CENTRAL VALLEY
FLOOD MANAGEMENT
PLANNING PROGRAM

FloodSAFE
CALIFORNIA

PUBLIC DRAFT | January 2012

2012 Central Valley Flood Protection Plan

A Path for Improving Public Safety, Environmental Stewardship, and Long-Term Economic Stability

Volume III
Attachments 8 through 8E

DEPARTMENT OF WATER RESOURCES
STATE OF CALIFORNIA

PUBLIC SAFETY ENVIRONMENTAL STEWARDSHIP ECONOMIC STABILITY

Cover Photo: Sacramento Weir (December 24, 1964), DWR Photo Lab

The Sacramento Weir and Bypass discharges excess flows from the Sacramento River (on the left) into the Yolo Bypass (not shown). The 1964-65 water year was marked by one of the most disastrous floods in California's history.

2012 Central Valley Flood Protection Plan

Public Draft, January 2012

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January 2012
Public Draft

Attachment 8

Technical Analysis Summary Report

CENTRAL VALLEY
FLOOD MANAGEMENT
PLANNING PROGRAM

FloodSAFE
CALIFORNIA

Public Draft

2012 Central Valley Flood Protection Plan

Attachment 8: Technical Analysis Summary Report

January 2012

STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

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1.0 Introduction

This section states the purpose of this report, gives background information (including a description of planning areas, goals, and approaches) and provides an overview of the report organization.

1.1 Purpose of this Report

This Technical Analysis Summary Report provides an overview of the technical analysis approach, tools, and data supporting development of the 2012 Central Valley Flood Protection Plan (CVFPP).

1.2 Background

As authorized by Senate Bill 5, also known as the Central Valley Flood Protection Act of 2008, the California Department of Water Resources (DWR) has prepared a sustainable, integrated flood management plan called the CVFPP, for adoption by the Central Valley Flood Protection Board (Board). The 2012 CVFPP provides a systemwide approach to protecting lands currently protected from flooding by existing facilities of the State Plan of Flood Control (SPFC), and will be updated every 5 years.

As part of development of the CVFPP, a series of technical analyses were conducted to evaluate hydrologic, hydraulic, geotechnical, economic, ecosystem, and related conditions within the flood management system and to support formulation of system improvements. These analyses were conducted in the Sacramento River Basin, San Joaquin River Basin, and Sacramento-San Joaquin Delta (Delta).

1.3 CVFPP Planning Areas

For planning and analysis purposes, and consistent with legislative direction, two geographical planning areas were important for CVFPP development (Figure 1-1):

- **SPFC Planning Area –** This area is defined by the lands currently receiving flood protection from facilities of the SPFC (see *State Plan of Flood Control Descriptive Document* (DWR, 2010)). The State of

California's (State) flood management responsibility is limited to this area.

- **Systemwide Planning Area –** This area includes the lands that are subject to flooding under the current facilities and operation of the Sacramento-San Joaquin River Flood Management System (California Water Code Section 9611). The SPFC Planning Area is completely contained within the Systemwide Planning Area which includes the Sacramento River Basin, San Joaquin River Basin, and Delta regions.

Planning and development for the CVFPP occurs differently in these planning areas. The CVFPP focused on SPFC facilities; therefore, evaluations and analyses were conducted at a greater level of detail within the SPFC Planning Area than in the Systemwide Planning Area.

1.4 2012 CVFPP Planning Goals

To help direct CVFPP development to meet legislative requirements and address identified flood-management-related problems and opportunities, a primary and four supporting goals were developed:

- **Primary Goal** – Improve Flood Risk Management
- **Supporting Goals**:
  - Improve Operations and Maintenance
  - Promote Ecosystem Functions
  - Improve Institutional Support
  - Promote Multi-Benefit Projects

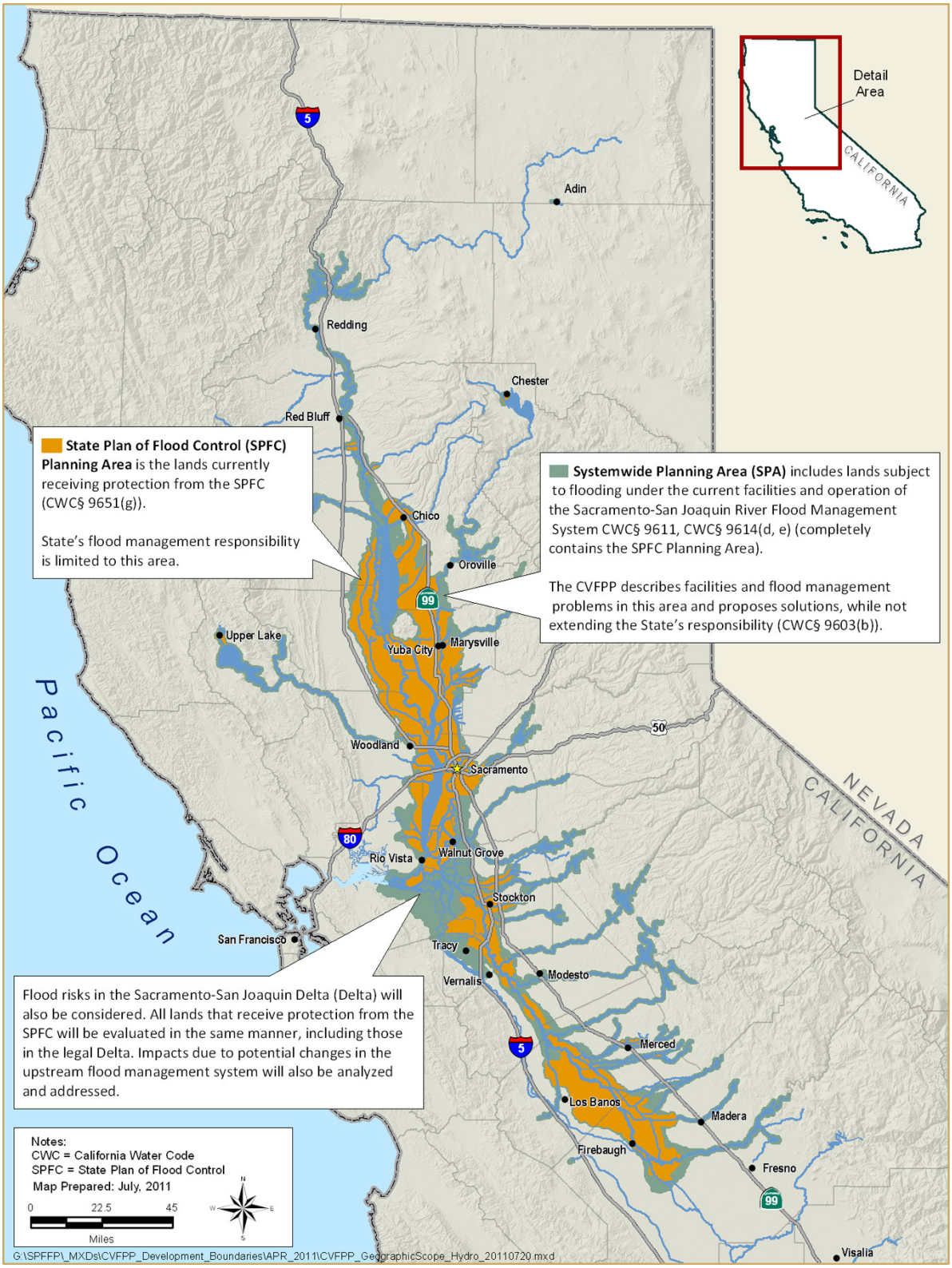

Figure 1-1. Central Valley Flood Protection Plan Planning Areas

1.5 2012 CVFPP Planning Approaches

No Project

- Continuation of existing conditions, including ongoing routine maintenance, floodfighting and post-flood repairs, and other flood management programs.
- Includes projects that are currently authorized, funded, permitted, and/or under construction.

In addition to the **No Project** approach, three fundamentally different approaches to flood management were initially compared to explore potential improvements in the Central Valley. These approaches are not alternatives; rather, they bracket a range of potential actions and help explore trade-offs in costs, benefits, and other factors important in decision making. The approaches are as follows:

- **Achieve SPFC Design Flow Capacity** – Address capacity inadequacies and other adverse conditions associated with existing SPFC facilities, without making major changes to the footprint or operation of those facilities.
- **Protect High Risk Communities** – Focus on protecting life safety for populations at highest risk, including urban areas and small communities.
- **Enhance Flood System Capacity** – Seek various opportunities to achieve multiple benefits through enhancing flood system storage and conveyance capacity.

Comparing these approaches helped identify the advantages and disadvantages of different combinations of management actions, and demonstrated opportunities to address the CVFPP goals to different degrees.

Based on this evaluation, a **State Systemwide Investment Approach** was developed that encompasses aspects of each of the approaches to balance achievement of the goals from a systemwide perspective, and includes integrated conservation elements. Figure 1-2 illustrates this plan formulation process.

As described above, this summary report describes the numerous technical analyses preformed to support the 2012 CVFPP.

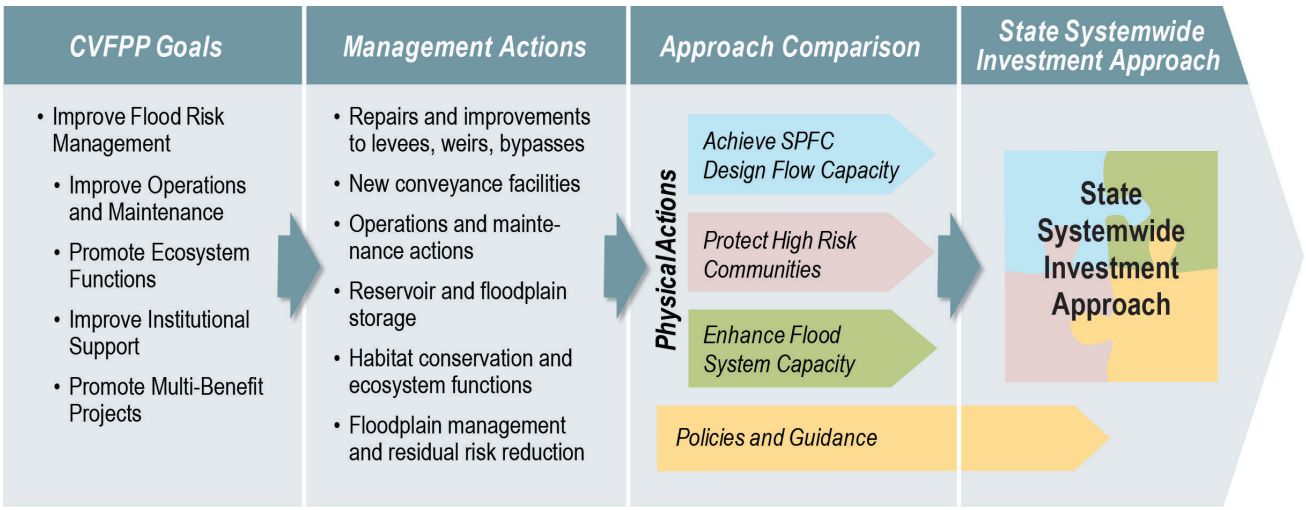

Figure 1-2. Formulation Process for State Systemwide Investment Approach

1.6 Report Organization

Organization of this document is as follows:

- Section 1 introduces and describes the purpose of this report and provides background information.
- Section 2 summarizes the physical approach elements of flood management actions evaluated in the 2012 CVFPP.
- Section 3 provides an overview of the methods used for comparing and evaluating No Project, the three preliminary approaches, and the State Systemwide Investment Approach.
- Section 4 provides an overview of other technical evaluations not used directly in the approach evaluations and comparisons.
- Section 5 describes the anticipated technical evaluation framework for the 2017 CVFP.
- Section 6 contains references for the sources cited in this document.
- Section 7 lists acronyms and abbreviations used in this document.

Attached to this report are 12 technical reports that document the technical analyses performed for the 2012 CVFPP. These documents are named in the List of Attachments section.

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2.0 Summary of Approach Elements Evaluated

Development of the CVFPP included formulating and evaluating three preliminary approaches to explore different potential physical changes to the existing flood management system and to assist in highlighting the need for policy or other management actions. Evaluation and comparison of the approaches focused primarily on the physical elements of the approaches. Technical studies were conducted to determine how physical changes to the system would affect performance of the system as a whole with respect to protecting public safety, reducing flood damages, restoring degraded ecosystems, and contributing to a wide range of multiple benefits. Technical analyses supporting the approach evaluations and comparisons are described in Section 3.

Tables 2-1 through 2-4 list the physical elements included in the No Project, three preliminary approaches, and State Systemwide Investment Approach. These physical elements include the following:

- Reservoir and floodplain storage features
- Bypass and weir modifications
- Flood structure improvements
- Levee improvements in urban areas, small communities, and rural-agricultural areas
- Ecosystem restoration features

Table 2-1. Storage Features Included in Approaches

Flood Management Element	No Project	Preliminary Approaches			State Systemwide Investment Approach
		Achieve SPFC Design Flow Capacity	Protect High Risk Communities	Enhance Flood System Capacity	
Reservoir Storage and Operations					
• Forecast-based/coordinated operations (Yuba/Feather) [1]	■	■	■	■	■
• Folsom Dam Joint Federal Project [2]	■	■	■	■	■
• Modify Lake Oroville release schedule (200 TAF effective increase in storage).				■	
Increase flood storage [3] • New Don Pedro Reservoir – 230 TAF • Friant Dam/Millerton Lake – 60 TAF • New Exchequer Dam/Lake McClure – 100 TAF				■	
Floodplain Storage					
• Sacramento River Basin – 200 TAF • San Joaquin River Basin – 100 TAF				■	

Notes:
[1] Coordinated operations implement two control points at confluence of Yuba and Feather rivers, and Feather River at Nicolaus.
[2] Folsom Dam Joint Federal Project (as authorized) modeled using USACE updated Folsom Dam operations model (provided by Kyle Keer at USACE Sacramento District, February 2011).
[3] Increase in flood storage was modeled as an increase in effective flood space allocation in these reservoirs. This increase can be achieved either through a physical raise of the existing dam or outlet/spillway structures, or reallocation of available storage space between the different water uses.
Key:
SPFC = State Plan of Flood Control
TAF = thousand acre-feet
USACE = U.S. Army Corps of Engineers

Table 2-2. Bypass System and Flood Structure Features Included in Approaches

Flood Management Element	No Project	Preliminary Approaches			State Systemwide Investment Approach
		Achieve SPFC Design Flow Capacity	Protect High Risk Communities	Enhance Flood System Capacity	
Bypass and Weir System					
• Tisdale Bypass and Fremont Weir dredging [1]	■	■	■	■	■
• Sutter Bypass widening • New Feather-Butte Basin Bypass • Fremont Weir widening • Yolo Bypass expansion • Sacramento Weir and Bypass widening • Lower San Joaquin River Bypass (widen Paradise Cut)				■	■
Flood Structure Improvements [2]					
• Gate structure for Feather River Bypass • Butte Basin small weir structures • Upgrade and modification of Colusa and Tisdale weirs • Sacramento Weir widening and automation • Gate structures and/or weir at Paradise Cut • Upgrade of structures in Upper San Joaquin Bypasses • Low-level reservoir outlets at New Bullard's Bar Dam • Fremont Weir widening and improvement • Additional pumping plants and small weirs				■	■
• Cache Creek sediment removal • Sacramento system sediment remediation downstream from weirs				■	■

Notes:
[1] Drawings of Fremont Weir sediment removal (DWR, 2006a) and Tisdale Weir sediment removal (DWR 2006b).
[2] Flood structure rehabilitation, erosion repair, and sediment removal were not modeled as part of any approach because of the negligible hydraulic effects on the system as a whole.
Key:
DWR = California Department of Water Resources
SPFC= State Plan of Flood Control

Table 2-3. Levee Improvement Features Included in Approaches

Flood Management Element	No Project	Preliminary Approaches			State Systemwide Investment Approach
		Achieve SPFC Design Flow Capacity	Protect High Risk Communities	Enhance Flood System Capacity	
Urban Levee Improvements [1]					
FloodSAFE Early Implementation Projects: • Natomas area levees improvements program,[2] Marysville ring levee,[3] Feather and Bear rivers levee improvements [4]	■	■	■	■	■
Levee improvements to pass 200-year water surface [5]			■	■	■
Levee reconstruction to safely pass SPFC design capacity flows [6]		■			
Small Community Levee Improvements [7]					
Protection from 100-year flood event for small communities within the SPFC Planning Area			■	■	■
Rural-Agricultural Levee Improvements					
Levee reconstruction to pass safely SPFC design capacity flows [6]		■		■	
Alternative rural improvements[8]: • Address known deficiencies based on 2011 inspection reports[9] • Restore crown and all-weather access roads					■

