
7. 05_SWWiden – Widening of the Sacramento Weir.
8. 04_YBWiden – Widening of the Yolo Bypass.
9. 02_SBWiden – Widening of the Sutter Bypass.
10. 01_Current – No changes to the current system.

This ranking above is only based on expected system flood damage reduction benefits under the 60% and larger upward-scaled 1997 flood events; no study of estimated costs was performed for this study. The resulting net benefits would likely result in a significant re-ranking of the above alternatives, with “03_FWWiden” potentially ranking as the preferred alternative.

The major finding from this analysis is that the Fremont Weir is the major operational bottleneck of the system, and that its expansion has the potential to greatly reduce future flood damages. In hindsight, this conclusion is highly intuitive. The Fremont Weir is at the junction of three primary system features (Sacramento River, Feather River and Sutter Bypass) and represents a first line of defense against flood damages in the greater Sacramento region. The Fremont Weir is uniquely capable of maximizing flood releases into the Yolo Bypass, a system component that carries a much lower marginal damage potential when compared to the mainstem Sacramento River channel.

Conversely, this study found that expansion of the Sutter Bypass has little flood damage reduction potential when performed in isolation. The Sutter Bypass appears to be much more appropriately sized for its contributory watersheds when contrasted with other system flood bypasses. The next most intriguing expansion option beyond the Fremont Weir is the addition of the Cherokee Bypass. The Cherokee Bypass diverts water from the Feather-Yuba system into the Sutter Bypass; because this bypass often has spare capacity, this system improvement creates a modest flood damage reduction opportunity in the Yuba City/Marysville region.

5.2 Impact of Findings and Areas for Further Study

This study and its findings should be weighed against the broad, simplified assumptions inherent in any large system optimization. It is now up to local, state, and federal agencies with flood control responsibilities to carry these preliminary findings forward and develop a more refined proof of concept. As one example, detailed simulation models could be created for each of the most interesting expansions identified in this study, based on the most current understanding of hydrologic, physical and operational system characteristics. The application of HEC-ResSim with results from HEC-ResFloodOpt is an example of such a study. It will give agencies a better idea of how expansions might affect the reservoir operations individually and altogether.

Another area for further study is an analysis of economics associated with both system damage potential and expansion costs. This study includes assumptions of flood damage potential that have not been refined in several years. The CVFPP is developing estimates for each part of this system.

5.3 Findings and Recommendations Related to HEC-ResFloodOpt

Several modeling software limitations were identified in the course of this study. Most notable of these short-comings is the apparent lateral weir calculation instabilities in the latest build of HEC-ResFloodOpt. Further studies of the Sacramento River flood control system using this software should at least take this flaw into consideration as it generally will require additional troubleshooting. Resolution of this calculation instability would increase confidence in the future use of HEC-ResFloodOpt. However, because all 10 cases studied in this thesis had similar weir instabilities between them, the findings relative to one another are applicable for drawing preliminary conclusions.

Another software limitation is that its compatibility is generally limited to Windows XP or earlier operating systems. When tested as part of this study, HEC-ResFloodOpt failed to run on a computer running Windows 7. To ensure the future relevance of this software, HEC-ResFloodOpt should be updated to provide compatibility with popular, recently developed operating systems. There is also a relatively restrictive limit to the number of decision variables and constraints that the solver within the optimization software can handle. At the beginning of this study, it was anticipated that this application of HEC-ResFloodOpt could be run with 1-hour time steps; it became apparent that running this model for 14 days at such a fine time step created more decision variables than the solver could accommodate. Expansion of the solver's decision variable capacity would be a straight-forward and valuable improvement.

The addition of Lagrange multipliers (shadow prices) as part of the default model output would be another potential future improvement of the software. Adding this capability to the software would increase the modeler's efficiency in finding important constraints in the system as they relate to impacts on the fundamental objective function. As an example, the marginal benefit of expansions to the Fremont Weir would have been immediately apparent when evaluating the shadow prices for the "01_Current" case under the 140% scaled 1997 storm.

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APPENDIX A OPTIMIZATION MODEL INPUT

Section A.1 shows the ASCII text file that was the input to the HEC-ResFloodOpt Software. For definitions of what each card means, see Appendix A of Dustin D. Jones' 1999 thesis. Section A.2 through Section A.6 show the changes that were made for each expansion of the model.

A.1 Full Model Input

```
T1 Sacramento Basin Model for 6 hr time periods (also works as HEC-5
T2 when S$, P$, LQ, L$, SO  cards are commented out)
T3 By: Christy Jones, Last edited 3/05/2013
C  This is the model originally created by Dustin Jones in 1999. It has
C  been updated to include current capacities in the Sacramento River
C  watershed system. The reservoir storage-outflow relationships have also been updated
C  to allow bigger storms to pass through the system. The diversion curves have been
C  extended for the same reason.
J1             1       6       2       4       1                               3
J3                                                                             2
C  ------------------------------------------------------------------
C                             Stony Creek
C  ------------------------------------------------------------------
C  =====   Black Butte Dam, Stony Creek   =====
C  (Operating levels and S-O from manual, SPK)
C  Level 1: Match point
C        2: Top of Gross Pool - 473.5'
C        3: Match point
C        4: Top of Std Proj Flood Pool - 483.1'
C        5: Spillway Design Pool - 509.8'
C        6: Top of Dam - 515'
C 1997 Reservoir storage curve with starting storage:
RL     2   35800   35000  143676  170000  223000  354000  389000
C 1995:
C RL     2   59000   35000  143676  170000  190100  354000  389000
C 1986:
C RL     2   43100   35000  143676  170000  223000  354000  389000
S$                 -0.05    0.01     0.5       1       2       3
RS     6   35000  143676  170000  223000  354000  389000
C  The first RQ curve forms a concave function - This curve was used by Dustin
C  because it was less computationally complicated. To pass higher flows
C  such as the 1997 scaled by 100%, it is necessary to use the convex function.
C RQ     6   16000   23000   24600   25800   35500   37500
C  These discharges form a convex function
RQ     6   16000   23000   24600   25000  103600  121600
C  Release change taken from BLB flood control diagram in Water Control Manual
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 - 1016'
C        3: Match point
C        4: Gross Pool - 1067'
C        5: Match point
C        6: Top of Dam - 1077.5'
C 1997:
RL     4 3333000 3200000 3250900 3900000 4552000 4750000 4850000
C 1995:
C RL     4 3480000 3200000 3250900 3900000 4552000 4750000 4850000
C 1986:
C RL     4 3393900 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
C Original
C QS     2  110000  500000
C QD     2       0  325000
C Extended convex function diversion to allow greater flow to pass
C QS     2  110000  650000
C QD     2       0  450000
C Extended convex function from HEC-ResSim
QS     2   90000  500000
QD     2       0  340000
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
LQ135000  279900  285600
L$   0.0    0.01    0.02    0.03
MQ135000  279900  285600
M$   0.0    0.01    4.78    4.79
RT    14      16     1.9
CR     1       1
DR    14      24     2.2     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
C Original
C QS     2   60000  200000
C QD     2       0   55200
C Extended convex function diversion to allow greater flow to pass
QS     2   60000  288260
QD     2       0   90000
C QD     2       0       0
C
C  =====   Colusa Weir, Sacramento River   =====
C  (Cottonwood Study, Russ SPK)
CP    16                                                                      9
ID Colusa Weir
LQ 65000   68100   69500
L$   0.0    0.01    0.02    0.03
MQ 65000   68100   69500
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 (HEC-ResSim has same curve).
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
C QS     2   23300   47000
C QD     2       0   18390
C Extended convex function diversion to allow greater flow to pass
C QS     2   23300   55520
C QD     2       0   25000
C Extended convex function from HEC-ResSim
QS     2   23500   96000
QD     2       0   65000
C QD     2       0       0
C
C  =====   Butte Slough Nr Meridian, Sutter Bypass   =====
CP    24
ID Meridian
```

```
LQ178000  634800  647800
L$  0.00    0.01    0.02    0.03
MQ178000  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
LQ216500  380000  385000
L$  0.00    0.01    0.02    0.03
MQ216500  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 For 1997 Curve:
C  Level 1: Match point
C        2: Top of Conservation - 1918.3'
C        3: Gross Pool - 1956'
C        4: Spillway Design Flood Pool - 1962.5'
C        5: Top of Dam - 1965'
C        6: Top of Parapet Wall - 1967.7'
C
C For 1995/86 Curve:
C  Level 1: Match point
C        2: Top of Conservation - 1918.3'
C       3: Match point
C        3: Gross Pool - 1956'
C        4: Spillway Design Flood Pool - 1962.5'
C        5: Top of Dam - 1965'
C
C 1997:
RL    28  794600  640000  790000  960000  998000 1010000 1020000
C RL    28  794600  640000  790000  900000  960000  998000 1010000
C 1995:
C RL    28  743592  640000  790000  900000  960000  998000 1010000
C 1986:
C RL    28  649700  640000  790000  900000  960000  998000 1010000
S$                 -0.02   -0.01     0.1     0.3     2.0       3
C For 1997 scaled storms
RS     6  640000  790000  960000  998000 1010000 1020000
RQ     6    3000    7000  127500  154300  162700  169000
C For 1995/86 scaled storms
C RS     6  640000  790000  900000  960000  998000 1010000
C RQ     6    3000    7000   85000  127000  153000  161000
C RS     6  640000  790000  900000  960000  998000 1010000
C RQ     6    3000   22100   95000  130000  152100  159000
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 - 834.1'
C        2: Top of Conservation - 848.5'
C        3: Match point - 884.5'
C        4: Gross Pool - 900'
C        5: Spillway Design Pool - 916.2'
C        6: Top of Dam - 922'
C 1997:
RL    32 2681250 2600000 2788000 3300000 3538000 3814000 3870000
C 1995:
C RL    32 2746100 2600000 2788300 3300000 3538000 3814000 3870000
C 1986:
C RL    32 2598095 2600000 2788300 3300000 3538000 3814000 3870000
S$                  -0.2    -0.1    0.05     0.5     2.0       3
RS     6 2600000 2788000 3300000 3537600 3814000 3870000
C  The first RQ curve forms a concave function - This curve was used by Dustin
C  because it was less computationally complicated. To pass higher flows
C  such as the 1997 scaled by 100%, it is necessary to use the convex function.
C RQ     6   40000   90000  220000  262000  310650  320500
C  These discharges form a convex function
RQ     6   40000   90000  220000  260000  650000  729000
R2  5000    2500
P$     1       1
CP    32                                                                      3
ID Oroville Dam
RT    32      34     1.2     0.2       8
C
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 Original from Dustin, Revised 4/16/99:
C QS     2   61000  460000
C QD     2       0  355000
C Revised by CJones, 3/09/2013 to extend the weir curve for higher flows:
QS     2   61000 1200000
QD     2       0 1013396
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:
RL    42  486000  440000  486000  610000 1010000 1130000 1300000
C 1995:
C RL    42  559600  440000  486000  610000 1010000 1130000 1300000
C 1986:
C RL    42  712500  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     1.2     0.2       6               -1
C 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
C QS     5   75000  170000  190000  210000  221600
C QD     5       0   96000  111000  121000  123000
C Revised by CJones, 3/09/2013 to extend the weir curve for higher flows:
C QS     6   75000  170000  190000  210000  221600  630000
C QD     6       0   96000  111000  121000  123000  193000
C Simplified the extended curve to try and smooth the diversion flow:
QS     3   60220  221600  630000
QD     3       0  123000  193000
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     4
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.
C BF     2      60                95030806               6                      12
C BF     2      64                96122600               6
BF     2      60                96122606               6                      12
C BF     2      60                86021006               6                      12
C
C  ******   INFLOW RECORDS   ******
ZR=IN2   A=STONY CR     B=BLACK BUTTE       C=FLOW-RES IN   E=6HOUR  F=CVHS
ZR=IN4   A=SACRAMENTO   B=SHASTA            C=FLOW-RES IN   E=6HOUR  F=CVHS
ZR=IN6   A=SACRAMENTO   B=BEND BRIDGE       C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN8   A=SACRAMENTO   B=VINA-WOODSON BR   C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN10  A=SACRAMENTO   B=AT ORD FERRY      C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN24  A=BUTTE SLOUGH B=NR MERIDIAN       C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN26  A=SUTTER BYPSS B=RD 1500           C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN28  A=NORTH YUBA   B=NEW BULLARDS BAR  C=FLOW-RES IN   E=6HOUR  F=CVHS
ZR=IN30  A=YUBA         B=NR MARYSVILLE     C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN32  A=FEATHER      B=OROVILLE          C=FLOW-RES IN   E=6HOUR  F=CVHS
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=CVHS
ZR=IN38  A=FEATHER      B=AT NICOLAUS       C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN40  A=SACRAMENTO   B=FREMONT_VERONA    C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN42  A=AMERICAN     B=FOLSOM            C=FLOW-RES IN   E=6HOUR  F=CVHS
ZR=IN48  A=SACRAMENTO   B=SACRAMENTO WEIR   C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN50  A=YOLO BYPASS  B=COLUSA DRAIN      C=FLOW-INC      E=6HOUR  F=CVHS
ZR=IN52  A=YOLO BYPASS  B=NR WOODLAND       C=FLOW-INC      E=6HOUR  F=CVHS
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=97_01_CURRENT
EJ
ER
```

A.2 Sutter Bypass Expansion

```
**********************************************************
NEW
**********************************************************
C  =====   Butte Slough Nr Meridian, Sutter Bypass   =====
CP    24
ID Meridian
LQ222500  679300  692300
L$  0.00    0.01    0.02    0.03
MQ222500  679300  692300
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
LQ261000  424500  429500
L$  0.00    0.01    0.02    0.03
MQ261000  424500  429500
M$  0.00    0.01    0.02    0.03
RT    26      40     1.2     0.2       4
C
**********************************************************


**********************************************************
ORIGINAL
**********************************************************
C  =====   Butte Slough Nr Meridian, Sutter Bypass   =====
CP    24
ID Meridian
LQ178000  634800  647800
L$  0.00    0.01    0.02    0.03
MQ178000  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
LQ216500  380000  385000
L$  0.00    0.01    0.02    0.03
MQ216500  380000  385000
M$  0.00    0.01    0.02    0.03
RT    26      40     1.2     0.2       4
C
**********************************************************
```

A.3 Fremont Weir Expansion

```
************************************************************
NEW
************************************************************
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 Original from Dustin, Revised 4/16/99:
C QS     2   61000  460000
C QD     2       0  355000
C Revised by CJones, 3/09/2013 to extend the weir curve for higher flows:
QS     2   61000 1200000
QD     2       0 1076198
C QD     2       0       0
C
************************************************************


************************************************************
ORIGINAL
************************************************************
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 Original from Dustin, Revised 4/16/99:
C QS     2   61000  460000
C QD     2       0  355000
C Revised by CJones, 3/09/2013 to extend the weir curve for higher flows:
QS     2   61000 1200000
QD     2       0 1013396
C QD     2       0       0
C
************************************************************
```

A.4 Yolo Bypass Expansion

```
************************************************************
NEW
************************************************************
C  -----------------------------------------------------------------
C                              Yolo Bypass
C  -----------------------------------------------------------------
C  =====   Colusa Drain, Yolo Bypass   =====
CP    50
ID Colusa Drain
LQ383000  520000  525000
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
LQ417000  613900  625600
L$  0.00    0.01    0.02    0.03
MQ417000  613900  625600
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
LQ520000  613900  625600
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
LQ530000  812800  828600
L$  0.00    0.02    0.03    0.04
MQ530000  812800  828600
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
LQ600000  608400  620000
L$  0.00    0.02    0.03    0.04
MQ600000  608400  620000
M$  0.00    0.02    0.44     0.5
RT    58       0
C
************************************************************
```

```
************************************************************
ORIGINAL
************************************************************
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
************************************************************
```

A.5 Sacramento Weir/Bypass Expansion

```
***********************************************************
NEW
***********************************************************
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     1.2     0.2       6              -1
C 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
C QS     5   75000  170000  190000  210000  221600
C QD     5       0   96000  111000  121000  123000
C Revised by CJones, 3/09/2013 to extend the weir curve for higher flows:
C QS     6   75000  170000  190000  210000  221600  630000
C QD     6       0   96500  113000  128000  134800  374000
C Simplified the extended curve to try and smooth the diversion flow:
QS     3   60220  221600  630000
QD     3       0  134800  374000
C
***********************************************************


***********************************************************
ORIGINAL
***********************************************************
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     1.2     0.2       6              -1
C 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
C QS     5   75000  170000  190000  210000  221600
C QD     5       0   96000  111000  121000  123000
C Revised by CJones, 3/09/2013 to extend the weir curve for higher flows:
C QS     6   75000  170000  190000  210000  221600  630000
C QD     6       0   96000  111000  121000  123000  193000
C Simplified the extended curve to try and smooth the diversion flow:
QS     3   60220  221600  630000
QD     3       0  123000  193000
C
***********************************************************
```