Notes:
[1] Urban area is areas with population greater than 10,000. They include Marysville, Yuba City/Live Oak/Gridley, Sacramento area, West Sacramento, Stockton area, and Lathrop and vicinity.
[2] Natomas area levee improvements (as constructed and/or planned/pending) are modeled using levee performance curves developed by the Urban Levees Evaluation (ULE) Program.
[3] Marysville levee improvements (as constructed) were modeled as reconstructed levees because ULE curve was not available. Reconstructed levees were modeled as levees with no probability of failure until overtopped.
[4] Three Rivers Levee Improvement Authority setback levee project (as constructed) was modeled as reconstructed levees.
[5] In simulating improvements to achieve an urban level of flood protection, the 200-year water surface profile from the No Project (baseline) simulation was used as the basis for establishing the probable failure point for urban levees. Actual level of protection in urban areas may be somewhat higher or lower than the 200-year, depending on the effects of other storage and conveyance features included in the approaches.
[6] Reconstructed SPFC levees were modeled as levees with no probability of failure until overtopped. In some reaches, levee crown elevations were increased to address freeboard deficiencies based on the information from the ULE and Non-Urban Levee Evaluation Programs. Level of protection for reconstructed levees varies.
[7] Small communities are areas with population less than 10,000. Small community improvements were not specifically modeled because of the negligible effects of improving small segments of SPFC levees. For the State Systemwide Investment Approach, small communities' protection is also subject to economic feasibility.
[8] Alternative rural improvements were not specifically modeled because of the negligible effects on levee performance curves.
[9] 2011 Inspection Report of the Central Valley State-Federal Flood Protection System. DWR Flood Project Integrity and Inspection Branch.
Key:
SPFC = State Plan of Flood Control

Table 2-4. Ecosystem Restoration Features Included in Approaches

Flood Management Element	No Project	Preliminary Approaches			State Systemwide Investment Approach
		Achieve SPFC Design Flow Capacity	Protect High Risk Communities	Enhance Flood System Capacity	
Ecosystem Restoration Features					
Fish Passage Improvements:[1] • Sutter Bypass and fish passage east of Butte Basin • Freemont Weir fish passage improvements • Yolo Bypass/Willow Slough Weir fish passage improvements • Yuba River fish passage and fish screen • Mendota Pool fish passage and fish screen				■	■
Setback levees:[2] • Lower Feather and Bear rivers • Sacramento River north of Tisdale Weir • Short reaches of Sacramento River south of Tisdale Weir • San Joaquin River between Merced and Stanislaus rivers				■	
Environmental conservation development [3] • For areas within new or expanded bypasses • For areas within connected floodplains in levee setback locations				■	■

Notes:
[1] Fish passage improvements were not simulated because of localized effect on system operations.
[2] Levee setbacks were modeled as 1,000- to 2,000-foot expansion of the floodway corridor, depending on the topography. Levees on both sides of the setback were modeled as reconstructed levees with no probability of failure until overtopped.
[3] Environmental conservation developments in the floodway would be designed to have limited hydraulic effects on the flood carrying capacity of the system. Therefore, these elements were not modeled because of anticipated localized effects.
Key:
SPFC =State Plan of Flood Control

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3.0 Evaluation Methods for Approach Comparison

To support development of the 2012 CVFPP, existing and available data and tools were primarily used to help understand the performance of the existing flood management system, and assess the effects of proposed improvements. This section describes the evaluation methods and analytical studies conducted to support evaluation and comparison of the preliminary approaches, and formulation of the State Systemwide Investment Approach.

3.1 Overview of Evalaution Methods

The analytical studies needed to support plan formulation included a series of sequential and parallel evaluations and analyses that commenced with hydrology to develop unregulated flow hydrographs into reservoirs and streams. This was followed by reservoir models to develop regulated flows for the riverine and estuary hydraulic models, which route floodflows and simulate water stages, flow rates, levee breaches, and out-of-bank flows. Geotechnical levee performance characterizations that describe levee failure probability throughout the system provided levee performance curves for the riverine hydraulic models. Out-of-bank flows were routed using floodplain hydraulic models to characterize the extent and depth of floodplains. Risk analysis was then conducted using geotechnical and hydrologic/hydraulic information and uncertainties to assess economic damages and life risk. Conceptual-level design and cost estimates were also developed for the proposed flood management features. Change to regional economic output and employment due to proposed flood improvement was assessed using cost and economic information.

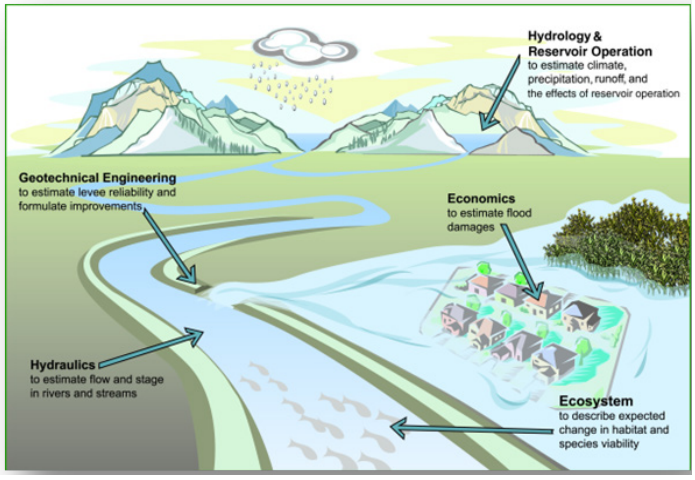
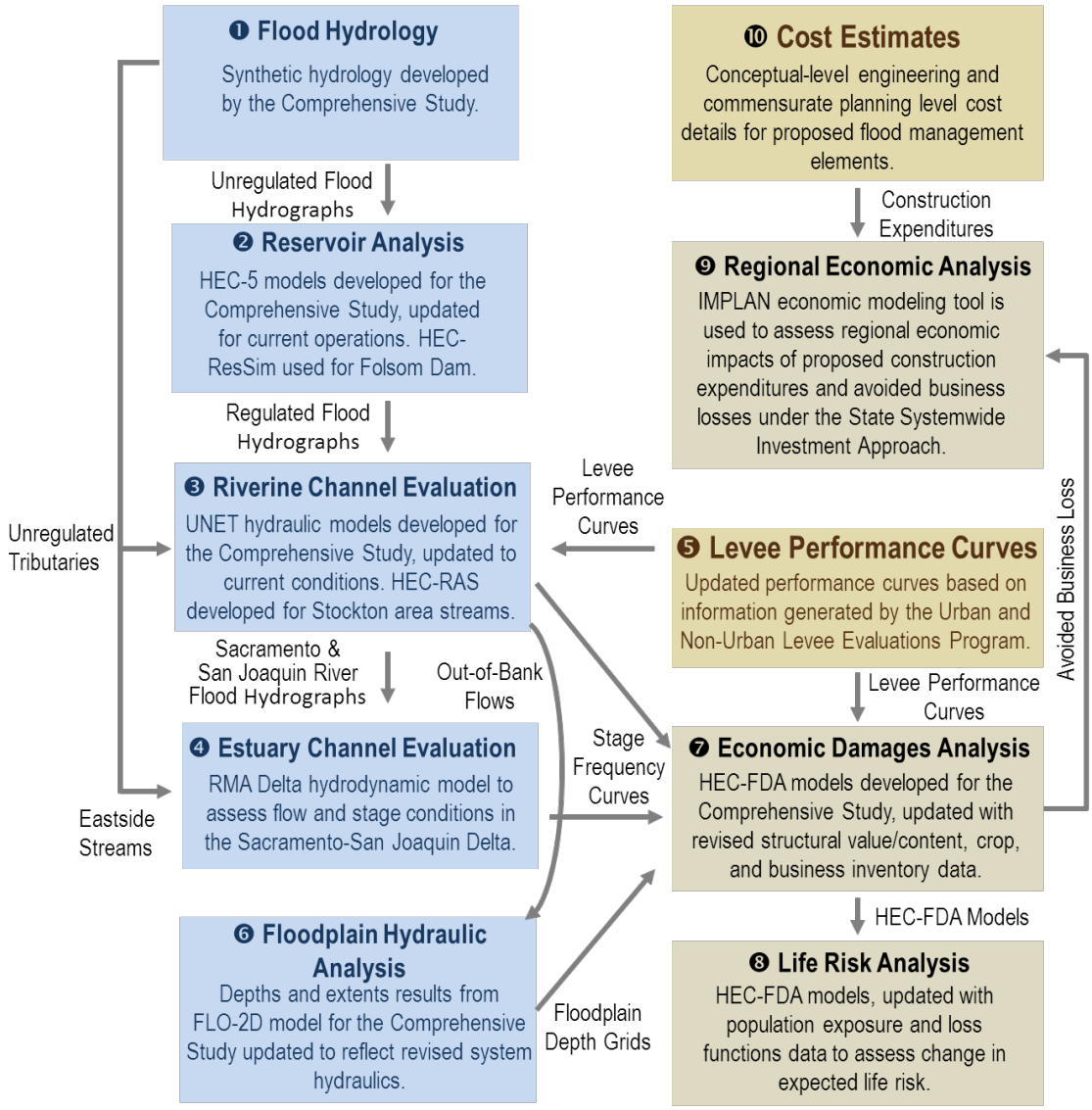


Figure 3-1 illustrates the technical analyses and tools supporting the 2012 CVFPP. These key technical analyses and tools are briefly described in the following sections.



Legend:

Comprehensive Study	*Sacramento and San Joaquin River Basins Study Comprehensive Study (USACE, 2002)*
HEC	USACE Hydrologic Engineering Center
HEC-FDA	HEC Flood Damage Analysis model
FLO-2D	Fullerton, Lenzotti, and O'Brien – Two Dimensional model
HEC-RAS	HEC River Analysis System model
HEC-ResSim	HEC Reservoir Operations Simulation model
HEC-5	HEC Reservoir Operations Simulation model (predecessor to HEC-ResSim)
RMA	RMA Finite Element Model of Sacramento-San Joaquin Delta hydrodynamics
UNET	One-Dimensional Unsteady Network Flow model (predecessor to HEC-RAS)
USACE	U.S. Army Corps of Engineers

Figure 3-1. Technical Analyses and Tools Supporting 2012 CVFPP Development

3.2 Flood Hydrology

Synthetic hydrology was adopted for the 2012 CVFPP based on the "composite floodplain" concept. This concept recognizes that a frequency-based floodplain is not created by a single flood event, but by a combination of several events, each of which shapes the floodplain at different locations. The composite floodplain represents the maximum extent of inundation possible at all locations for any simulated storm events. To construct a composite floodplain, a series of storm centerings, which is a set of storms with different return periods (annual exceedence probabilities), assigned to a set of tributaries, was developed to characterize flooding in different parts of the Sacramento and San Joaquin river basins. This synthetic flood hydrology generated unregulated flow hydrographs into reservoirs and streams. The synthetic hydrology developed for the Comprehensive Study (USACE, 2002) was adopted for the 2012 CVFPP. Details of synthetic hydrology development and use are documented in Attachment 8A: Hydrology.

3.3 Reservoir Analysis

Reservoirs and storage facilities in the Sacramento and San Joaquin river basins provide an important flood management function in regulating flood flows. Using the synthetic flood hydrographs, reservoir models simulate operations of the Sacramento River and San Joaquin River multipurpose reservoirs to generate regulated flood releases. Reservoir analysis for the CVFPP used the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center 5 (HEC-5) reservoir models (USACE, 1998) developed for the Comprehensive Study (USACE, 2002). These HEC-5 models were updated to accurately represent current operations. In addition, HEC Reservoir Simulation (HEC-ResSim) model for Folsom Lake was used to simulate modified releases from Folsom Lake under the Joint Federal Project (Reclamation, 2009). The reservoir analysis evaluated potential changes to flood storage and releases in reservoirs in the Sacrament and San Joaquin river basins to improve flood management. Details of these technical evaluations are documented in Attachment 8B: Reservoir Analysis.

3.4 Riverine Channel Evaluations

Riverine hydraulic models were used to define flow rates and water stages, levee breach locations, and out-of-bank flows along the Sacramento and San Joaquin rivers and their tributaries under various synthetic flood events. The Unsteady Network (UNET) hydraulic model (USACE, 1997) developed for the Comprehensive Study (USACE, 2002) was selected for use in the CVFPP study because it provides extensive coverage of the flood management system in the Sacramento and San Joaquin river basins. These models were updated to represent current conditions, including updated levee performance information and other changes in channel and levee characteristics. In addition, HEC River Analysis System (HEC-RAS) models for the Calaveras River, Mormon Slough, and Bear Creek were developed to simulate streams in the Stockton area. Details of tools updates and technical evaluations are documented in Attachment 8C: Riverine Channel Evaluations.

3.5 Estuary Channel Evaluations

Estuary channel evaluations focused on analyzing potential impacts that occur in the Delta as a result of upstream changes to operations and facilities of the Sacramento-San Joaquin River flood management system. Flows from the riverine hydraulic models for the Sacramento and San Joaquin rivers were the inputs to the estuary channel hydraulic model to develop Delta flows and stages. The USACE version of the Resource Management Associates, Inc. (RMA), Delta hydrodynamic model was used to simulate tidally influenced flow conditions in the Delta (RMA, 2005). Details of these technical evaluations are documented in Attachment 8D: Estuary Channel Evaluations.

3.6 Levee Performance Curves

Updated levee performance curves to reflect levee performance were developed for the entire SFPC levee system in the Sacramento and San Joaquin river basins using information generated by the DWR Urban and Non-Urban Levee Evaluations (ULE and NULE) Programs (URS Corporation, 2010; Kleinfelder, 2010). Performance curves for specific levee segments provided the relationship between river water surface elevation (or stage) and the probability that a levee segment would fail when exposed to that water surface elevation. For each levee segment, performance curves were developed for each failure mode: under-seepage, stability, through-seepage, and erosion. These independent performance curves were then mathematically combined to produce the cumulative or

overall performance curve for the segment or reach. These levee performance curves were inputs to the hydraulics and economic models to describe geotechnical probability of levee failure. Details of levee performance curve development are documented in Attachment 8E: Levee Performance Curves.

3.7 Floodplain Hydraulic Analysis

The riverine and estuary hydraulic analyses generated out-of-bank flows caused by overtopping or levee failures. These flows traveling out of stream channels and across the topography of the floodplain were used in the floodplain hydraulic modeling to delineate the floodplains and provide information on floodplain extent and depth for the various synthetic flood events. Floodplain information generated by the Fullerton, Lenzotti and O'Brien – Two Dimensional (FLO-2D) hydraulic models developed for the Comprehensive Study (USACE, 2002) was updated to reflect the change in system performance and levee conditions through developing revised flood depth grids. Details of the development and application of the floodplain information are documented in Attachment 8F: Flood Damage Analysis.

3.8 Flood Damage Analysis

Risk-based analysis of the economic consequences of flood inundation developed estimates of expected (long-term average) annual economic damages. These estimates included structure and content damages, crop damages in inundated agricultural lands, and business income and production losses. To describe the hydrologic, hydraulic, and geotechnical performance of the system and uncertainties, the flood damage analysis used levee performance curves, stage-frequency curves from riverine and estuary hydraulic models, and flood depth information from the floodplain hydraulic analysis. To describe the economic consequences of flood inundation, the analysis used information from a 2010 reconnaissance-level structural inventory, 2010 spatial pattern of cropping, and contents-structure ratios and depth-damage functions (USACE, 2008). The risk-based analysis was conducted using the HEC Flood Damage Analysis (HEC-FDA) model, which computes the expected value of damage while explicitly accounting for uncertainties. Details of the economic evaluations are documented in Attachment 8F: Flood Damage Analysis.

3.9 Life Risk Analysis

Risk-based analysis of the public safety consequences of flood inundation developed estimates of expected annual life risk in similar fashion to the flood damage analysis. The life safety analysis used HEC-FDA models developed for the economic damages analysis to generate annual expected life risk. For population exposure and inundation consequences, the analysis used 2000 U.S. Census population data, which was the best available information at the time the analysis was conducted, and mortality-depth curves (Jonkman, 2007). Details of the life risk evaluations are documented in Attachment 8G: Life Risk Analysis.

3.10 Regional Economic Analysis

Regional economic analysis evaluates the effects of changes in production or expenditures due to proposed flood management improvements on a region's economy. It estimates direct, indirect, and induced employment and economic output effects related to changes in potential business income losses, and proposed construction expenditures to improve flood management facilities. The IMPLAN economic modeling tool was used for the regional economic analysis (Minnesota IMPLAN Group, 2009). This regional economic analysis was conducted only for the State Systemwide Investment Approach. Details of the regional economic evaluations are documented in Attachment 8H: Regional Economic Analysis for the State Systemwide Investment Approach.

3.11 Cost Estimates

Conceptual-level engineering and commensurate level of cost details were developed for the flood management elements included in the CVFPP preliminary approaches and the State Systemwide Investment Approach. These costs were not based on bid-ready engineering documents, but rather on conceptual designs and remedial actions extracted from multiple evaluation efforts. The cost estimates carry an appropriate level of contingency for a conceptual-level planning effort. Details of the cost estimate methodology are included in Attachment 8J: Cost Estimates.

4.0 Additional Supporting Evaluations

Other evaluations not directly used in approach comparison were conducted to investigate potential opportunities for floodplain restoration, assess the effects of climate change on flood management, and identify potential opportunities to incorporate groundwater recharge into flood management activities. These studies are described in the following sections.