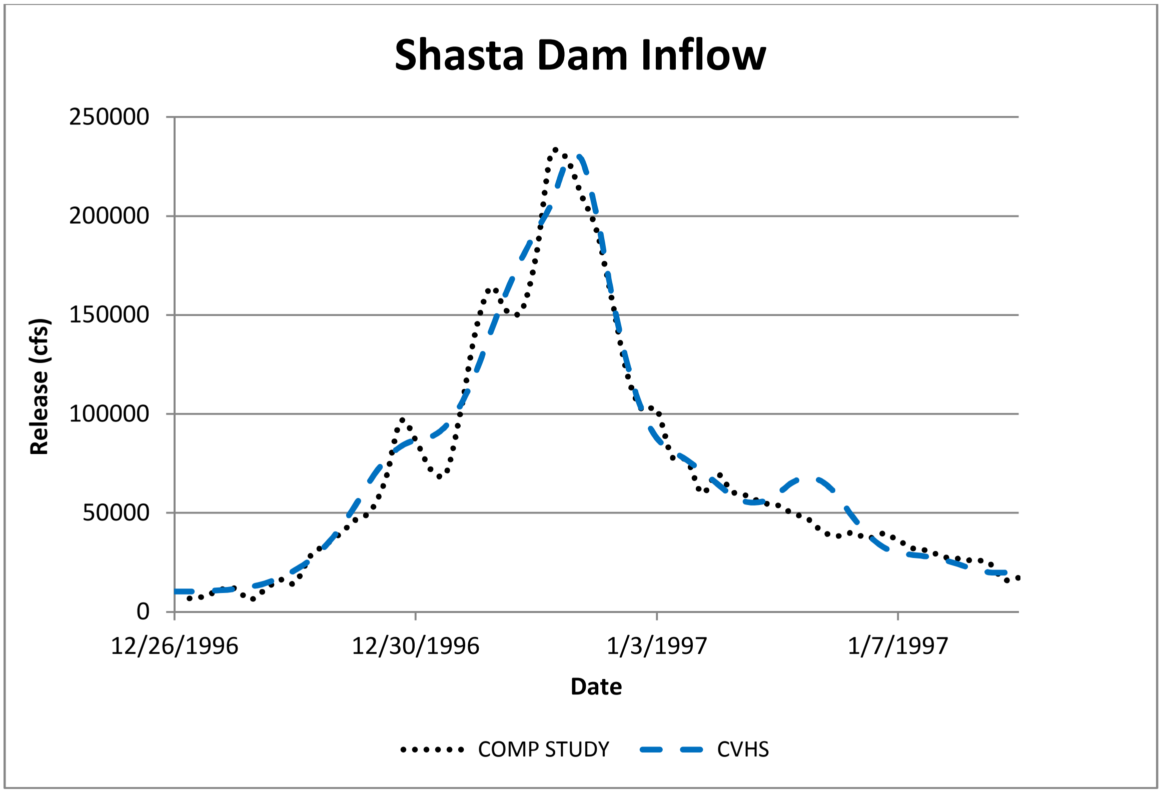
A.6 Cherokee Bypass Expansion

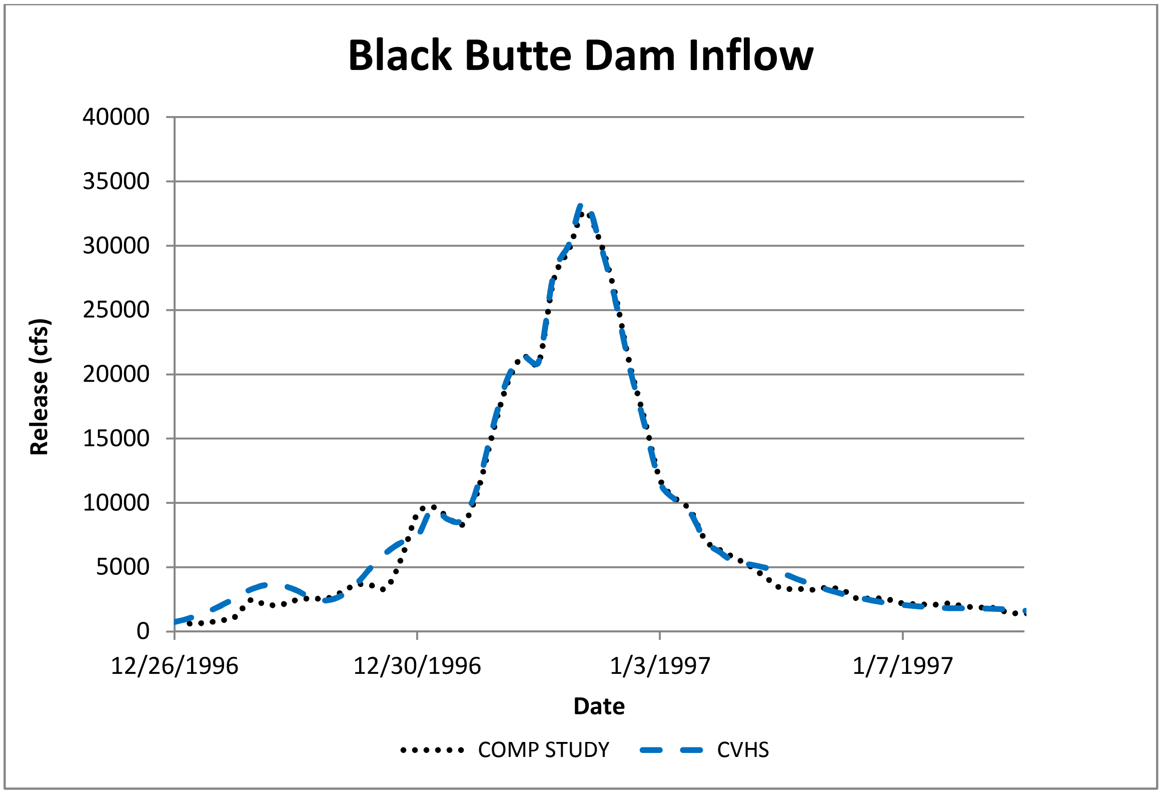
```
************************************************************
NEW
************************************************************
C  ---------------------------------------------------------------------
C                            Feather River
C  ---------------------------------------------------------------------
C  =====   Oroville Dam, Feather River   =====
C  (Operating levels and S-O from manual, SPK)
C  Level 1: Match Point - 834.1'
C        2: Top of Conservation - 848.5'
C        3: Match point - 884.5'
C        4: Gross Pool - 900'
C        5: Spillway Design Pool - 916.2'
C        6: Top of Dam - 922'
C 1997:
C RL    32 2681250 2600000 2788000 3300000 3538000 3814000 3870000
C 1995:
C RL    32 2746100 2600000 2788300 3300000 3538000 3814000 3870000
C 1986:
RL    32 2598095 2600000 2788300 3300000 3538000 3814000 3870000
S$                  -0.2    -0.1    0.05     0.5     2.0       3
RS     6 2600000 2788000 3300000 3537600 3814000 3870000
C  The first RQ curve forms a concave function - This curve was used by Dustin
C  because it was less computationally complicated. To pass higher flows
C  such as the 1997 scaled by 100%, it is necessary to use the convex function.
C RQ     6   40000   90000  220000  262000  310650  320500
C  These discharges form a convex function
RQ     6   40000   90000  220000  260000  650000  729000
R2  5000    2500
P$     1       1
CP    32
ID Oroville Dam
RT    32      33     1.9
CR     1       1
C
C  =====   Cherokee Bypass Canal   =====
CP    33
ID Cherokee Bypass
C LQ 15150  150000  258900
C L$  -100    0.00     0.5     1.0
C MQ 15150  150000  258900
C M$  -100    0.00     0.1    7.21
RT    33      34     1.2     0.2       8
DR    33      24     1.9
CR     1       1
QS     2  150000  250000
QD     2       0   32000
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
************************************************************
```

```
*************************************************************
ORIGINAL
*************************************************************
C  -----------------------------------------------------------------
C                             Feather River
C  -----------------------------------------------------------------
C  =====   Oroville Dam, Feather River   =====
C  (Operating levels and S-O from manual, SPK)
C  Level 1: Match Point - 834.1'
C        2: Top of Conservation - 848.5'
C        3: Match point - 884.5'
C        4: Gross Pool - 900'
C        5: Spillway Design Pool - 916.2'
C        6: Top of Dam - 922'
C 1997:
C RL    32 2681250 2600000 2788000 3300000 3538000 3814000 3870000
C 1995:
C RL    32 2746100 2600000 2788300 3300000 3538000 3814000 3870000
C 1986:
RL    32 2598095 2600000 2788300 3300000 3538000 3814000 3870000
S$                 -0.2    -0.1    0.05     0.5     2.0       3
RS     6 2600000 2788000 3300000 3537600 3814000 3870000
C  The first RQ curve forms a concave function - This curve was used by Dustin
C  because it was less computationally complicated. To pass higher flows
C  such as the 1997 scaled by 100%, it is necessary to use the convex function.
C RQ     6   40000   90000  220000  262000  310650  320500
C  These discharges form a convex function
RQ     6   40000   90000  220000  260000  650000  729000
R2  5000    2500
P$     1       1
CP    32                                                                     3
ID Oroville Dam
RT    32      34     1.2     0.2       8
C
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
*************************************************************
```