4.1 Floodplain Restoration Opportunities Analysis

To support the identification, development, and implementation of specific restoration actions, a Floodplain Restoration Opportunity Analysis was conducted. This analysis identified areas with greater and/or more extensive potential opportunities for ecological restoration of floodplains. These areas were identified through considering physical suitability, and opportunities and constraints related to existing land cover and land uses, locations and physical condition of levees, locations of other major infrastructure, conservation status of land, and locations that stakeholders are interested in restoring.

To evaluate physical suitability, the concept of floodplain inundation potential (FIP) was applied in a geographic information system (GIS) analysis of corridors along the Sacramento and San Joaquin rivers and their major tributaries. To assess physical suitability for restoration actions, the FIP analysis adapted concepts from the HEC Ecosystem Functions Model (HEC-EFM) (USACE, 2009), the Frequently Activated Floodplain concept (Williams et al., 2009), and the Height Above River GIS tool (Dilts et al., 2010). FIP analysis identified areas of floodplain, both directly connected to a river and disconnected from the river (e.g., behind natural or built levees or other flow obstructions) that could be inundated by particular floodplain flows. The flows evaluated by the analysis included a spring flow sustained for at least 7 days and occurring in 2 out of 3 years, and with 2- and 10-year return flood flows.

The identified areas with restoration potential were then prioritized based on location, acreage, and potential ecosystem functions and services. This analysis provides the foundation for subsequent planning efforts to develop specific restoration opportunities in conjunction with planned flood

management improvements. Floodplain restoration opportunities analysis is documented in the Supporting Documentation for the Conservation Framework.

4.2 Climate Change Analysis

The prediction of extreme events is one of the most challenging areas for climate change because of the high degree of uncertainties and the limitations of modeling tools and available information. Traditional top-down, risk-based assessments for flood management could not be properly applied because the scenarios from the International Panel on Climate Change do not present a statistical relationship to support the risk analysis (Dessai and Hulme, 2003).

As part of the ongoing development of the 2012 CVFPP, two topic work groups dealing with climate change developed, recommended, and described a unique threshold approach for analyzing climate change in the context of flood management. The Threshold Analysis Approach is a bottom-up approach focusing on vulnerability and associated prudent investments, which aim at broadening the chance of adaptation regardless of which climate change scenarios may be realized, rather than focusing on maximizing the benefits from selected scenarios. The thresholds or vulnerabilities can be assessed at system, regional, and community levels and the concepts are not limited to flood management applications. For the 2012 CVFPP, a pilot study was conducted using the draft Feather-Yuba coordinated operation model developed under the DWR Central Valley Hydrology Study (CVHS). The vulnerability of dam flow release capacity and of downstream flow objectives was assessed in the context of a surrogate index of Atmospheric Rivers (Dettinger, 2011). The results show promise for the proposed methodology, although much work and research are needed for a full application, which is expected for the 2017 CVFPP update. Attachment 8K: Climate Change Analysis documents the climate change analysis conducted for the 2012 CVFPP.

4.3 Groundwater Recharge Opportunities Analysis

Groundwater recharge opportunities analysis identified potential opportunities for enhanced groundwater recharge in conjunction with flood management activities for the dual benefit of increased flood management flexibility and increased water supply reliability. Three broad categories of groundwater recharge were evaluated: recharge projects associated with reservoir reoperation, groundwater banking projects associated with

capturing unappropriated floodflows, and recharge associated with activities in the floodplain. This analysis is documented in Attachment 8L: Groundwater Recharge Opportunities Analysis.

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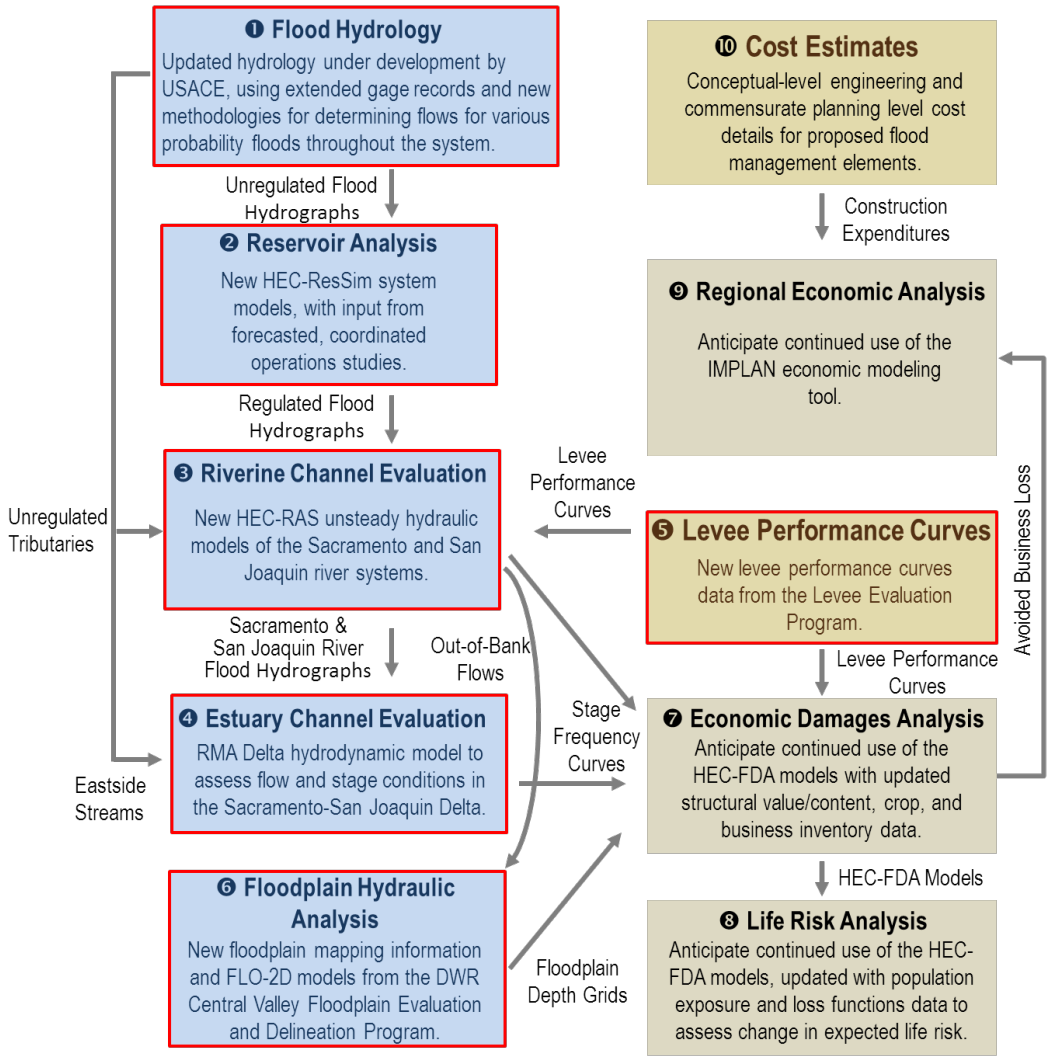
5.0 Continued Tool Development for 2017 CVFPP Update

Currently, multidisciplinary efforts are ongoing to develop new data and tools for use beyond 2012. While results of these efforts will not be available for use in the 2012 CVFPP, this next generation of information will be available to support more detailed technical analyses for the 2017 CVFPP update. Figure 5-1 highlights new information and tools that are being developed to support the 2017 CVFPP update, which are briefly described below:

- Updated flood hydrology being developed in coordination with USACE through the DWR CVHS.
- New reservoir operations models (HEC-ResSim) to simulate the operation of the major flood management reservoirs, under development through the DWR CVHS.
- New riverine hydraulic models (HEC-RAS) to simulate flows in the Sacramento and San Joaquin river channels, under development through the Central Valley Flood Evaluation and Delineation (CVFED) Program.
- Updated floodplain hydraulic models (FLO-2D) to estimate flood depth and extent, under development through the CVFED Program.
- New information from ULE and NULE to inform understanding of the reliability of flood management features in the entire SPFC Planning Area.



Legend:

DWR	California Department of Water Resources
HEC	USACE Hydrologic Engineering Center
HEC-FDA	HEC Flood Damage Analysis model
FLO-2D	Fullerton, Lenzotti, and O'Brien – Two Dimensional model
HEC-RAS	HEC River Analysis System model
HEC-ResSim	HEC Reservoir Operations Simulation model
RMA	RMA finite element model of Sacramento-San Joaquin Delta hydrodynamics
USACE	U.S. Army Corps of Engineers
▭	Indicates use of new technical tool or data to support the 2017 CVFPP update

Figure 5-1. New Technical Data and Tools Being Developed to Support 2017 CVFPP Update

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7.0 Acronyms and Abbreviations

Board ..... Central Valley Flood Protection Board

CVFED ..... Central Valley Flood Evaluation and Delineation Program

CVFPP ..... Central Valley Flood Protection Plan

CVHS ..... Central Valley Hydrology Study

Delta..... Sacramento-San Joaquin Delta

DWR ..... California Department of Water Resources

FIP ..... flood inundation potential

FLO-2D ..... Fullerton, Lenzotti, and O'Brien – Two Dimensional

GIS..... Geographic Information System

HEC ..... Hydrologic Engineering Center

HEC-5 ..... Hydrologic Engineering Center 5

HEC-EFM..... HEC Ecosystem Functions Model

HEC-FDA ..... HEC Flood Damage Analysis

HEC-RAS ..... HEC River Analysis System

HEC-ResSim ..... HEC Reservoir Simulation

NULE ..... Non-Urban Levee Evaluation

RMA ..... Resource Management Associates, Inc.

SPFC ..... State Plan of Flood Control

State..... State of California

ULE ..... Urban Levee Evaluations

UNET ..... Unsteady Network

USACE..... U.S. Army Corps of Engineers

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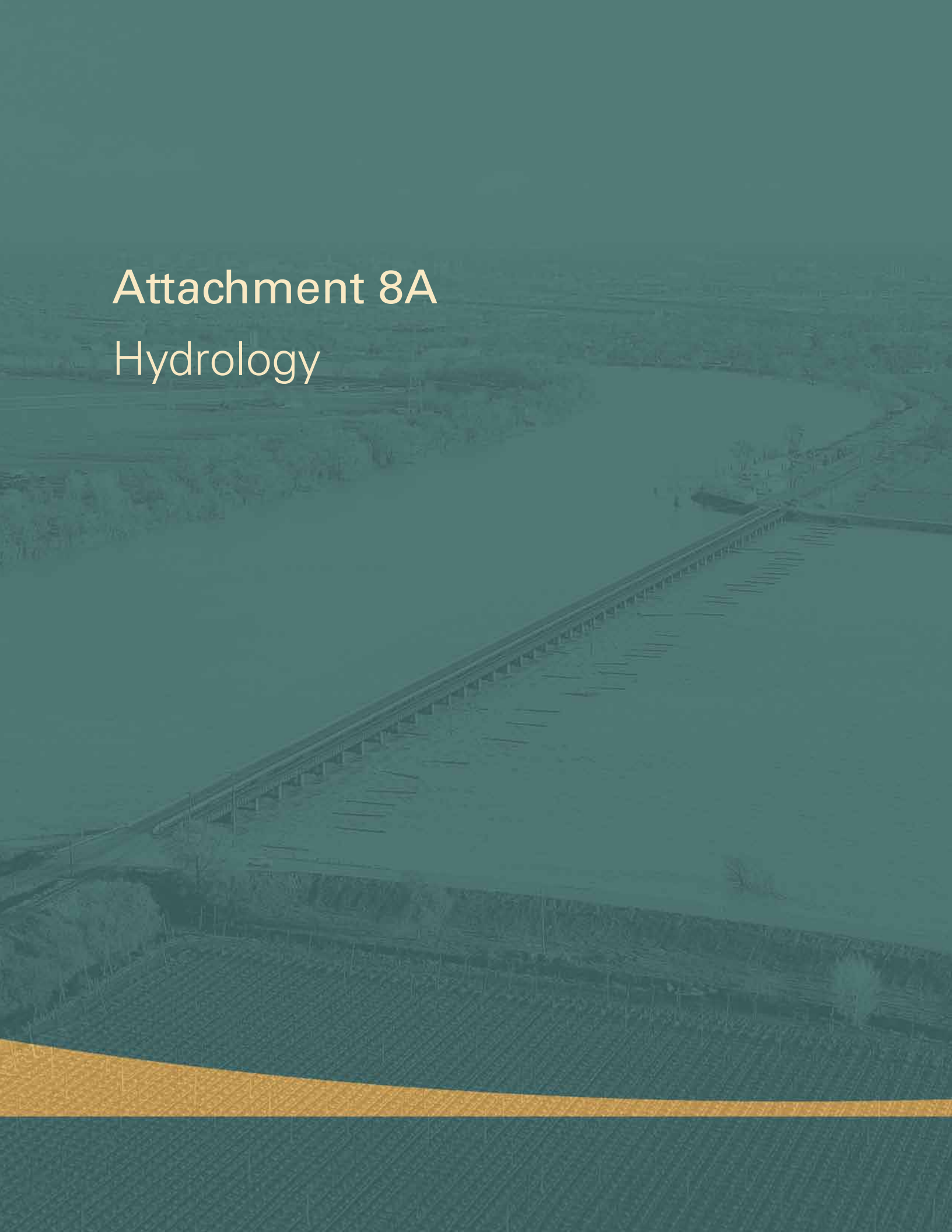
STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

Attachment 8A

Hydrology



CENTRAL VALLEY
FLOOD MANAGEMENT
PLANNING PROGRAM

FloodSAFE
CALIFORNIA

Public Draft

2012 Central Valley Flood Protection Plan

Attachment 8A: Hydrology

January 2012

STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

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1.0 Introduction

This section states the purpose of this attachment, gives background information (including a description of planning areas, goals, and approaches), discusses assumptions and limitations to the data, and provides an overview of the hydrology report organization.

1.1 Purpose of this Attachment

As part of development of the 2012 Central Valley Flood Protection Plan (CVFPP), a series of technical analyses were conducted to evaluate hydrologic, hydraulic, geotechnical, economic, ecosystem, and related conditions within the Sacramento and San Joaquin river basins flood management system and to support formulation of system improvements.

An important step in conducting these analyses was to establish existing (No Project) hydrologic conditions on a regional/generalized basis. Hydrologic conditions were input into hydrologic and hydraulic models, as described in subsequent attachments.

The 2012 CVFPP used a subset of the hydrology developed for the *Sacramento and San Joaquin River Basins Comprehensive Study* (Comprehensive Study) (USACE and DWR, 2002a). Hydrology from the Comprehensive Study is applicable for use in the 2012 CVFPP because no major flood has occurred in the Sacramento and San Joaquin river basins to modify the hydrology since development of the Comprehensive Study (the last major flood occurred 5 years before the study, in 1997). While levee repairs and improvements have been made since the Comprehensive Study, channel and floodplain conditions in the Sacramento and San Joaquin river basins have not altered significantly.

The 2012 CVFPP hydrology used six of the seven Comprehensive Study synthetic annual exceedence probability (AEP) storm events: 10, 4, 2, 1, 0.5, and 0.2 percent. The 50 percent AEP storm was not used because the Sacramento and San Joaquin river basins' flood management system was assumed to handle storms of at least this magnitude.

To reduce the complexity of analysis for the CVFPP, 10 of the 23 flood runoff centerings (storm centerings) from the Comprehensive Study were used for the 2012 CVFPP hydrology to provide peak flows as input into the riverine hydraulic models (refer to Section 3 for more details). The

following five Sacramento River Basin storm centerings were used to develop hydrographs for use as inputs to the reservoir operations and riverine hydraulic models:

- Shasta Lake to Ord Ferry (Shasta-centered)
- Ord Ferry to Feather River (Ord Ferry-centered)
- Yuba River near Marysville (Yuba-centered)
- Sacramento River at the latitude of Sacramento (Sacramento-centered)
- American River at Fair Oaks (American-centered).

Five San Joaquin River Basin storm centerings were used as inputs to the reservoir operations and riverine hydraulic models, as follows:

- San Joaquin River at Friant (Friant-centered)
- San Joaquin River at the latitude of El Nido (El Nido-centered)
- San Joaquin River at the latitude of Newman (Newman-centered)
- San Joaquin River at the latitude of Vernalis (Vernalis-centered)
- Merced River at Exchequer (Exchequer-centered)

Figure 1-1 shows the locations of the 10 storm centerings used for the 2012 CVFPP. These locations were chosen because they are either on the mainstem of the rivers (i.e., produce large runoff on a basin-wide level) or are on major tributaries (i.e., generate extremely large floods on individual rivers).

The following sections summarize Comprehensive Study Appendix B – Synthetic Hydrology, which includes the assumptions, hydrologic analyses, and findings used to develop the Comprehensive Study hydrology (USACE and DWR, 2002b). As stated above, portions of this hydrology were used as inputs for the 2012 CVFPP technical analyses.