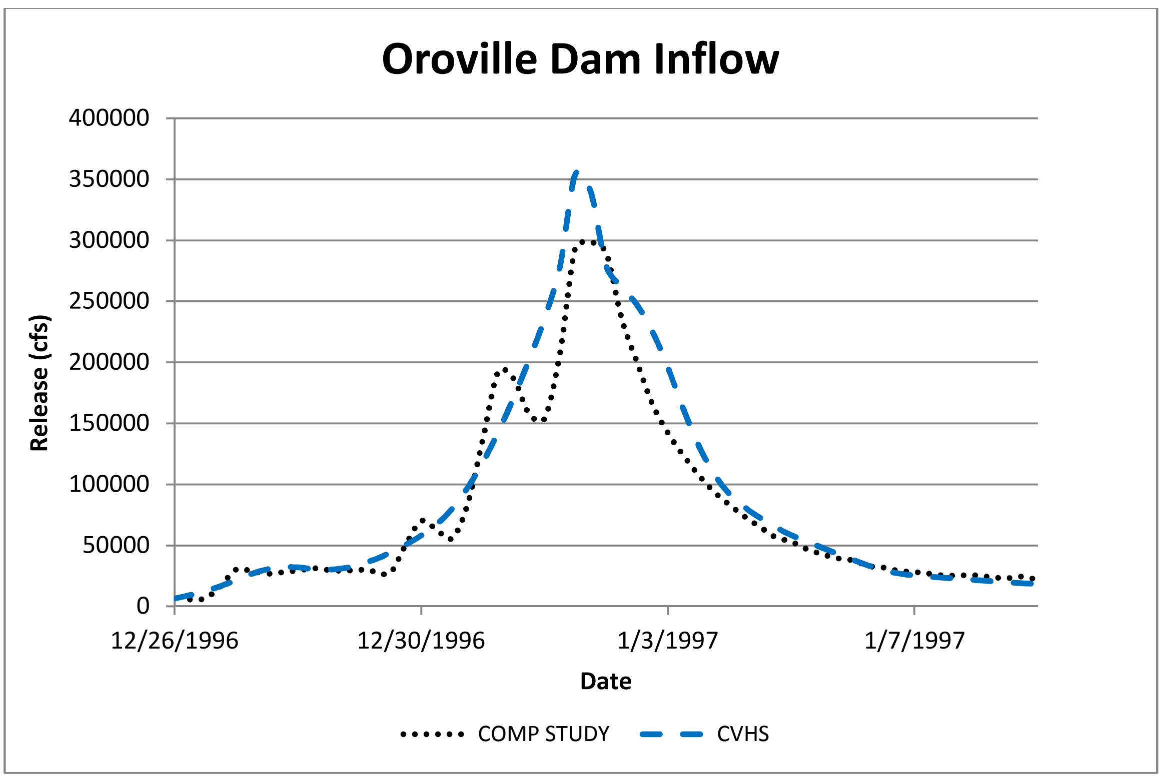
APPENDIX B DIFFERENCE IN CVHS AND COMP STUDY FLOW INPUT

The following figures show the inflow to the Sacramento River Watershed optimization models for all of the reservoirs and the local inflow points in the system.

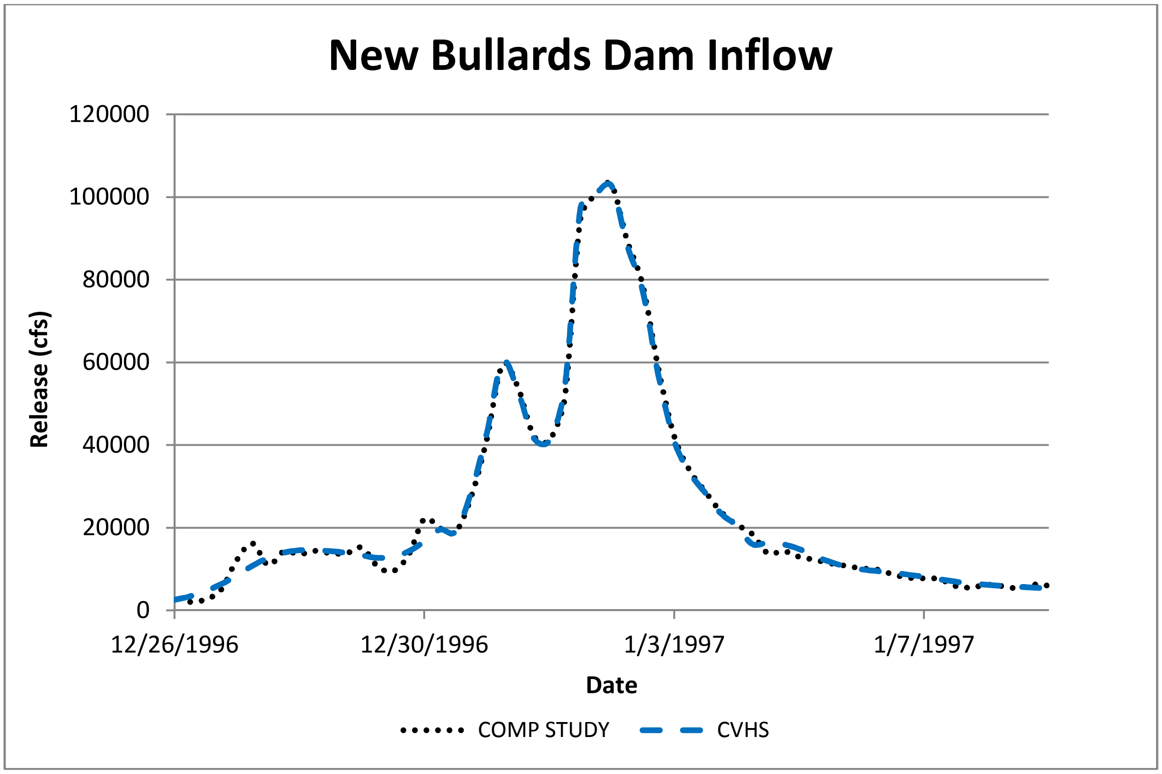




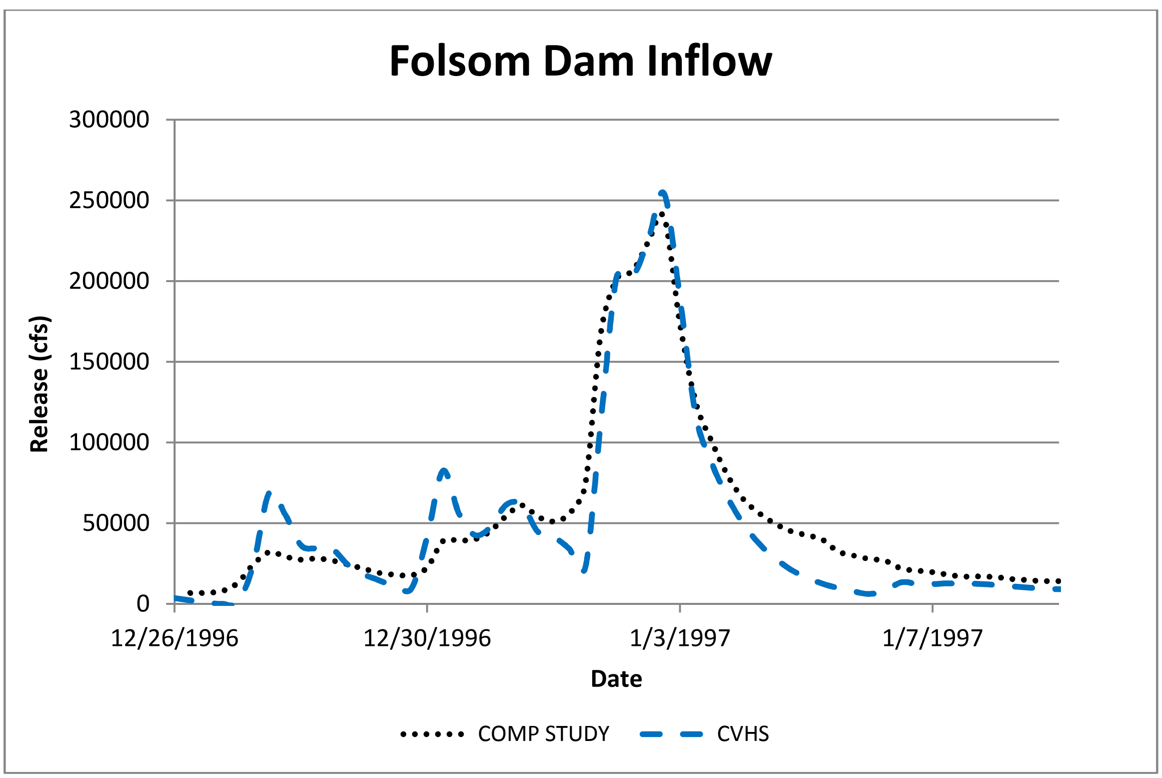
Black Butte Dam Inflow
40000
35000
30000
25000
20000
15000
10000
5000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS



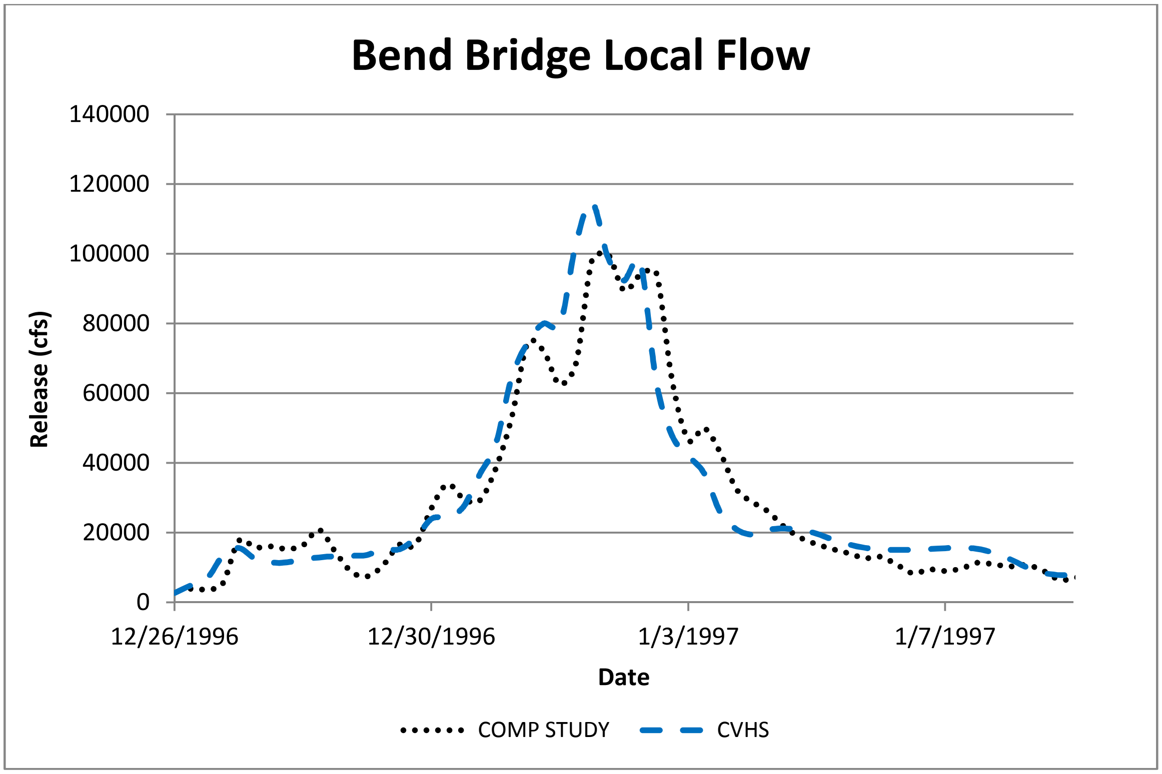
Oroville Dam Inflow
400000
350000
300000
250000
200000
150000
100000
50000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS

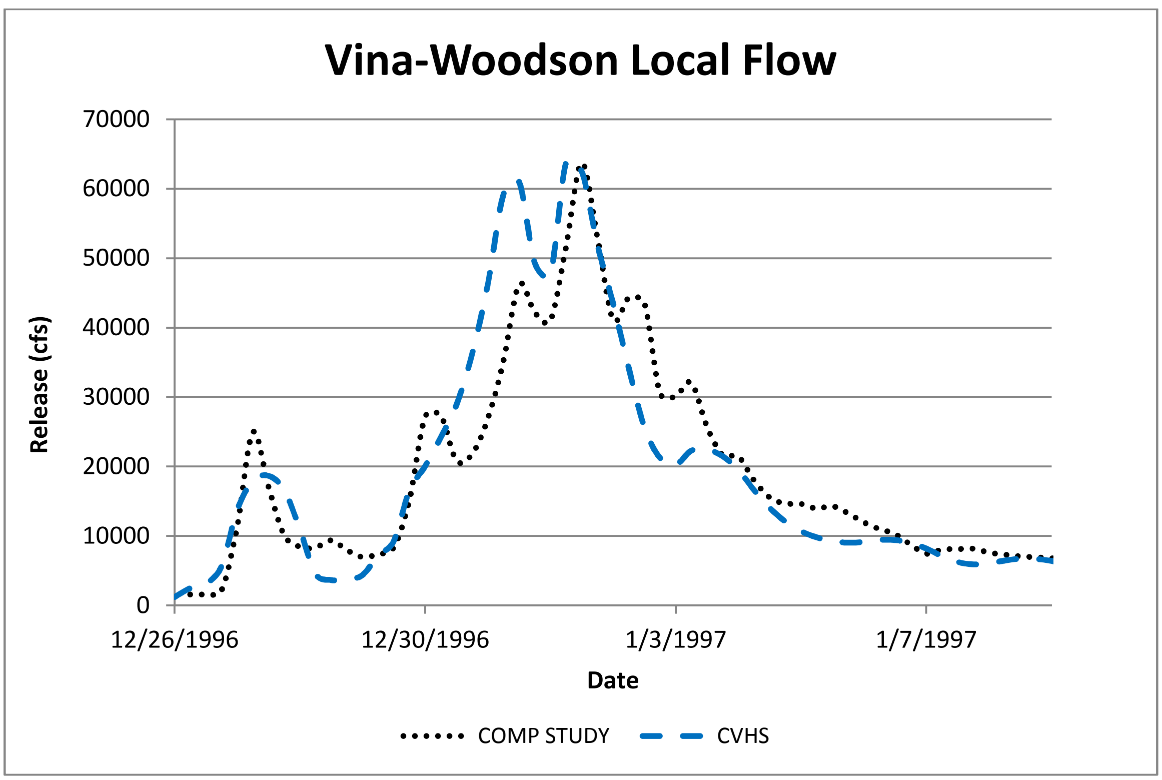


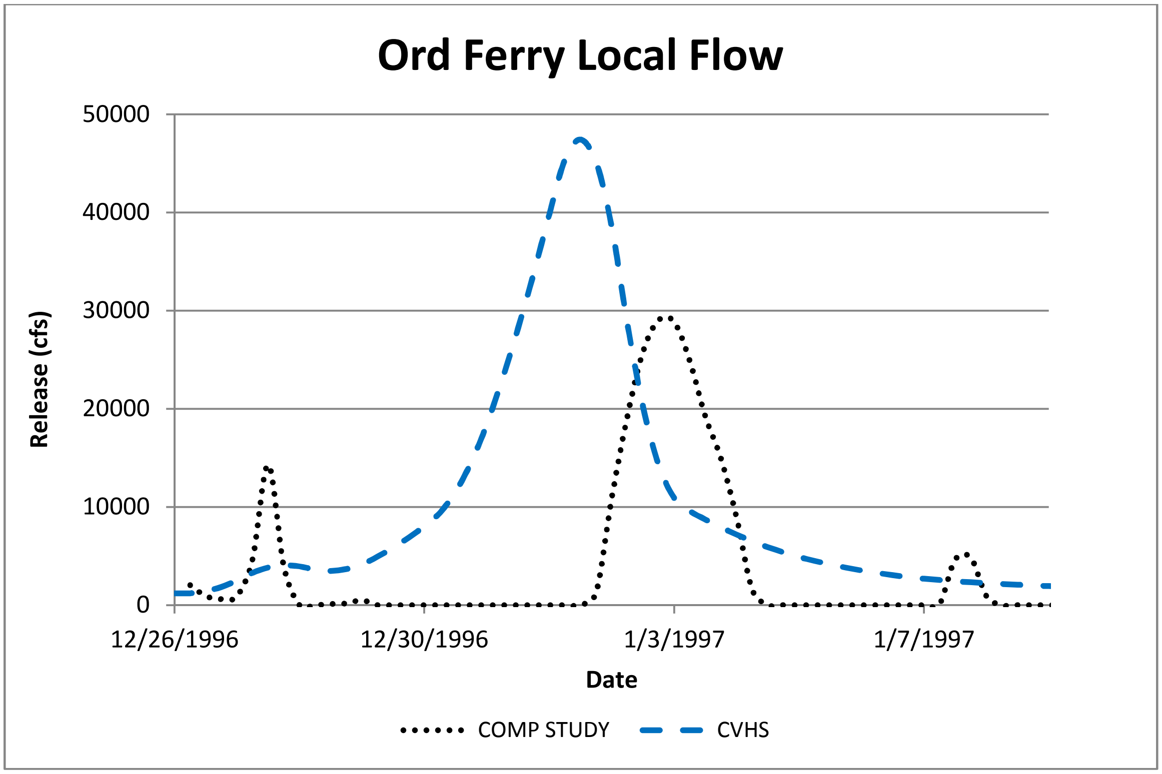
New Bullards Dam Inflow
Release (cfs)
120000
100000
80000
60000
40000
20000
0
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS