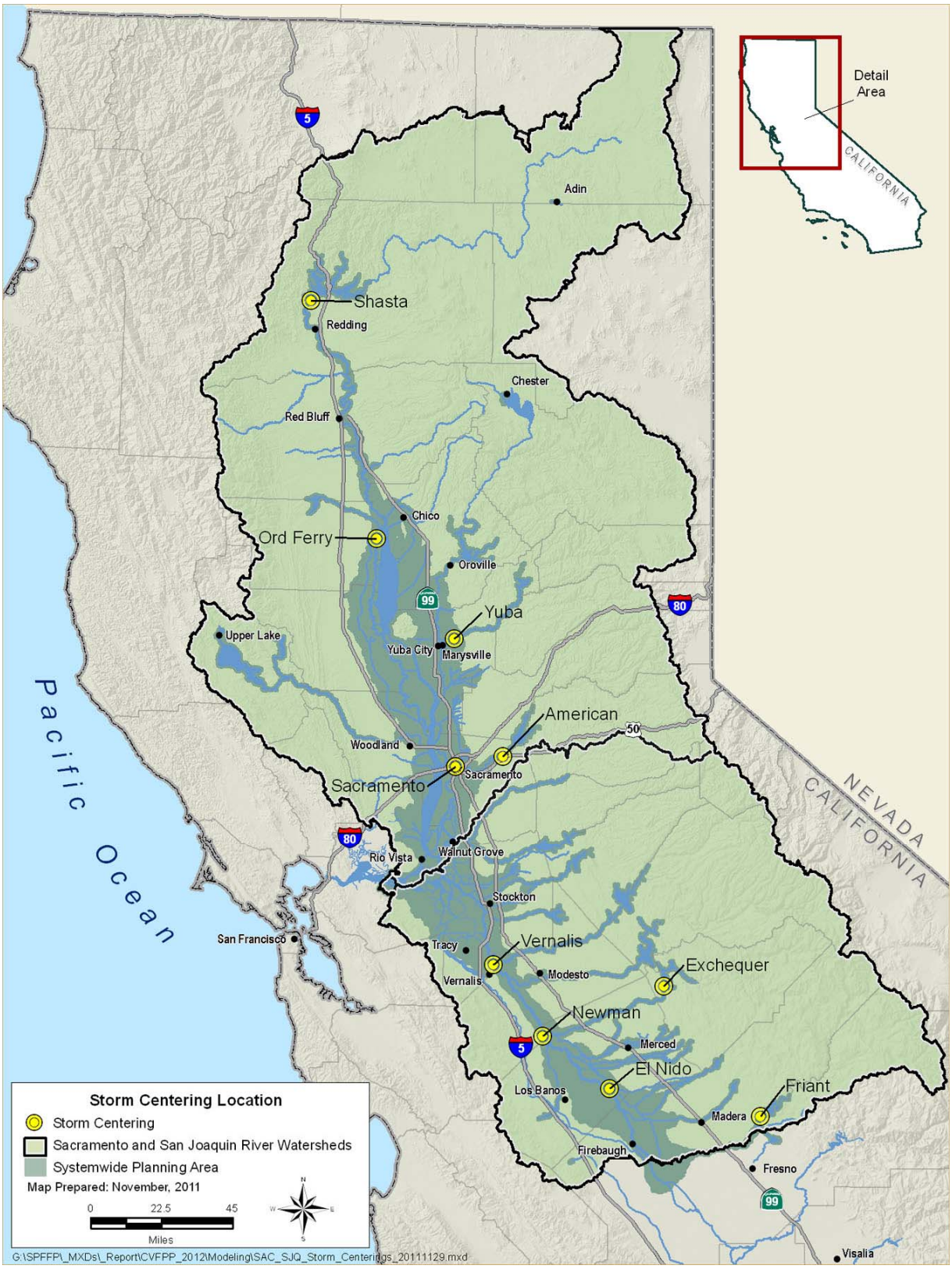

Figure 1-1. 2012 Central Valley Flood Protection Plan Storm Centering Locations

1.2 Background

As authorized by Senate Bill 5, also known as the Central Valley Flood Protection Act of 2008, the California Department of Water Resources (DWR) has prepared a sustainable, integrated flood management plan called the CVFPP, for adoption by the Central Valley Flood Protection Board (Board). The 2012 CVFPP provides a systemwide approach to protecting lands currently protected from flooding by existing facilities of the State Plan of Flood Control (SPFC), and will be updated every 5 years.

As part of development of the CVFPP, a series of technical analyses were conducted to evaluate hydrologic, hydraulic, geotechnical, economic, ecosystem, and related conditions within the flood management system and to support formulation of system improvements. These analyses were conducted in the Sacramento River Basin, San Joaquin River Basin, and Sacramento-San Joaquin Delta (Delta).

1.3 CVFPP Planning Areas

For planning and analysis purposes, and consistent with legislative direction, two geographical planning areas were important for CVFPP development (Figure 1-2):

- **SPFC Planning Area** – This area is defined by the lands currently receiving flood protection from facilities of the SPFC (see *State Plan of Flood Control Descriptive Document* (DWR, 2010)). The State of California's (State) flood management responsibility is limited to this area.

- **Systemwide Planning Area** – This area includes the lands that are subject to flooding under the current facilities and operation of the Sacramento-San Joaquin River Flood Management System (California Water Code Section 9611). The SPFC Planning Area is completely contained within the Systemwide Planning Area which includes the Sacramento River Basin, San Joaquin River Basin, and Delta regions.

Planning and development for the CVFPP occurs differently in these planning areas. The CVFPP focused on SPFC facilities; therefore, evaluations and analyses were conducted at a greater level of detail within the SPFC Planning Area than in the Systemwide Planning Area.

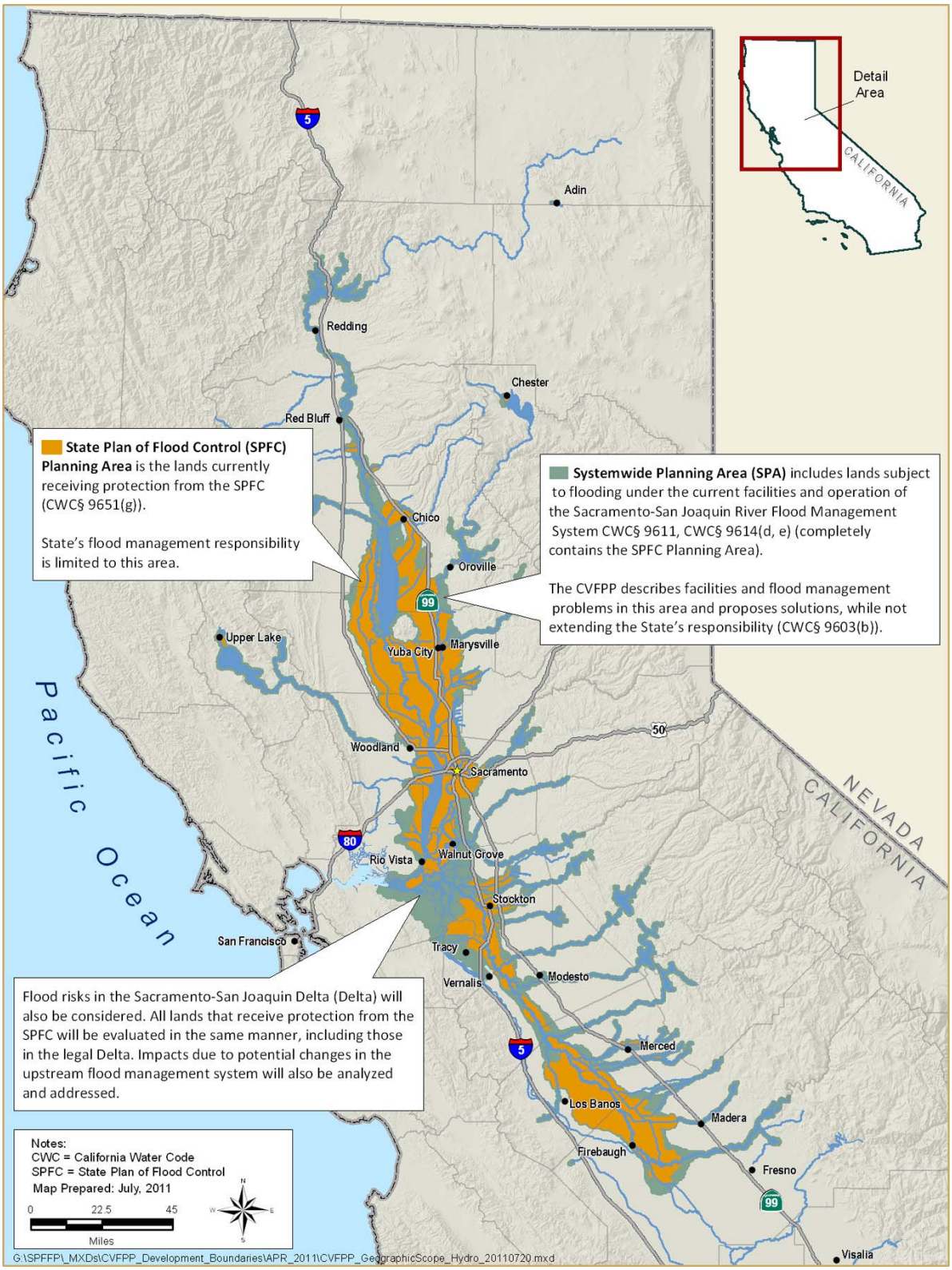

Figure 1-2. Central Valley Flood Protection Plan Planning Areas

Hydrology development for the 2012 CVFPP extends beyond the Systemwide Planning Area and encompasses the Sacramento and San Joaquin river basins.

1.4 2012 CVFPP Planning Goals

To help direct CVFPP development to meet legislative requirements and address identified flood-management-related problems and opportunities, a primary and four supporting goals were developed:

- **Primary Goal** – Improve Flood Risk Management
- **Supporting Goals**:
  - Improve Operations and Maintenance
  - Promote Ecosystem Functions
  - Improve Institutional Support
  - Promote Multi-Benefit Projects

The hydrology discussed in this attachment was used as the basis for the hydrologic and hydraulic modeling performed. Results from the models subsequently enabled assessments of the relative potential of different actions to achieve these goals.

1.5 2012 CVFPP Planning Approaches

In addition to **No Project**, three fundamentally different approaches to flood management were initially compared to explore potential improvements in the Central Valley. These approaches are not alternatives; rather, they bracket a range of potential actions and help explore trade-offs in costs, benefits, and other factors important in decision making. The approaches are as follows:

- **Achieve SPFC Design Flow Capacity** – Address capacity inadequacies and other adverse conditions associated with existing SPFC facilities, without making major changes to the footprint or operation of those facilities.
- **Protect High Risk Communities** – Focus on protecting life safety for populations at highest risk, including urban areas and small communities.

- **Enhance Flood System Capacity** – Seek various opportunities to achieve multiple benefits through enhancing flood system storage and conveyance capacity.

Comparing these approaches helped identify the advantages and disadvantages of different combinations of management actions, and demonstrated opportunities to address the CVFPP goals to different degrees.

Based on this evaluation, a **State Systemwide Investment Approach** was developed that encompasses aspects of each of the approaches to balance achievement of the goals from a systemwide perspective, and includes integrated conservation elements. Figure 1-3 illustrates this plan formulation process.

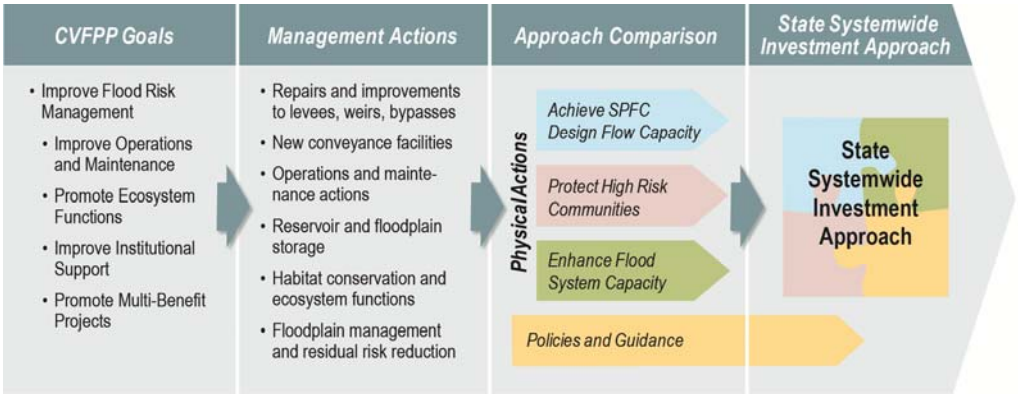

Figure 1-3. Formulation Process for State Systemwide Investment Approach

As described above, this attachment lays the foundations for numerous technical analyses preformed to support the 2012 CVFPP. This attachment does not specifically relate to any of the approaches.

1.6 Basic Assumptions and Limitations

The 2002 Comprehensive Study includes a thorough hydrologic analysis of numerous floodplains and tributaries in the Sacramento and San Joaquin river basins. The 2012 CVFPP includes the same basic assumptions and limitations discussed in the Comprehensive Study.

The Comprehensive Study hydrology may or may not fulfill the technical requirements of site-specific investigations within the Sacramento and San Joaquin river basins. Before using the hydrology for any additional studies, the size and scope of each study, even at the prefeasibility level, will need to be evaluated to determine if the Comprehensive Study hydrology can be directly applied. In most cases, more detailed hydrology will need to be

performed. Assumptions and limitations for the data and analyses used in the Comprehensive Study include the following:

- Data are stationary.
- Natural flow frequency curves are strictly rainflood frequency curves. Snowmelt runoff is not directly incorporated into the analysis.
- Centering hydrographs are predicated on flood runoff, not precipitation. The approach was driven entirely by historical flow data; precipitation was not used in any portion of the methodology.
- Storm runoff centerings were formulated based on the "Composite Floodplain" concept (refer to Section 3 for more details).
- The unregulated frequency curves computed for the Comprehensive Study were created by following procedures outlined in Bulletin 17B (USGS, 1982).
- Travel times and attenuation factors (Muskingum coefficients) are fixed for all simulated exceedence frequencies.
- Mainstem unregulated flow frequency curves were designed to quantify the total flows that the basins produced in rain floods, not the average natural flows expected at mainstem locations during any of the synthetic exceedence frequency storm events.
- Patterns for synthetic floods are formulated based on historical storms.

1.7 Report Organization

Organization of this document is as follows:

- Section 1 introduces and describes the purpose of this report, provides background information, and discusses assumptions and limitations used in the study.
- Section 2 briefly describes hydrology in the Sacramento and San Joaquin river basins.
- Section 3 describes the methodology used in the hydrologic analyses.
- Section 4 contains references for the sources cited in this document.
- Section 5 lists acronyms and abbreviations used in this document.

2.0 Hydrology Description

Hydrology used for the 2012 CVFPP encompasses the watersheds of the two major river systems in California's Central Valley, the Sacramento River in the north and the San Joaquin River in the south. The watersheds of these river systems have a combined drainage area of more than 43,000 square miles, an area nearly as large as the state of Florida. The watersheds of the Sacramento and San Joaquin rivers are shown in Figure 2-1.

Because of its climate and geography, flooding is a frequent and natural event in the Sacramento and San Joaquin river basins. Historically, the Sacramento River Basin has been subject to floods that result from winter and spring rainfall as well as rainfall combined with snowmelt. The San Joaquin River Basin has been subject to floods that result from both rainfall that occurs during the late fall and winter months, and unseasonable and rapid melting of the winter snowpack during the spring and early summer months.

Sacramento and San Joaquin river basins hydrologic conditions, such as topography, soils, vegetation, climate, temperature, precipitation, snowpack, and the flood management system, are briefly summarized below from Comprehensive Study Appendix B (USACE and DWR, 2002b).

2.1 Sacramento River Basin

The Sacramento River Basin covers an area of 26,300 square miles (above Rio Vista) and is about 240 miles long and up to 150 miles wide. It is bounded by the Sierra Nevada on the east, the Coast Ranges on the west, the Cascade and Trinity mountains on the north, and the Delta on the south. Major tributaries to the Sacramento River include the Feather and American rivers from the east; Cottonwood, Cache, and Putah creeks from the west; and numerous other smaller creeks flowing into the Sacramento River from both the east and west.

Figure 2-1. Sacramento and San Joaquin River Basin Watershed Map

The following text provides an overview of the hydrologic conditions in the Sacramento River Basin:

- **Topography** varies from flat valley areas and low rolling foothills to steep mountainous terrain (see Table 2-1 for elevation and slope data).

- **Soil cover** is moderately deep. Classifications vary from sands, silts, and clays in the valley areas to porous volcanic areas in the northern end of the basin. In the American and Feather river basins, soils range from alluvial deposits in the valley areas to granitic rock in the upper elevations.

- **Vegetation** in the higher elevations of the Sacramento River Basin is dominated by coniferous forest. The foothills and valley areas are dominated by an oak-brush-grassland environment. Extensive valley areas in the Sacramento River Basin are cultivated for agricultural purposes.

- **Climate** is temperate and varies according to elevation. In the valley and foothill areas, summers are hot and dry and winters are cool and moist.

- Average annual **temperatures** (degrees Fahrenheit (°F)) in the Sacramento River Basin range from the mid-60s in the valley areas to the low 50s at the higher elevations. Temperatures can range from nearly 120°F in the northern valley to below zero in the Sierra Nevada Range.

- Normal annual **precipitation** amounts vary widely throughout the basin, ranging from the low teens in valley areas to 90 inches in some mountain areas. The Sierra Nevada and Coast Ranges have an orographic effect on precipitation. Precipitation increases with altitude, but basins on the east side of the Coast Ranges lie in a rain shadow and receive considerably less precipitation than do basins of similar altitude on the west side of the Sierra Nevada.

- While convective rainfall in the Sierra Nevada can occur in the summer, precipitation is often in the form of **snowpack** at elevations over 5,000 feet above mean sea level (msl) in the Sacramento River Basin during winter and early spring months. Elevations in the northern portion of the Sacramento River Basin reach nearly 14,000 feet above msl in the headwaters of the Sacramento River. Lassen Peak, which exceeds 10,000 feet above msl in the Cascade Range, receives as much as 90 inches of annual precipitation, primarily as snow.

- The basic **flood management system** in the Sacramento River Basin consists of a series of levees and bypasses placed to protect specific areas and take advantage of the natural overflow basins. The flood management system includes levees along the Sacramento River south of Ord Ferry; levees along the lower portion of the Feather, Bear, and Yuba rivers; and levees along the American River. Additionally, the system benefits from three natural drainage basins: Butte, Sutter, and Yolo. These basins run parallel to the Sacramento River and receive excess flows from the Sacramento, Feather, and American rivers via natural overflow channels and over weirs. When the Sacramento River is high, the three basins form one continuous waterway.

Table 2-1. Sacramento River Watershed Topography

Reach	Elevation Range (feet above mean sea level[1])	Slope
Sacramento River Basin above Shasta Dam	1,000 feet to over 14,000 feet	Varies
Sacramento River Basin below Shasta Dam and above Red Bluff	280 feet to approximately 10,000 feet	5 feet per mile
Red Bluff to Ord Ferry	Less than 100 feet to 10,000 feet	1 foot per mile
Ord Ferry to Fremont Weir	Less than 100 feet to 3,000 feet	0.9 feet per mile
Below Fremont Weir	0 feet to 10,000 feet	0.4 feet per mile
Feather and American rivers	Less than 50 feet to 10,000 feet	Varies

Note:
[1] Mean sea level is at 0 feet.

In addition to the leveed system, the flood management system uses reserved flood storage space in selected reservoirs on the Sacramento, Feather, and American rivers and some of their larger tributaries. These reservoirs help to reduce damaging rain flood peaks by holding back floodwaters and, ideally, releasing water into the rivers at a slower rate. Additional information on the flood management system in the Sacramento River basin can be found in the *State Plan of Flood Control Descriptive Document* (DWR, 2010).

2.2 San Joaquin River Basin

The San Joaquin River Basin lies between the crests of the Sierra Nevada and Coast Ranges and extends from the northern boundary of the Tulare Lake Basin, near Fresno, to the Delta, near Stockton, as shown in Figure 2-1. The basin has an area of about 13,500 square miles, as measured at Vernalis, extending about 120 miles from the northern to southern boundaries. Major tributaries to the San Joaquin River include the Fresno, Chowchilla, Merced, Tuolumne, and Stanislaus rivers from the east, and numerous other smaller creeks flowing into the San Joaquin River from both the east and west.

The following text briefly provides an overview of hydrologic conditions in the San Joaquin River Basin:

- **Topography** varies in the San Joaquin River Basin. The Sierra Nevada Range has an average crest elevation of about 10,000 feet above msl with occasional peaks as high as 13,000 feet above msl. Crest elevations of the Coast Ranges reach to about 5,000 feet above msl. The valley area measures about 100 miles by 50 miles and slopes gently from both sides toward a shallow trough somewhat west of the center of the valley. Valley floor elevations range from 250 feet above msl at the south to near sea level at the Delta. The trough forms the channel for the lower San Joaquin River and has an average slope of about 0.8 feet per mile between the Merced River and Paradise Cut in the Delta.

- **Soils** in the valley basin bottoms are poorly drained and fine textured. Some areas are affected by salts and alkali and require reclamation before they are suitable for crops. Bordering and just above the basin are soils of the fans and floodplains. These soils are generally level, very deep, well drained, nonsaline and nonalkaline, and well suited to a wide variety of crops. The soils of the terraces bordering the outer edges of the valleys generally are of poorer quality with dense clay subsoils or hardpans at shallow depths. These soils generally support pasture and rangeland.

- **Vegetation** types include cultivated crops and pasture grasses, and forbs, hardwood forests, chaparral mountain brush, and coniferous forests. The distribution of these vegetation types is primarily a function of elevation, with cultivated crops located almost entirely in valley floor areas, hardwood forests and chaparral brush located at mid-elevations, and coniferous forests at the higher elevations.

- **Climate** is characterized by wet, cool winters; dry, hot summers; and somewhat wide variations in relative humidity. In the valley area, relative humidity is very low in summer and high in winter. At higher elevations, summers are warm and slightly moist and winters are cold and wet, with significant snow accumulations at higher elevations.

- **Temperatures** vary considerably because of seasonal changes and the large range of elevation. Temperatures in the lower elevations are normally above freezing but range from slightly below freezing during the winter to highs of more than 100°F during the summer. At intermediate and high elevations, the temperature may remain below freezing for extended periods during the winter.

- Normal annual **precipitation** in the basin varies from 6 inches on the valley floor near Mendota to about 70 inches at the headwaters of the San Joaquin River. Most of the precipitation occurs during from November through April. Precipitation is negligible during the summer months, particularly on the valley floor. Similar to the Sacramento River Basin, the Sierra Nevada and Coast Ranges have an orographic effect on precipitation in the San Joaquin River Basin. Precipitation increases with altitude, but basins on the east side of the Coast Ranges lie in a rain shadow and receive considerably less precipitation than do basins of similar altitude on the west side of the Sierra Nevada.

- Precipitation is often in the form of **snowpack** at elevations over 5,000 feet above msl in the San Joaquin River Basin during winter and early spring months. Ground surface elevations in southern portions of the San Joaquin River Basin reach nearly 14,000 feet above msl in the headwaters of the San Joaquin River. These higher elevations relative to the northern Sierra Nevada mean that peak snowmelt lasts longer into the growing season in the San Joaquin River Basin.

- The **flood management system** includes leveed sections along the San Joaquin River; levees along the lower portions of Ash and Berenda sloughs; Bear Creek; and the Fresno, Stanislaus, and Calaveras rivers. The Chowchilla Canal Bypass diverts excess San Joaquin River flow and sends it to the Eastside Bypass. In addition to Chowchilla Canal Bypass flow, the Eastside Bypass intercepts flows from minor tributaries and rejoins the San Joaquin River between Fremont Ford and Bear Creek. Channel capacity on the San Joaquin River decreases moving downstream until its confluence with the Merced River, where San Joaquin River channel capacity then begins to increase. The San Joaquin River levee and diversion systems are not designed to contain the objective release (maximum allowable flow downstream from a reservoir before the beginning of flooding) from each of the project

reservoirs simultaneously. Flows in the San Joaquin River that are less than design flow for a given reach may still cause damage to levees in that reach.

In addition to the leveed system, the flood management system uses reserved flood storage space in selected reservoirs on the San Joaquin, Fresno, Chowchilla, Merced, Tuolumne, and Stanislaus rivers and some of their larger tributaries. These reservoirs help to reduce damaging rain flood peaks or snowmelt by holding back floodwaters and, ideally, releasing water into the rivers at a slower rate. Additional information on the flood management system in the San Joaquin River Basin can be found in the *State Plan of Flood Control Descriptive Document* (DWR, 2010).

The San Joaquin River Basin also receives floodflows from the Tulare Lake Basin. The Kings River weirs divert floodflows north via the Kings River North, James Bypass, Fresno Slough, and Mendota Pool system into the San Joaquin River Basin. Flows greater than as specified in flood management operating policies are sent into the Tulare Lake Basin via Kings River South.

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3.0 Hydrologic Analyses

This section summarizes the methodology used during the Comprehensive Study to prepare flood runoff centerings and flood hydrographs that feed into reservoir system (hydrologic) and hydraulic models; those simulations culminated in delineation of floodplains and estimates of potential flooding damages.

As described in Section 1, a subset of the methods and findings from the Comprehensive Study were used for the 2012 CVFPP. For additional details regarding the Comprehensive Study hydrologic analysis, refer to Comprehensive Study Appendix B (USACE and DWR, 2002b).

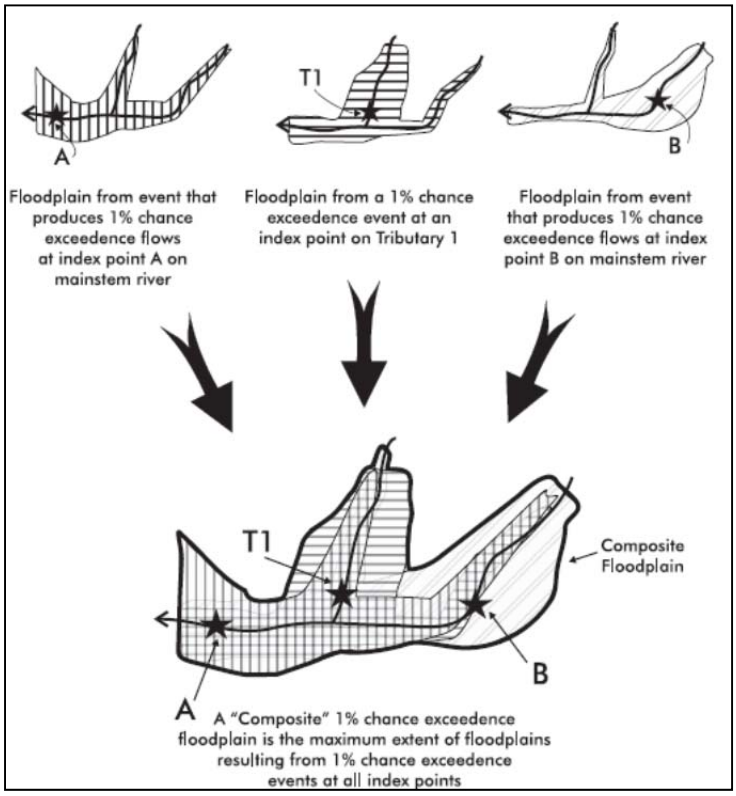
Synthetic 50, 10, 4, 2, 1, 0.5, and 0.2 percent AEP storms were developed for the Comprehensive Study. The seven synthetic AEP storms provided a basis for defining existing conditions, analyzing alternatives, and plan formulation. The Comprehensive Study hydrologic analysis, completed by the Water Management Section of the U.S. Army Corps of Engineers (USACE), included following:

- Updated natural flow frequency curves were prepared for locations within the basins
- Retrospective of historical floods that have impacted Central Valley rivers and synthetic flood runoff centerings were developed to represent flood events of a specific exceedence frequency
- Seven synthetic exceedence frequency flood hydrographs were developed

3.1 Composite Floodplain

The Comprehensive Study hydrologic analysis used the “Composite Floodplain” concept, which recognizes that the floodplains generated through modeling of the seven synthetic exceedence frequency events were not created by a single flood event, but by a combination of several events, each of which shaped the floodplain at different locations. This concept is illustrated in Figure 3-1 and further described in *Hydraulic Technical Documentation of the Comprehensive Study* (Comprehensive Study Appendix D) (USACE and DWR, 2002c). Moving downstream in a watershed, a Composite Floodplain becomes increasingly complex. With

the confluence of each additional tributary, the number of possible scenarios of flow combinations that could shape the floodplain grows. The role of tributaries in shaping floodplains individually and as a system is the foundation of the Composite Floodplain concept and a cornerstone of the Comprehensive Study's hydrologic analysis. The synthetic hydrology was developed so that the Composite Floodplain represents the maximum extent of inundation possible at all locations for any simulated synthetic exceedence frequency storm events.



Source: USACE and DWR 2002b

Figure 3-1. Composite Floodplain Concept

3.2 Study Approach

The Comprehensive Study's hydrologic analysis investigated three fundamental subjects during the formulation of synthetic flood events:

1. Amount of runoff produced during each of the seven synthetic AEP storms.

2. Contribution of individual tributaries to this total volume.

3. Translation of these flood volumes and distributions to hourly time series for input into a reservoir simulations model.

3.3 Analysis

Unregulated frequency curves were developed at key mainstem and tributary locations in both the Sacramento and San Joaquin river basins in the Comprehensive Study. Unregulated frequency curves plotted historical points and statistical distributions of unimpaired flows (no reservoir influence). Curves displayed volumes or average flow rates for different time durations over a range of AEPs. These curves were used to translate (1) hydrographs to frequencies (e.g., in 1997, the 3-day natural inflow to Friant Dam was roughly 50,000 cubic feet per second (cfs), which translates to a 1.54 percent AEP storm), and (2) frequencies to flood volumes (e.g., according to the curves, the 3-day natural inflow to Friant Dam associated with an annual 10 percent AEP storm is approximately 20,000 cfs). After a curve was developed, runoff volume for any of the seven synthetic exceedence frequency flood events could be obtained from the plot for that curve's specific location.

3.3.1 Methodology for Deriving Unregulated Frequency Curves

The unregulated frequency curves computed for the Comprehensive Study were created by following the procedures outlined in Bulletin 17B, *Guidelines for Determining Flood Flow Frequency* (USGS, 1982). This report directs federal agencies to use the procedures included therein for all "planning activities involving water and related land resources." Bulletin 17B requires the use of a Pearson Type III distribution with log transformation of the data (Log Pearson Type III distribution) as the method to analyze flood flow frequency.

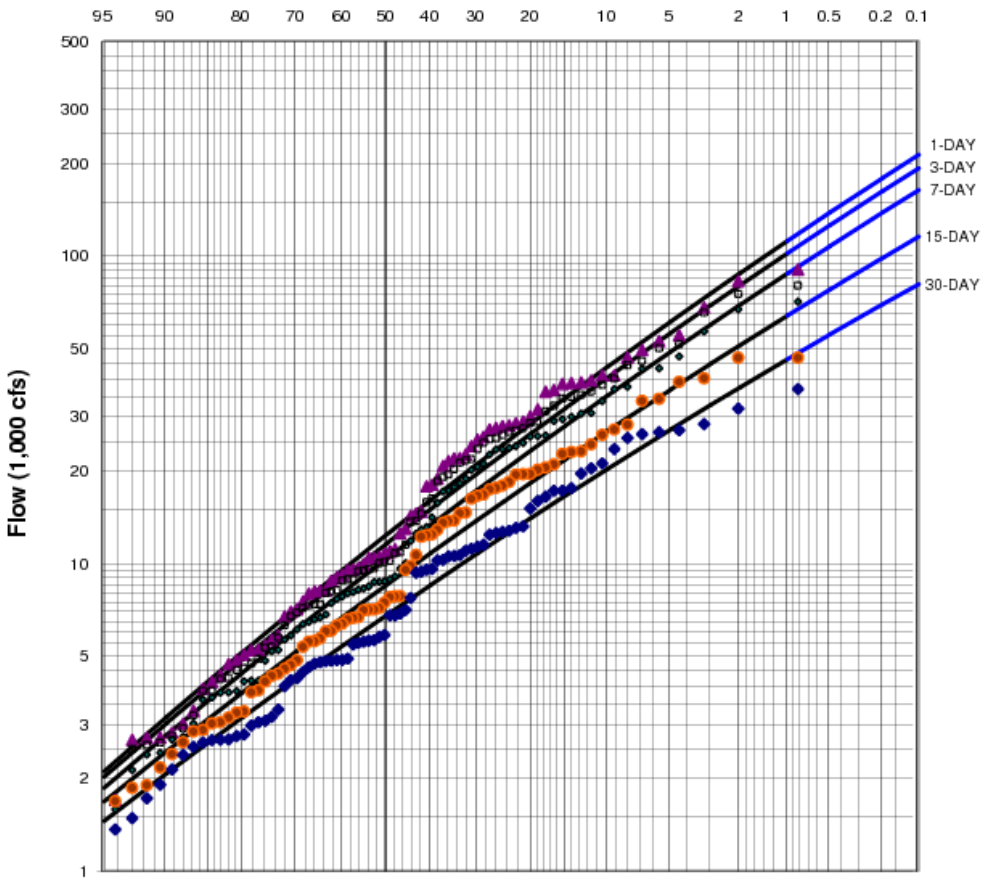
Development of the unregulated frequency curves for the tributaries required daily natural flow data for all target locations. (This development of data is shown in Attachment B.1 to Comprehensive Study Appendix B.) Most of the data were obtained from USACE archives, U.S. Geological Survey (USGS) publications, Central Valley, federal, and other water agencies (including the U.S. Department of the Interior, Bureau of Reclamation, Oroville-Wyandotte Irrigation District, South Sutter Water District, Placer County Water Association, Nevada Irrigation District, Surface Water Data Inc., Southern California Edison, Sacramento Metropolitan Utility District, and Pacific Gas and Electric Company). Data from tributaries were routed to downstream locations for use in constructing mainstem "index" frequency curves.