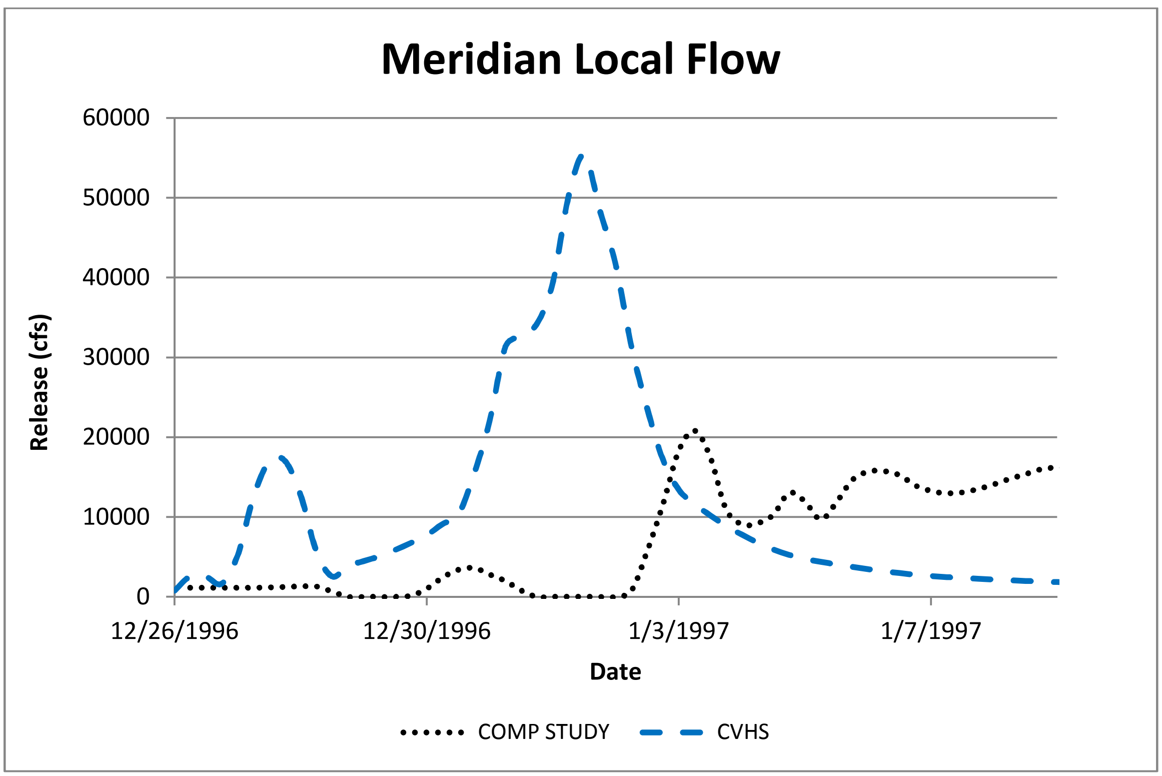


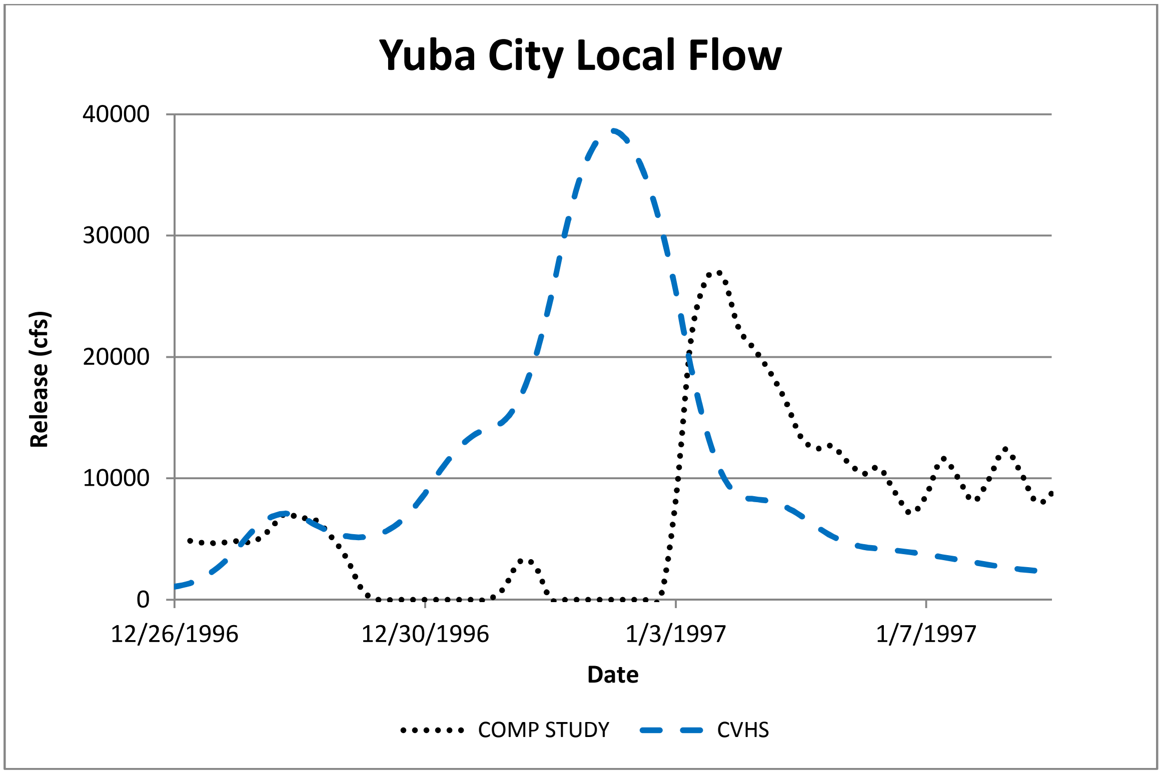
Folsom Dam Inflow
Release (cfs)
300000
250000
200000
150000
100000
50000
0
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS

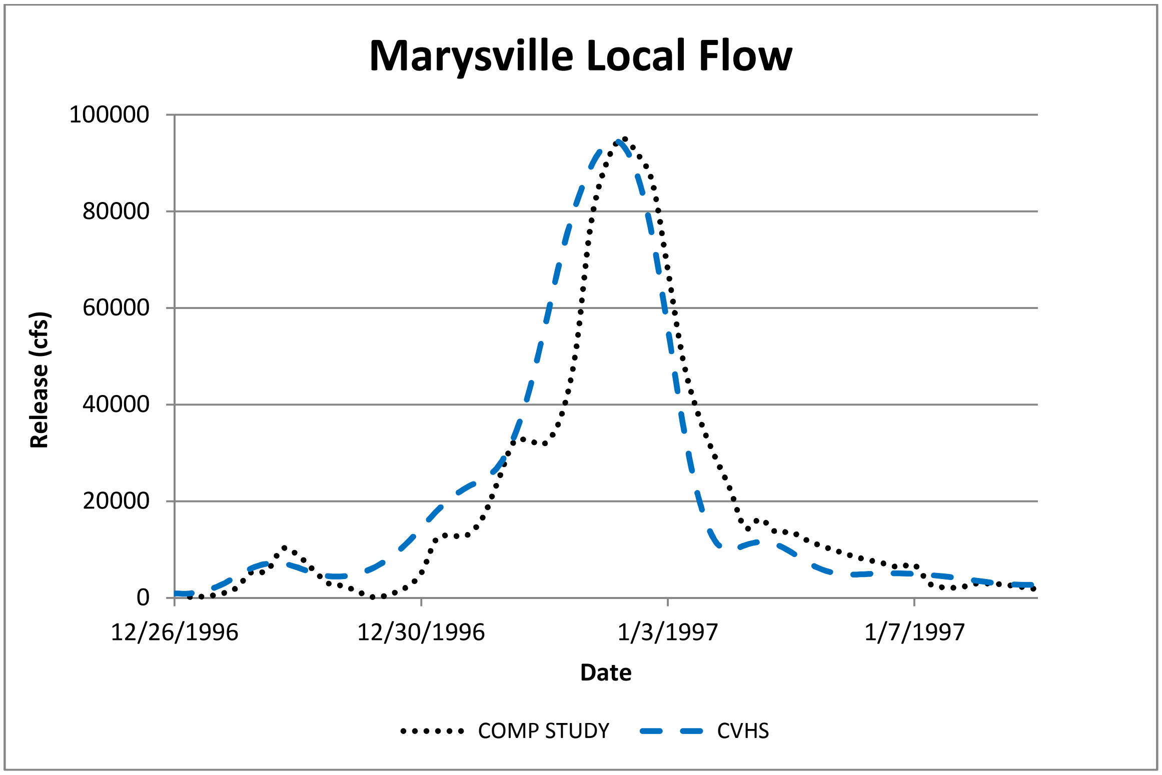

Bend Bridge Local Flow
140000
120000
100000
80000
60000
40000
20000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS

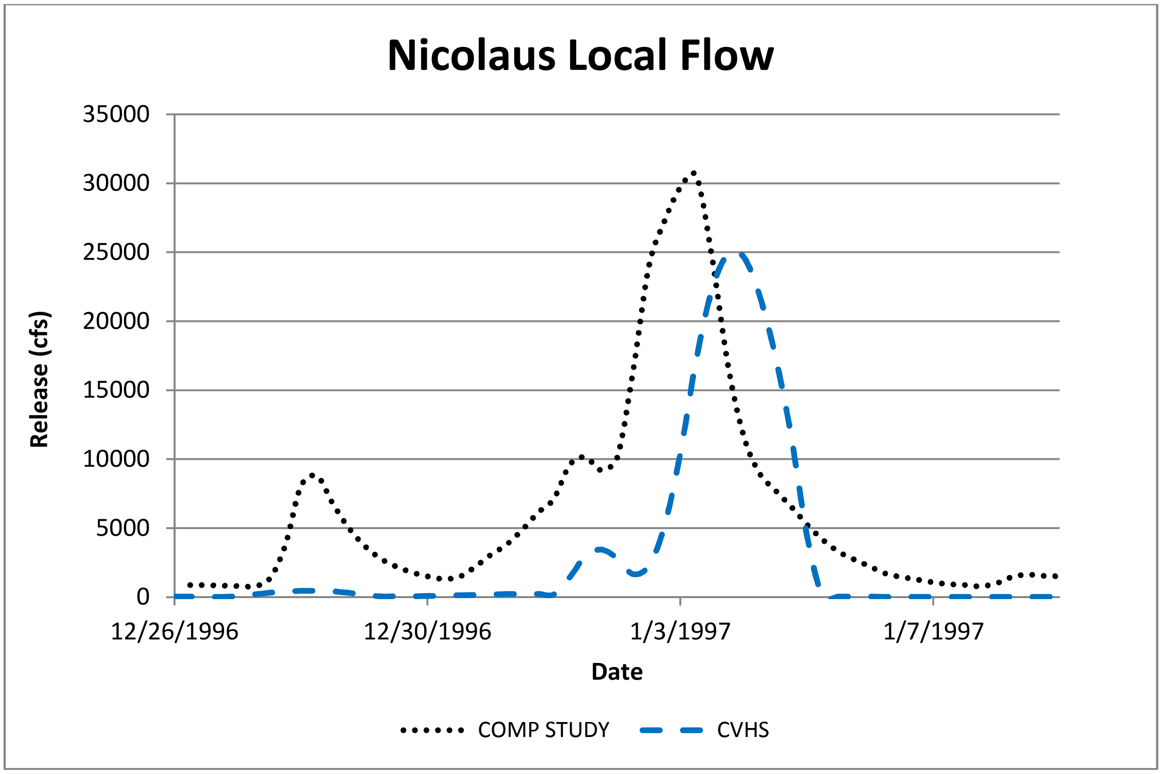

Vina-Woodson Local Flow
70000
60000
50000
40000
30000
20000
10000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS

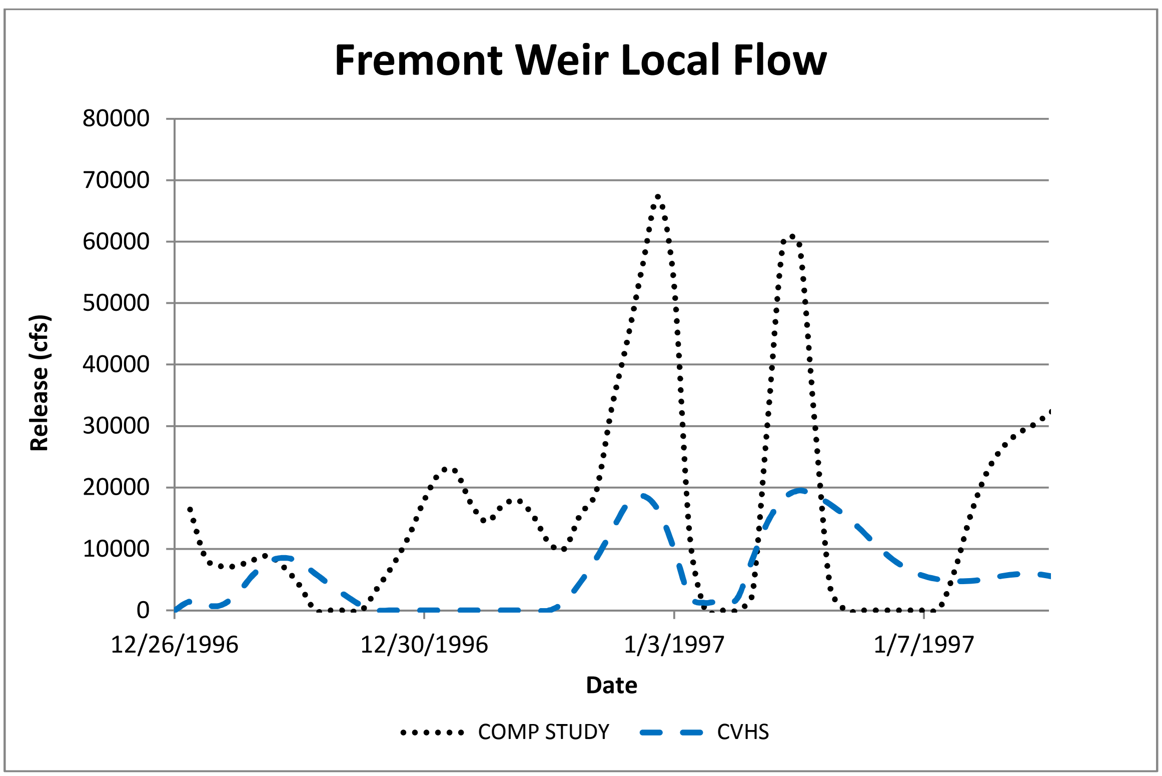

Ord Ferry Local Flow
50000
40000
30000
20000
10000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS

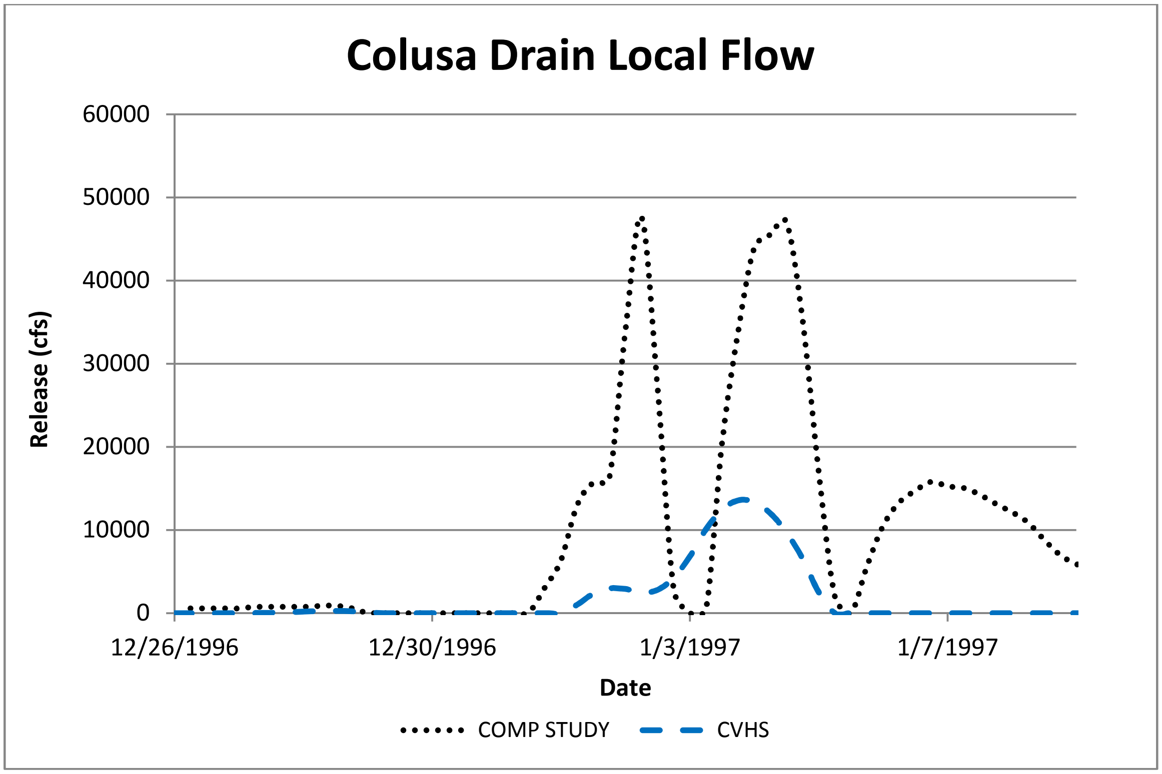

Meridian Local Flow
60000
50000
40000
30000
20000
10000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS


Yuba City Local Flow
40000
30000
20000
10000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS

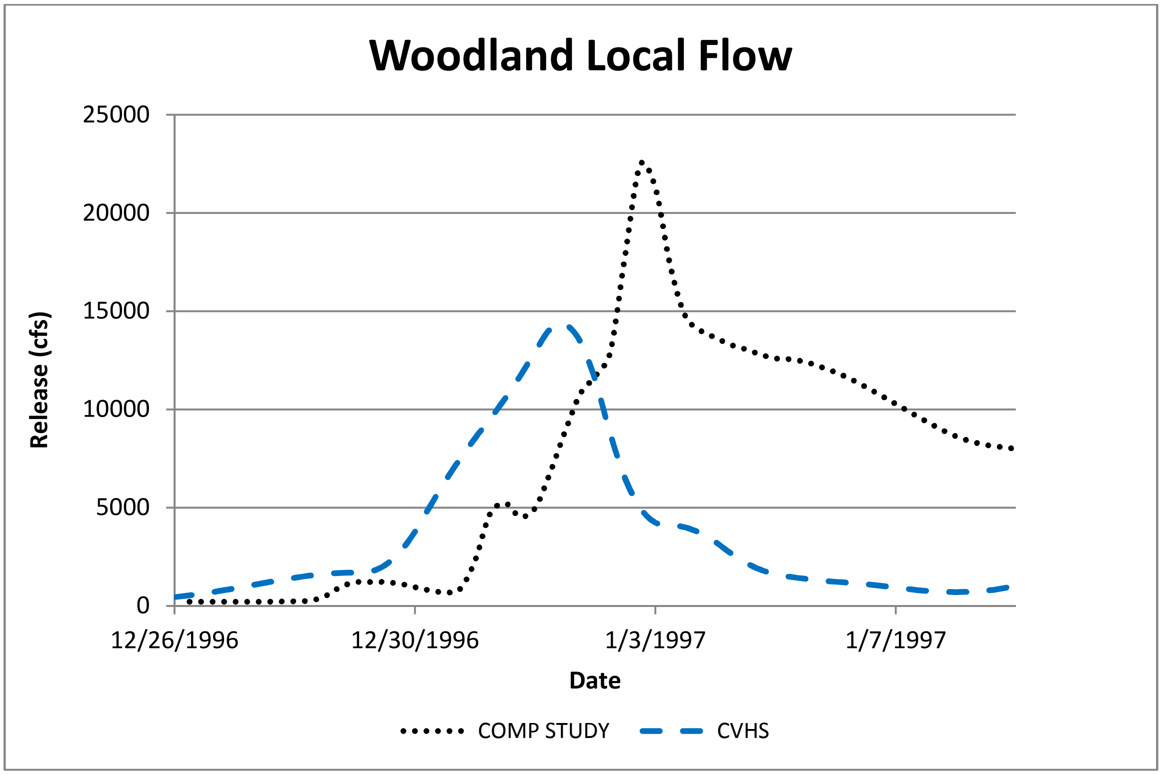

Marysville Local Flow
100000
80000
60000
40000
20000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS


Nicolaus Local Flow
35000
30000
25000
20000
15000
10000
5000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS

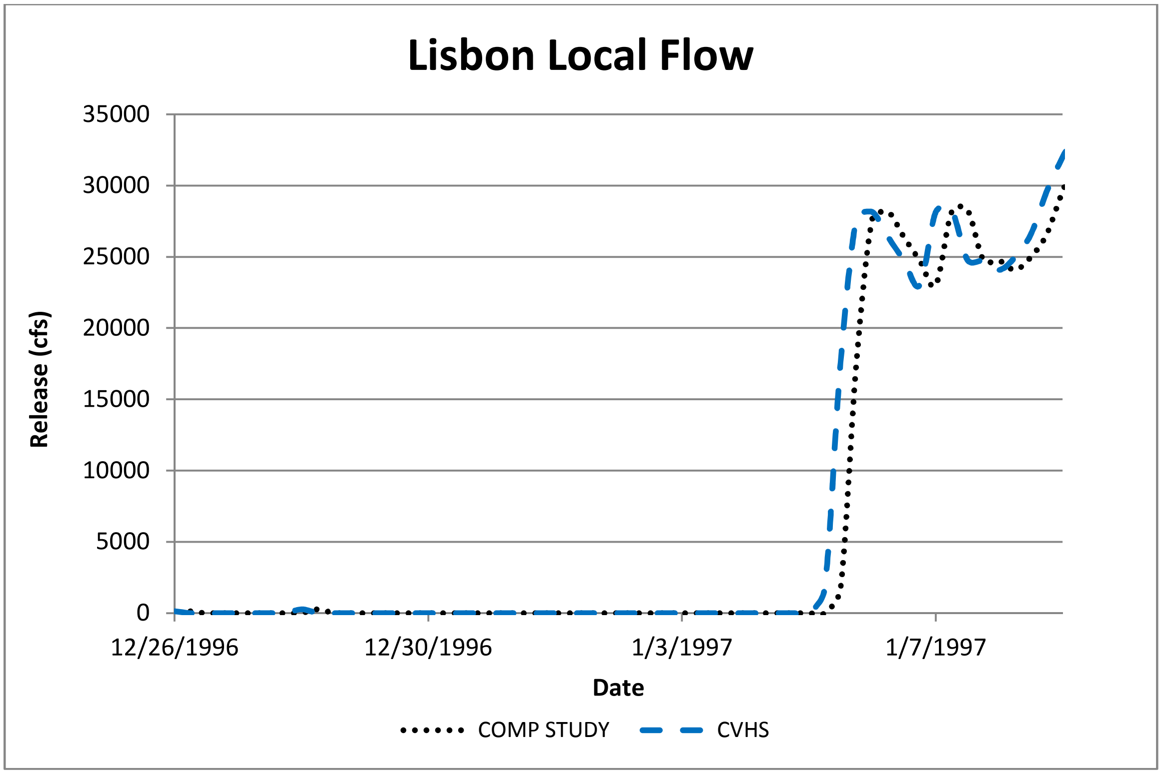

Fremont Weir Local Flow
80000
70000
60000
50000
40000
30000
20000
10000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS



Colusa Drain Local Flow
60000
50000
40000
30000
20000
10000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS



Woodland Local Flow
25000
20000
15000
10000
5000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS


Lisbon Local Flow
35000
30000
25000
20000
15000
10000
5000
0
Release (cfs)
12/26/1996
12/30/1996
1/3/1997
1/7/1997
Date
COMP STUDY
CVHS