Unregulated frequency curves for the Sacramento and San Joaquin river basins used for the Comprehensive Study are documented in Attachment B.2 to Comprehensive Study Appendix B. These curves were derived from a statistical analysis of the recorded data after the data had been transformed to log values. The mean, standard deviation, and skew of the log-transformed data were computed for each stream gage or reservoir. The data were screened for high and low outliers and, if found, adjustments to the statistics were computed as outlined in Bulletin 17B. In addition, the resulting statistics were reviewed and sometimes adjusted or smoothed to account for sampling error differences among the various durations, or after comparison with similar gages in a watershed or region. Each frequency curve also plots historical flood events, given their estimated frequency. Determination of a historical event using a frequency plot is described in Comprehensive Study Appendix B (USACE and DWR, 2002b).

Unregulated frequency curves were prepared for 8 mainstem locations and 43 tributary locations (i.e., 51 curve sets), as shown in Attachment B.2 to Comprehensive Study Appendix B. An example of an unregulated frequency curve is shown in Figure 3-2. In all cases, curves were developed or updated to reflect post-1997 hydrology. For any location, the amount of runoff volume produced during simulation of any of the seven synthetic exceedence frequency flood events can be read from the family of best-fit curves or computed directly from the final statistical distribution of each duration. For example, the 1 percent AEP (100-year) 30-day runoff volume for the San Joaquin River near Newman, shown in Figure 3-2, can be determined by reading the average flow of 46,000 cfs, multiplying by the number of seconds in 30 days, and dividing by 43,560 to get 2.7 million acre-feet.

The approach formulated for the hydrologic analysis was driven entirely by historical flow data. Each year of record included the influence of snowmelt, infiltration, interception, precipitation distribution, timing of runoff, storm development characteristics, and physical basin attributes for that annual rain flood event. Historical flow data records provided a sufficient sample of flood events to characterize hypothetical flood volumes and tributary system relationships.

No synthetic precipitation events were required in development of the Comprehensive Study hydrology; precipitation was not used in any portion of the methodology.



ADOPTED STATISTICS:

	Mean	Std.Dev.	Skew
1-day	4.079	0.445	-0.2
3-day	4.052	0.439	-0.2
7-day	4.002	0.431	-0.2
15-day	3.918	0.407	-0.2
30-day	3.820	0.387	-0.2

NOTES:

1. No overbank flow.
2. Median plotting positions.
3. Drainage area: 9,520 sq. mi.
4. Period of record: 1917-1998.

SACRAMENTO-SAN JOAQUIN COMPREHENSIVE STUDY
SAN JOAQUIN RIVER BASIN, CALIFORNIA

RAIN FLOOD FREQUENCY CURVES
SAN JOAQUIN RIVER NEAR NEWMAN
UNREGULATED CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Source: USACE and DWR 2002b

Figure 3-2. Example Rain Flood Frequency Curves – San Joaquin River near Newman

3.3.2 Historical Flood Event Analysis

The historical flood event analyses described in Comprehensive Study Appendix B were based on natural flow data analysis, which resulted in the compilation of the 51 curve sets (8 mainstem and 43 tributary) that quantified flood volumes at discrete locations within the Sacramento and San Joaquin river basins. At mainstem locations, total volumes reflected the combined flows of upstream tributaries. To perform simulations with reservoir and hydraulic models, this total volume needed to be redistributed into the system of tributaries through a flood pattern.

In nature, storms trigger high flows on large-scale river systems and isolated tributaries as a function of storm structure, air temperature, water content, storm path, orographic influence, basin alignment, and many other geophysical and meteorological variables. Ultimately, all storms are unique, but certain dynamics are common to a variety of storm types, especially those that trigger productive (in terms of volume) events in the Sacramento and San Joaquin river basins. Development of patterns is possible through a number of methods, including random generation, use of a singular historical event, and uniform or ramped concurrencies. The most realistic patterns for synthetic floods are formulated based on historical storms. A detailed analysis of several events was undertaken to identify flood trends and distributions that could be incorporated into generalized patterns.

3.3.3 Retrospective of Historical Flood Events

Nineteen historic flood events were analyzed for the Comprehensive Study. These events were chosen based on the natural 3-day rain flood volumes produced at Sacramento and San Joaquin river basins flood management reservoirs. On a project-by-project basis, any event that was both the largest 3-day natural flow experienced during that water year and one of the five largest 3-day natural flows in the gage history of that project was selected for analysis. Although this selection process focused on tributary events, often the same year was selected for multiple projects. This was especially true for the largest flood years on record (i.e., 1956, 1986, and 1997). Therefore, the 19 storms represented a mixed population of storms that focused on individual tributaries as well as storms that had a powerful systemwide effect.

For each year, a time window was set that contained both the tributary event, which had been selected for inclusion that year, and that provided additional time to allow the storm pattern to complete its influence throughout the basin. Duration flows (1-, 3-, 7-, 15-, and 30-day average flows) within this event window were analyzed for all several mainstem locations and significant tributaries. These flows were translated into

annual percent chance exceedence values based on the unregulated flow and index frequency curves developed for mainstem and tributary locations during the natural flow analysis.

By comparing AEPs instead of flow rates, the distribution of storm patterns is normalized spatially. Percent chance exceedences provide a consistent measure of intensity from basin to basin, while flow rates, as a function of drainage area, alignment, and other factors, are tributary-specific. Investigating chance exceedences clarifies patterns regarding how individual storm systems impact a system of tributaries. Considering multiple storm events highlights trends linking tributary responses and orographic influence in rare events that form the basis for, and can be incorporated into, the development of generalized storm patterns.

3.3.4 Flood Matrix

All AEP storms, locations of interest, flood durations, and year of event were tabulated into Sacramento River Basin and San Joaquin River Basin storm matrices, referred to jointly as the "Matrix," as shown in Comprehensive Study Appendix B, Attachment B. 3. The Matrix, a valuable product of the Comprehensive Study, includes the 19 historical flood events analyzed for comparison of runoff for all major tributaries in a complex hydrologic system. The matrices are presented upstream to downstream, allowing storm and tributary dynamics to be reviewed in diverse permutations of flood durations, storm combinations, and tributary sets.

The Comprehensive Study matrix investigations pointed to several trends that were eventually incorporated into the synthetic flood runoff centering, such as the presence of spatial trends and storm "bull's eyes" within individual storm events. Bull's eyes were created as historical storms impacted certain spatial areas with greater intensity than surrounding areas. Nearly all events in the Matrix displayed some sort of spatial trend or bias toward a specific area.

Mainstem locations below these bull's eyes experienced greater exceedence frequencies because at those locations the intensity of flooding was a function of all upstream tributaries, not just those that were especially intense. In this sense, the mainstem acted as a buffer that absorbed and moderated localized extremes.

A key finding was that orographic effects were most pronounced in the rarest, least frequently occurring events. Orographic effects in the Sacramento River Basin were definitely visible, but not as well defined as those in the San Joaquin River Basin. It is likely that the more pronounced orographic influence in the southern Central Valley is related to the average

ridge crest elevation along the Sierra Nevada, which is generally lower in the Sacramento River Basin than in the San Joaquin River and Tulare Lake basins; however, this remains uncertain.

The Matrix also showed that natural dynamics are highly variable. Storm cells nested within a larger storm structure are powerful and can trigger individual tributary flooding. Even with supporting evidence for orographic influence, there are Matrix examples of floods that demonstrate a consistently opposite bias; in the San Joaquin River Basin during the March 1995 floods and in the Sacramento River Basin during the 1983 floods, annual percent chance exceedences for foothill tributaries were lower than for neighboring higher basins.

3.4 Synthetic Flood Runoff Centering

The Comprehensive Study's guidelines for flood runoff centering were formulated using the trends identified in the historical storm analysis and the Composite Floodplain concept. A flood runoff centering is defined simply as a set of synthetic exceedence frequencies assigned to a mainstem and/or set of tributaries. As described in Comprehensive Study, Appendix B, centerings were developed separately for the Sacramento and San Joaquin river basins; each tributary was included in all centerings within its basin.

Two basic types of flood runoff centerings were analyzed. The first type consists of basin-wide flood events (mainstem centerings), which are significant on a regional basis and produce large runoff volumes throughout the system. The second type is tributary-specific floods (tributary centerings), which generate extremely large floods in individual rivers, but are not widespread enough to produce the runoff volumes typical of basin-wide events.

3.4.1 Mainstem Flood Runoff Centering

Mainstem centerings were designed to stress widespread areas. Index frequency curves were prepared for the mainstem centerings. These curves provide the hypothetical volumes that a basin will produce during simulations of each of the seven synthetic exceedence frequency flood events. The role of the mainstem centerings was to distribute these volumes back into a basin, tributary by tributary, in accordance with patterns visible in historical flood events. Once the volume was distributed, it was translated into hydrographs and routed through reservoir simulation models to produce the seven synthetic exceedence frequency regulated hydrographs needed to construct floodplains throughout the Sacramento and San Joaquin river basins. Table 3-1 gives an example of a mainstem

flood runoff centering and shows the coincident AEP for flows at various locations.

Table 3-1. Example Mainstem Flood Runoff Centering – Sacramento River at Latitude of Ord Ferry

Storm Centering	Flood Event (percent AEP)						
	50%	10%	4%	2%	1%	0.50%	0.20%
Sacramento River at Shasta	81.97	16.92	5.71	2.41	1.25	0.65	0.28
Clear Creek at Whiskeytown	61.73	15.04	9.03	5.61	2.92	1.52	0.65
Cow Creek near Millville	61.73	13.53	8.02	3.89	2.02	1.05	0.45
Cottonwood Creek near Cottonwood	61.73	15.04	9.03	5.61	2.92	1.52	0.65
Battle Creek Below Coleman Fish Hatchery	61.73	13.53	8.02	3.89	2.02	1.05	0.45
Mill Creek near Los Molinos	87.72	**15.04**	7.22	5.94	3.10	1.61	0.69
Elder Creek near Paskenta	87.72	19.34	12.50	10.10	5.26	2.74	1.17
Thomes Creek at Paskenta	87.72	19.34	12.50	10.10	5.26	2.74	1.17
Deer Creek near Vina	87.72	15.04	7.22	5.94	3.10	1.61	0.69
Big Chico Creek near Chico	87.72	15.04	7.22	5.94	3.10	1.61	0.69
Stony Creek at Black Butte	87.72	19.34	12.50	10.10	5.26	2.74	1.17
Butte Creek near Chico	87.72	15.04	10.20	8.42	4.39	2.28	0.97
Feather River at Oroville	87.72	19.34	9.62	8.42	4.39	2.28	0.97
Yuba River at New Bullards Bar	87.72	19.34	11.76	9.18	4.78	2.49	1.06
Yuba River at Englebright	87.72	19.34	11.76	9.18	4.78	2.49	1.06
Deer Creek near Smartsville	87.72	19.34	11.76	9.18	4.78	2.49	1.06
Bear River near Wheatland	87.72	19.34	12.03	10.10	5.26	2.74	1.17
Cache Creek at Clear Lake	87.72	19.34	18.05	12.63	6.58	3.42	1.46
North Fork Cache Creek at Indian Valley	87.72	19.34	18.05	12.63	6.58	3.42	1.46
American River at Folsom	87.72	19.34	14.29	12.63	6.58	3.42	1.46
Putah Creek at Berryessa	87.72	19.34	18.05	12.63	6.58	3.42	1.46

Source: USACE and DWR 2002b
Note:
The values listed for each index point and flood event represent the percent chance of occurrence in any year. For example, during a 10 percent AEP storm centered at Ord Ferry, concurrent flows would be experienced on Mill Creek that correspond to about a 15 percent AEP storm at Mill Creek near Los Molinos (bold).
Key:
AEP = annual exceedence probability

Mainstem centerings reflect a generalized flood pattern based on a number of historical events. Through incorporation of multiple floods into one characteristic pattern, relationships between tributaries become more stable and the influence of powerful, but isolated, storm cells is downplayed.

Characteristic patterns were developed for each mainstem location. When available, historical events that showed flood bull's-eyes in the watershed

above the mainstem location of interest were used to formulate synthetic patterns. The orographic effects noted in the Matrix analysis were also incorporated, especially for the largest, less frequently occurring synthetic exceedence frequency events.

To develop patterns consistently, guidelines for mainstem pattern construction were formulated and are presented in Table 7 of Comprehensive Study Appendix B. After an initial pattern was formulated in the Comprehensive Study, hydrographs were constructed at tributary locations (in accordance with the pattern) and routed back to the mainstem location with the same procedure used during construction of the index frequencies, as shown in Attachment B.4 of the Comprehensive Study. Duration maxima (1-, 3-, 7-, 15-, and 30-day) were computed for the mainstem hydrograph and compared with average flows from the index curve. The initial pattern was then increased or decreased by a fixed percentage and the comparison process was repeated. This iterative procedure continued until the final centering produced flood volumes at the mainstem location that were roughly equal to the hypothetical volumes specified by the index curves.

3.4.2 Tributary Flood Runoff Centering

Tributary centerings were designed to stress individual tributary systems. Whereas the mainstem centerings were formulated as spatially distributed events that were productive on a systemwide basis, tributary centerings were designed to simulate extreme floods on individual rivers generated by storm systems that were not widespread enough to produce runoff volumes typical of basin-wide events. In this sense, tributary centerings seek to reflect the powerful and isolated storm cells intentionally downplayed by the mainstem centerings. Development of tributary centering is further described in Comprehensive Study Appendix B (USACE and DWR, 2002b).

Once a tributary centering was prepared, it was deemed complete pending a test that translated centerings to hydrographs and routed tributary flows to the nearest downstream index curve location. Duration maxima (1-, 3-, 7-, 15-, and 30-day) were then computed for each of the resultant seven synthetic exceedence frequency natural flow hydrographs and compared with average flows from the corresponding index frequency curves. For each tributary centering, it was confirmed that the flows experienced at the mainstem points were lower than those generated by the corresponding mainstem centering. This affirmed that the floodplains in mainstem locations were more likely to be shaped by the widespread floods simulated with mainstem centerings.

3.4.3 Development of Seven Synthetic AEP Storm Natural Flow Hydrographs

Storm frequencies, described above, needed to be translated to hourly flood flow hydrographs for use in reservoir simulations and hydraulic modeling. The Comprehensive Study's translation process involved three steps:

1. Obtaining average floodflow rates from unregulated tributary frequency curves.

2. Separating these average flows into wave volumes.

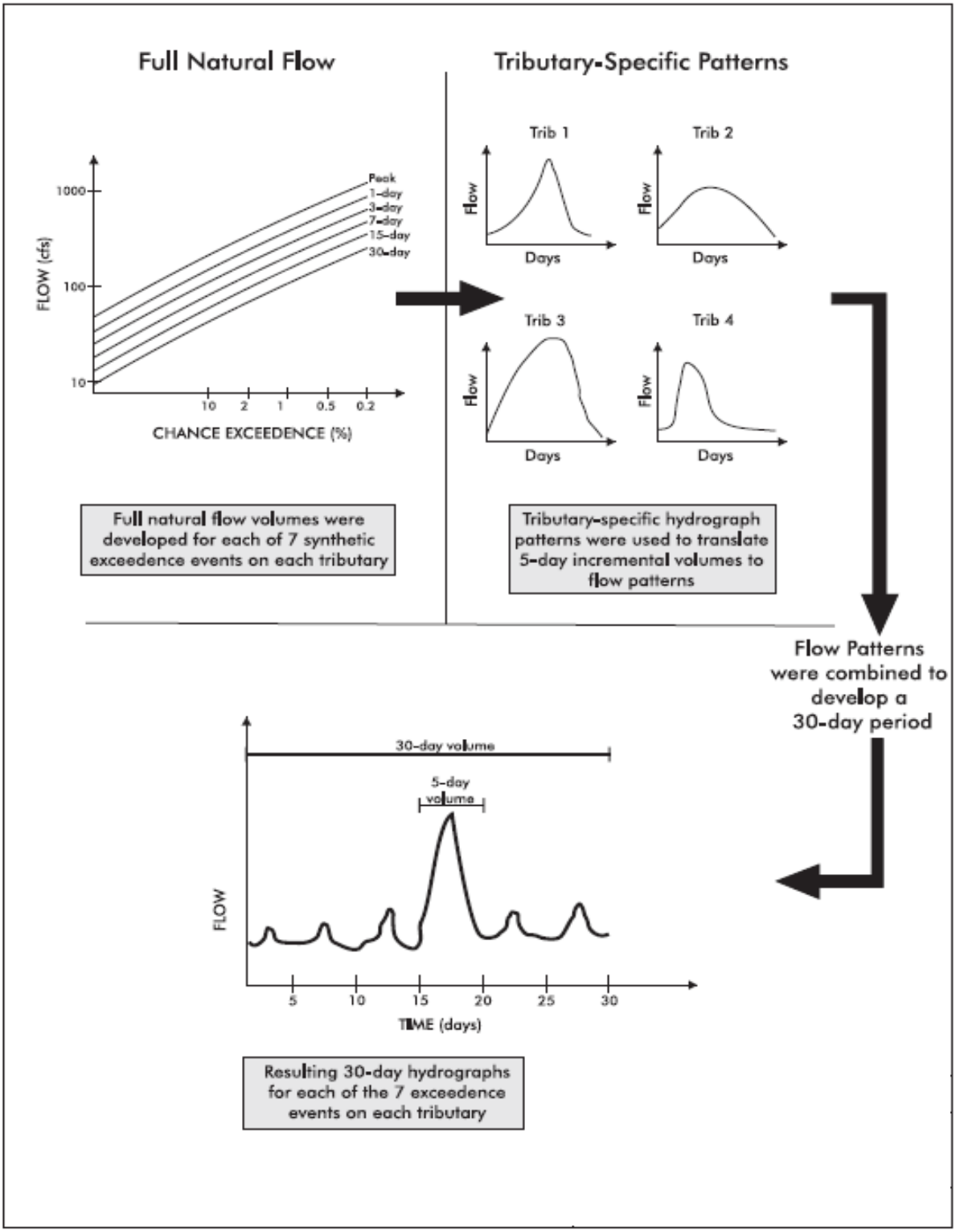
3. Combining and distributing volumes into a six-wave series of 5-day waves covering a 30-day flood period.

This process was performed only at tributary locations; mainstem flood hydrographs always resulted from the routed contributions of upstream tributaries. The process is illustrated in Figure 3-3 and Plate 4 in Comprehensive Study Appendix B (USACE and DWR, 2002b).

Values from the unregulated frequency curves represented the average flow anticipated over a specific time interval. For instance, the 5-day value was the average flow expected during the highest 5 days of flooding during any of the seven synthetic exceedence events. Likewise, the 10-day value was the average over the highest 10 days of flooding, etc. Although not always the case, it was typical for the highest 5-day period to be part of the highest 10-day period as well as part of the highest 15-day, 20-day, and other periods.

Flood volumes were computed by multiplying average flows by their respective durations. These values represented the total volumes of water anticipated during the highest 5, 10, 15, 20, 25, or 30 days of flows. The volumes were portioned into time segments by subtracting volumes of the shorter durations from the next longer duration (i.e., 5-day volume is subtracted from 10-day volume to calculate the volume produced between the extents of the 5-day and 10-day periods. This procedure was repeated for the 10-, 15-, 20-, 25-, and 30-day durations and resulted in a set of seven synthetic exceedence frequency flood volumes produced by a tributary. These seven volumes were treated as wave volumes in a series of six 5-day waves.

In the Comprehensive Study, the basic pattern of all synthetic flood hydrographs was a 30-day hourly time series consisting of six waves, each 5 days in duration. Volumes were ranked and distributed into a basic pattern. The highest wave volume was always distributed into the fourth, or main, wave. The second and third highest volumes preceded and followed

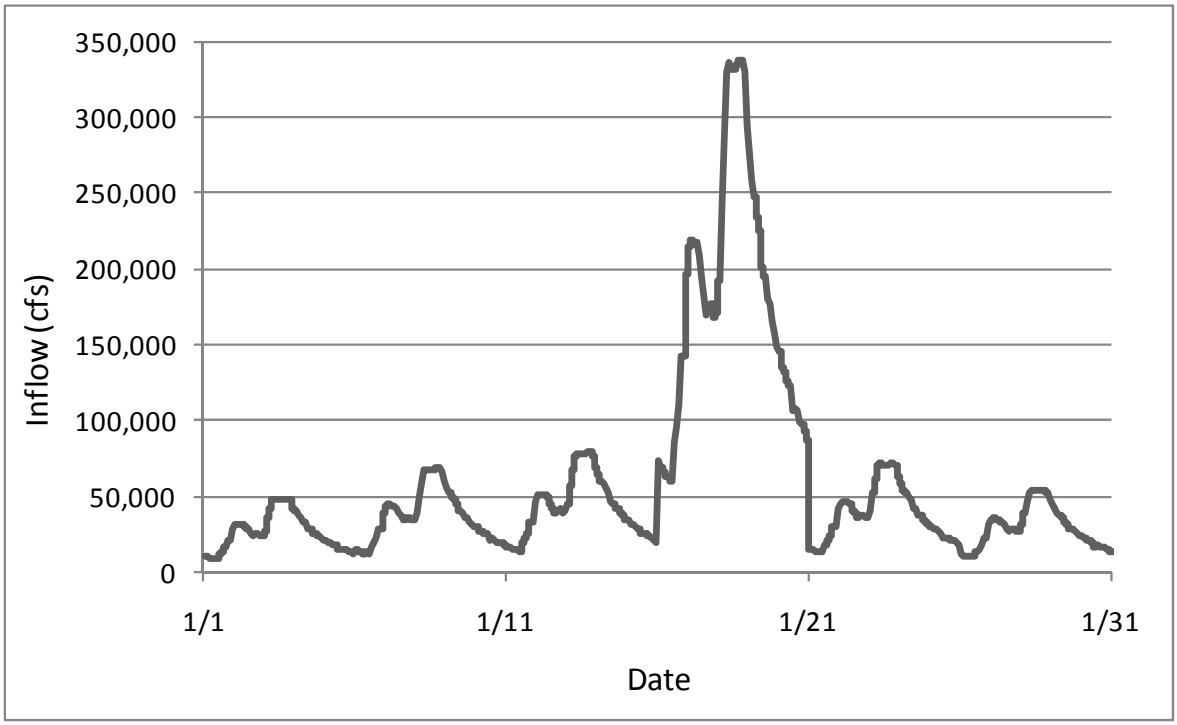


Source: USACE and DWR 2002b

Figure 3-3. Example Synthetic Flood Hydrograph Construction

the main wave, respectively. The fourth highest volume was distributed into the second wave and the fifth highest was distributed into the final of the six waves. The sixth and smallest wave volume was distributed into the first wave of the series. The shape of each wave was identical and the magnitude was determined by the total volume that the wave needed to convey. Figure 3-4 is an example of a synthetic flood hydrograph for inflow into Oroville.

The distribution of tributary flood volumes into 5-day wave patterns was automated using the same spreadsheet that translated frequencies to average flows. Hydrographs were automatically computed and copied into text files for direct entry into the USACE Hydrologic Engineering Center's data storage system used to hold input data for the reservoir operations and hydraulic models.



Source: USACE and DWR 2002b

Figure 3-4. Example Synthetic Flood Hydrograph – 1 Percent AEP Inflow to Oroville

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5.0 Acronyms and Abbreviations

°F ..... degrees Fahrenheit

AEP ..... annual exceedence probability

Board ..... Central Valley Flood Protection Board

cfs ..... cubic feet per second

Comprehensive Study 2002 Sacramento and San Joaquin River Basins Comprehensive Study

Comprehensive Study Appendix B Appendix B, Synthetic Hydrology Technical Documentation, the 2002 Sacramento and San Joaquin River Basins Comprehensive Study

Comprehensive Study Appendix D Hydraulic Technical Documentation of the Comprehensive Study

CVFPP ..... Central Valley Flood Protection Plan

Delta ..... Sacramento-San Joaquin Delta

DWR ..... California Department of Water Resources

msl ..... mean sea level

SPFC ..... State Plan of Flood Control

State ..... State of California

USACE ..... U.S. Army Corps of Engineers

USGS ..... U.S. Geological Survey

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STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

Attachment 8B

Reservoir Analysis

CENTRAL VALLEY
FLOOD MANAGEMENT
PLANNING PROGRAM

FloodSAFE
CALIFORNIA

Public Draft

2012 Central Valley Flood Protection Plan

Attachment 8B: Reservoir Analysis

January 2012

STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

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1.0 Introduction

This section provides the purpose of this attachment, background information (including planning areas, goals, and approaches), an overview of flood management in the Central Valley, past and ongoing reservoir operations studies, and report organization.

1.1 Purpose of this Attachment

As part of preparation of the Central Valley Flood Protection Plan (CVFPP), potential management actions were developed for flood management in the Central Valley; these management actions were evaluated and combined into various approaches. One of the management actions considered for the 2012 CVFPP was to increase flood management flexibility in major multipurpose reservoirs in the Sacramento and San Joaquin river basins. This flexibility could be accomplished through a variety of methods such as changes to reservoir operational criteria, construction of new reservoirs, or physical modifications to existing reservoirs. For the 2012 CVFPP, only changes in reservoir operational criteria (i.e., flood storage allocation and objective release) were considered to provide downstream flood management benefits for this reconnaissance-level analysis.

Because the potential to realize flood management benefits from changing reservoir operational criteria was uncertain, the 2012 CVFPP Reservoir Analysis was performed to first determine if there was any opportunity associated with operational criteria changes. The objective of the analysis described in this attachment was to demonstrate whether there is any potential improvement in systemwide flood management (e.g., lower downstream peak flood stage) from changes to reservoir operational criteria. Results from this analysis provide insight for more detailed and coordinated studies to explore operational criteria changes.

Implementing reservoir operational criteria changes for real-world operations is complicated and has wide-spread implications. Because most of the flood management reservoirs in the Central Valley are operated for multiple purposes, changing operational criteria for flood management benefits may have unintended effects on other reservoir purposes (e.g., water supply, hydropower). In addition, changes to the operational criteria of an individual reservoir can affect how other reservoirs operate. The complicated and interconnected nature of these flood management

reservoirs makes it imperative that willing reservoir owners and operators, and the U.S. Army Corps of Engineers (USACE), who have jurisdiction over reservoir flood operations, coordinate. Any changes would also require coordination among ongoing reservoir studies such as the California Department of Water Resources' (DWR) existing Forecast-Coordinated Operations (F-CO) Program, planned Forecast-Based Operations (F-BO) Program, and ongoing System Reoperation Program. In addition, to implement such changes would require a detailed project-level feasibility study to evaluate effects on other reservoir purposes, followed by significant administrative actions.

Therefore, because of the preliminary and exploratory nature of the 2012 CVFPP Reservoir Analysis, it is an initial assessment of potential reservoir-related opportunities to support the 2012 CVFPP development. This analysis does not propose any specific changes to current reservoirs operations be made or suggest that these changes are the only options for modifying operational criteria. The 2012 CVFPP Reservoir Analysis is a preliminary analysis of opportunities and effects with a systemwide perspective, and future studies are needed to more thoroughly consider other potential effects (e.g., water supply, environmental, hydropower) and the feasibility of modifying operational criteria at individual reservoirs.

For modeling purposes, this preliminary analysis considered a few potential scenarios to improve systemwide flood management flexibility, which were included in the Enhance Flood System Capacity Approach for the 2012 CVFPP (see Section 1.5, below). Reservoir operational criteria changes were ultimately not moved forward into the State Systemwide Investment Approach because of: (1) the preliminary nature of this analysis; (2) uncertainty associated with the potential effects of reservoir operational criteria changes; and (3) the need to coordinate with operators and owners on more detailed, reservoir-specific analyses. An exception is the already authorized operational changes associated with the Folsom Dam Raise, which are included in both the No Project condition and State Systemwide Investment Approach.

1.2 Background

As authorized by Senate Bill 5, also known as the Central Valley Flood Protection Act of 2008, the DWR has prepared a sustainable, integrated flood management plan called the CVFPP, for adoption by the Central Valley Flood Protection Board (Board). The 2012 CVFPP provides a systemwide approach to protecting lands currently protected from flooding by existing facilities of the State Plan of Flood Control (SPFC), and will be updated every 5 years.

As part of development of the CVFPP, a series of technical analyses were conducted to evaluate hydrologic, hydraulic, geotechnical, economic, ecosystem, and related conditions within the flood management system and to support formulation of system improvements. These analyses were conducted in the Sacramento River Basin, San Joaquin River Basin, and Sacramento-San Joaquin Delta (Delta).

1.3 CVFPP Planning Areas

For planning and analysis purposes, and consistent with legislative direction, two geographical planning areas were important for CVFPP development (Figure 1-1):

- **SPFC Planning Area –** This area is defined by the lands currently receiving flood protection from facilities of the SPFC (see *State Plan of Flood Control Descriptive Document* (DWR, 2010b)). The State of California's (State) flood management responsibility is limited to this area.

- **Systemwide Planning Area –** This area includes the lands that are subject to flooding under the current facilities and operation of the Sacramento-San Joaquin River Flood Management System (California Water Code Section 9611). The SPFC Planning Area is completely contained within the Systemwide Planning Area which includes the Sacramento River Basin, San Joaquin River Basin, and Delta regions.

Planning and development for the CVFPP occurs differently in these planning areas. The CVFPP focused on SPFC facilities; therefore, evaluations and analyses were conducted at a greater level of detail within the SPFC Planning Area than in the Systemwide Planning Area.

The 2012 CVFPP Reservoir Analysis focused on major multipurpose reservoirs located within the Systemwide Planning Area. Because this analysis built on the approach, models, and data developed for the Comprehensive Study (USACE, 2002a), the Delta and Mokelumne, Cosumnes, and Calaveras rivers and small streams that enter the Delta were not part of the planning area for the 2012 CVFPP Reservoir Analysis, because they were not a primary focus of the Comprehensive Study. While this analysis did not specifically quantify flood management benefits solely within the SPFC Planning Area, the scenarios were compared using locations that were generally within the SPFC Planning Area.

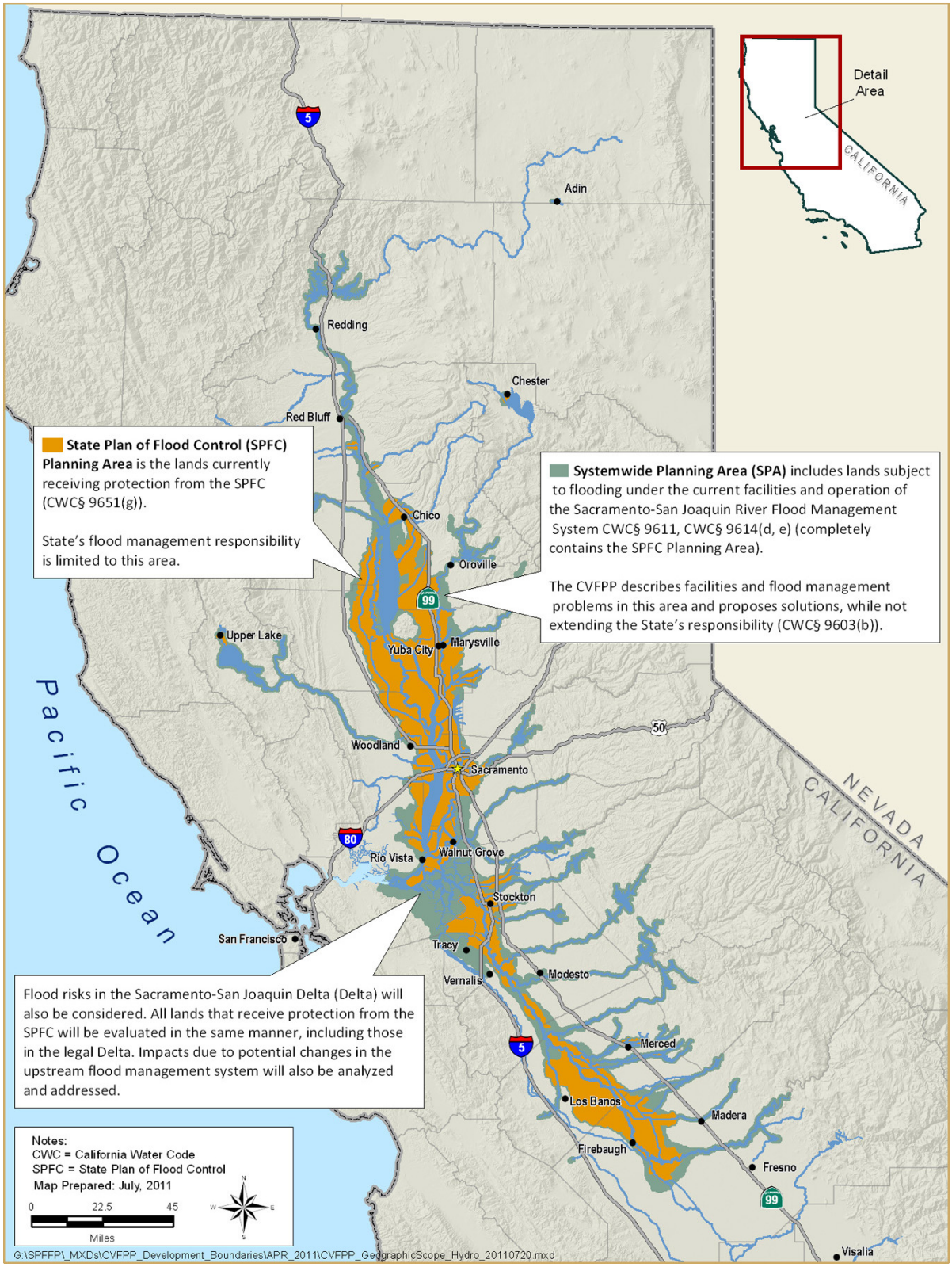

Figure 1-1. Central Valley Flood Protection Planning Areas

1.4 2012 CVFPP Planning Goals

To help direct CVFPP development to meet legislative requirements and address identified flood-management-related problems and opportunities, a primary and four supporting goals were developed:

- **Primary Goal** – Improve Flood Risk Management
- Supporting Goals:
  - Improve Operations and Maintenance
  - Promote Ecosystem Functions
  - Improve Institutional Support
  - Promote Multi-Benefit Projects

The goal of the 2012 CVFPP Reservoir Analysis was to explore the potential to improve flood risk management on a systemwide level by changing reservoir operational criteria to improve operational coordination among the reservoirs, thereby lowering downstream peak water levels.

1.5 2012 CVFPP Planning Approaches

In addition to the **No Project** approach, three fundamentally different approaches to flood management were initially compared to explore potential improvements in the Central Valley. These approaches are not alternatives; rather, they bracket a range of potential actions and help explore trade-offs in costs, benefits, and other factors important in decision making. The approaches are as follows:

- **Achieve SPFC Design Flow Capacity** – Address capacity inadequacies and other adverse conditions associated with existing SPFC facilities, without making major changes to the footprint or operation of those facilities.
- **Protect High Risk Communities** – Focus on protecting life safety for populations at highest risk, including urban areas and small communities.
- **Enhance Flood System Capacity** – Seek various opportunities to achieve multiple benefits through enhancing flood system storage and conveyance capacity.

Comparing these approaches helped identify the advantages and disadvantages of different combinations of management actions, and demonstrated opportunities to address the CVFPP goals to different degrees.

Based on this evaluation, a **State Systemwide Investment Approach** was developed that encompasses aspects of each of the approaches to balance achievement of the goals from a systemwide perspective, and includes integrated conservation elements. Figure 1-2 illustrates this plan formulation process.

The 2012 CVFPP Reservoir Analysis used the No Project condition as a baseline for reservoir operational criteria. The scenarios considered in this analysis were included as elements of the Enhance Flood System Capacity Approach, but were ultimately not moved forward into the State Systemwide Investment Approach because detailed studies and extensive coordination are needed. The only reservoir operational criteria change included in the State Systemwide Investment Approach is the authorized Folsom Dam JFP.

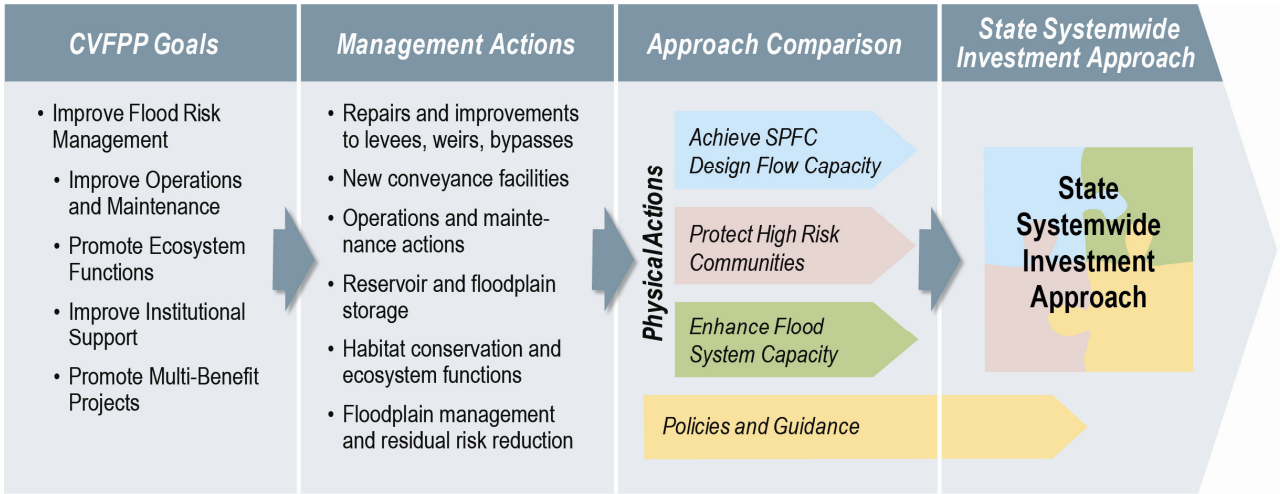

Figure 1-2. Formulation Process for State Systemwide Investment Approach

1.6 Overview of Flood Management in the Central Valley

The Central Valley of California encompasses watersheds of its two major river systems, the Sacramento River in the north and the San Joaquin River in the south. These basins drain more than 43,000 square miles, and the rivers come together in the Delta and discharge to the Pacific Ocean through San Francisco Bay.

Because of the climate and geography of the Central Valley, flooding is a frequent and natural event. Major flooding on the Sacramento and San Joaquin river systems has been documented since the mid-1800s, and has resulted in the loss of lives and massive property damage. This has prompted various planning efforts by State, federal and local entities over the last century and resulted in structural (i.e., construction of physical structures such as dams and reservoirs) and nonstructural (i.e., regulation of floodplain development) efforts. Development of multipurpose reservoirs began in 1932 with authorization of the Central Valley Project (CVP). Multipurpose reservoirs are operated to meet various objectives, such as flood management, water supply, and environmental requirements. The last major flood management facility to be completed was New Melones Reservoir in 1979. Despite improvements to flood management in the Central Valley, damages from flooding have continued, leading to the perceived need for further actions.

Major multipurpose reservoirs in the Sacramento and San Joaquin river basins considered for this analysis are listed in Table 1-1. Note that multipurpose reservoirs located on the eastside tributaries (e.g., Camanche Reservoir) are not included in this table or analysis because hydrologic routing tools are not yet available for those tributaries that enter the San Joaquin River within the boundaries of the Delta. More details on assumptions about reservoirs analyzed are contained in Section 3 of this report. Figure 1-3 is a schematic illustrating the location of the major multipurpose reservoirs considered for this analysis (highlighted in magenta) in relationship to the overall system. The figure shows the size, ownership, and flood management classification for every reservoir in the Sacramento and San Joaquin river basins.

Table 1-1. Major Multipurpose Reservoirs in Sacramento and San Joaquin River Basins Considered in this Analysis

Reservoir	River	Gross Pool Storage (TAF)[1]	Maximum Flood Space (TAF)[1]	Owner	Year Completed
Sacramento River Basin					
Shasta Dam and Shasta Lake	Sacramento River	4,552	1,300	Reclamation	1949
Oroville Dam and Lake Oroville	Feather River	3,538	750	DWR	1968
New Bullards Bar Dam and Reservoir	Yuba River	966	170	YCWA	1970
Folsom Dam and Lake	American River	977	670	Reclamation	1956
San Joaquin River Basin					
Friant Dam and Millerton Lake	San Joaquin River	521	170[2]	Reclamation	1949
Buchanan Dam and H.V. Eastman Lake	Chowchilla River	150	45	USACE	1975
New Exchequer Dam and Lake McClure	Merced River	1,025	350[2]	Merced ID	1967
New Don Pedro Dam and Lake	Tuolumne River	2,030	340	TID/MID	1970
New Melones Dam and Lake	Stanislaus River	2,420	450	Reclamation	1979

Source: adapted from USACE, 1999

Notes:
[1] Storage and flood management space values are rounded to the nearest 1,000 acre-feet.
[2] Maximum flood management space may vary depending on upstream storage and/or snowpack.

Key:
DWR = California Department of Water Resources
Merced ID = Merced Irrigation District
MID = Modesto Irrigation District
Reclamation = U.S. Department of the Interior, Bureau of Reclamation
TAF = thousand acre-feet
TID = Turlock irrigation District
USACE = U.S. Army Corps of Engineers
YCWA = Yuba County Water Agency

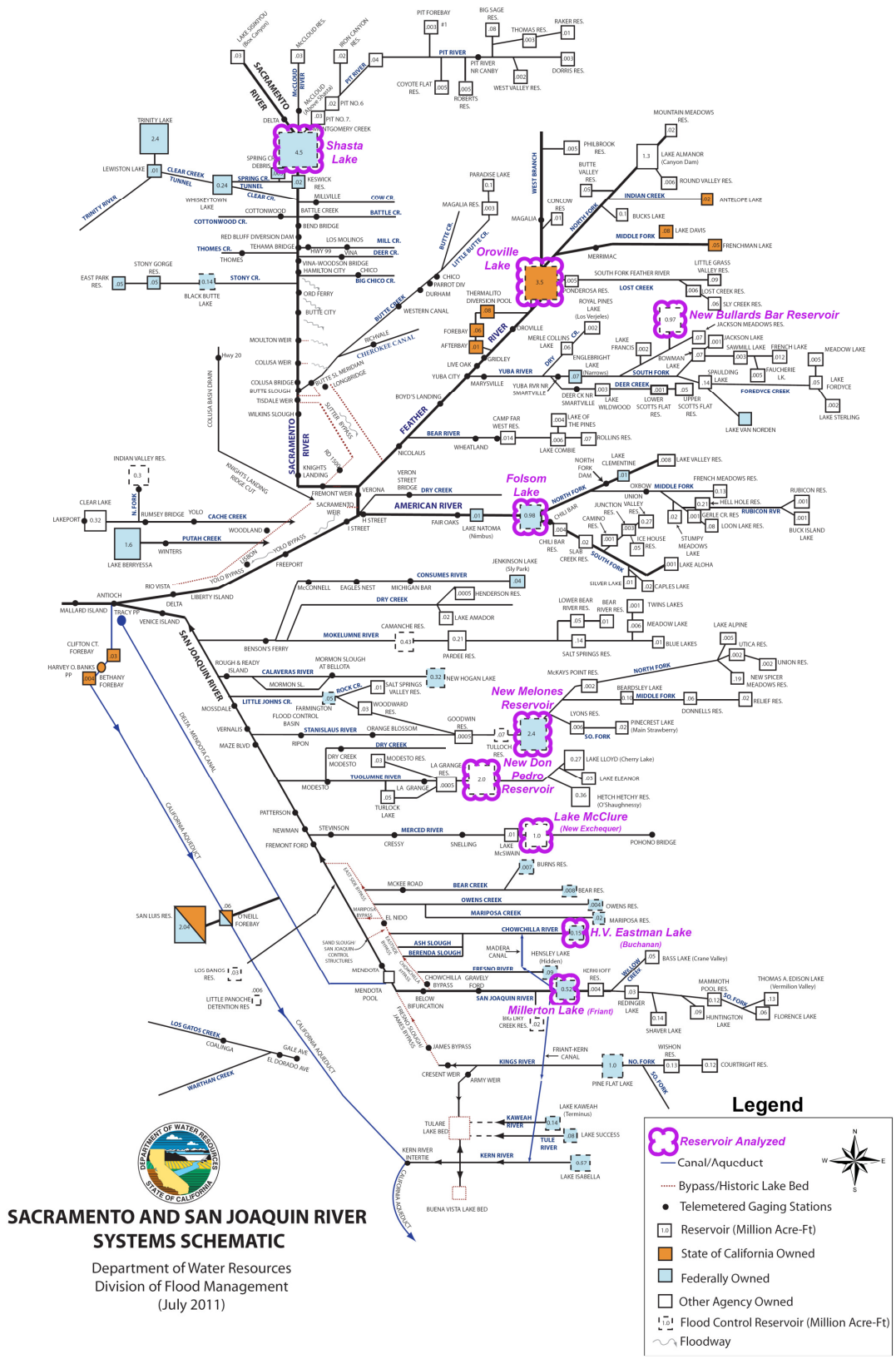

Figure 1-3. Sacramento and San Joaquin River Systems Schematic

1.7 Past and Ongoing Central Valley Flood Reservoir Studies

Numerous investigations regarding flood management reservoirs in the Central Valley have been completed or are ongoing. Most of these flood management reservoirs operate for multiple purposes and changes to any aspect of the reservoir often directly or indirectly affect its flood management operations even though the change may focus on one of the reservoir's other purposes (e.g., water supply, hydropower). In addition, changes to the operational criteria of an individual reservoir can affect how other reservoirs operate. The complicated and interconnected nature of these flood management reservoirs makes the coordination between studies imperative. This section highlights a few of the major studies that State and federal governments are participating in that may affect flood management operations and were considered in the 2012 CVFPP Reservoir Analysis.

1.7.1 2002 Sacramento and San Joaquin River Basins Comprehensive Study

The Comprehensive Study was a joint effort by the Reclamation Board of California (the predecessor of the Board) and USACE, in coordination with State, federal, and local organizations to develop a comprehensive plan for flood damage reduction and ecosystem restoration along the Sacramento and San Joaquin rivers following disastrous floods in January 1997. The Reclamation Board and USACE began working together in 1998 to prepare a comprehensive plan for the combined watersheds of the Sacramento and San Joaquin river basins (USACE, 2002a).

One of the major undertakings of the Comprehensive Study was to develop analytical tools to evaluate how changes to the system would affect the performance of the system as a whole with respect to reducing flood damages, protecting public safety, and restoring degraded ecosystems. The following are examples of computer modeling tools developed under the Comprehensive Study (USACE, 2002b):

- Synthetic hydrology
- HEC-5 reservoir operations models
- UNET hydraulic models
- FLO-2D hydraulic models
- HEC-FDA economic models

These computer modeling tools have the capability to evaluate how broad changes to the system affect its overall performance and to potentially redirect impacts to other parts of the system. Further refinement of these

models could support future planning for regional changes to the flood management system. Reservoir modeling is documented in Technical Studies Documentation Appendix C of the Comprehensive Study (USACE, 2002d).

The tools and methodology developed for the Comprehensive Study were used as a basis for this analysis with updates, as necessary (see Section 3). While new tools and hydrology are being developed by DWR, they were not available for use in the 2012 CVFPP Reservoir Analysis.

The Comprehensive Study synthetic hydrology and hydraulic models were also used for the 2012 CVFPP. Refer to Attachment 8A: Hydrology and Attachment 8C: Riverine Channel Evaluations, respectively, for more details.

1.7.2 Forecast-Coordinated Operations Program

The goal of the F-CO program is to improve flood protection and better protect life and property for communities downstream from flood management reservoirs by reducing peak flood flows through better river flow forecasting and improved coordination. The key to improving flood protection is the coordination of local, State and federal operations during major flood events. This coordination is further enhanced through use of a decision support system and state-of-the-art technology for flood forecasting. The F-CO program allows water managers to operate the reservoirs in advance of and during major flood events with an improved level of forecast certainty, thus reducing peak river flows and the risk of exceeding river channel capacity. The F-CO program also improves notification processes and increases flood warning times to emergency operation managers, State and local offices of Emergency Services, levee districts and the downstream areas in danger of major flooding. Partners in the F-CO program include the California-Nevada River Forecast Center, USACE and reservoir operators.

This non-structural program has been implemented on the Yuba-Feather system as a pilot project and has proven to be one of the most cost-effective flood management improvement measures (described below). Following the success of the Yuba-Feather pilot project, DWR is currently expanding the F-CO program into the San Joaquin River watershed. DWR is currently in the early stage of partnering with some of the reservoir operators in the San Joaquin system.

The F-CO program can be coordinated with operational criteria changes to improve the efficiency by which reservoir storage is managed thereby minimizing potential impacts on the reservoirs' multiple purposes, and to

improve flood protection by maximizing their flood management operations.

Feather-Yuba Forecast-Coordinated Operations

The Feather-Yuba F-CO program began in 2005 to improve flood protection and better protect life and property for communities along and downstream from the Yuba and Feather rivers without impacting the water supply of Lake Oroville and New Bullards Bar Reservoir. The primary objective of the program is to reduce peak floodflows through improved river flow forecasting and improved coordination between Lake Oroville and New Bullards Bar Reservoir (YCWA, 2008).

This program is a cooperative effort by the Yuba County Water Agency (YCWA), DWR, the National Oceanic and Atmospheric Administration (NOAA), and USACE. Under this program, State, federal, and local operations during major flood events will be further enhanced through use of a decision support system and flood forecasting technology; thus, river peak flows and the risk of exceeding channel capacity could be reduced.

The Feather-Yuba F-CO program has completed the following two phases:

- **Phase 1 for design** – To identify and develop tools to improve the quality of flood forecasting and information technology needs.
- **Phase 2 for implementation** – To install 19 remote gaging stations with telemetry systems that transmit data to the California Data Exchange Center. After installation of the gages, efforts will focus on developing a reservoir operations model and integrating the model with the National Weather Service River Forecasting Center system.

The coordinated operation resulting from the Feather-Yuba F-CO program was included as part of the No Project condition (see Section 3).

1.7.3 Forecast-Based Operations Program

After significant progress is made in F-CO program implementation, the next potential opportunity is an F-BO program. Pursuit of F-BO will be based on the interest of the reservoir operators.

The concept of F-BO allows for pre-releasing or storing water based on forecasted reservoir inflows, while taking into consideration the uncertainty of the forecasted inflows and the associated risks of spills and water supply deficits. Such operations more likely require changes in the reservoir flood control manual. The F-BO phase of the project involves (a) the use of forecasting, and (b) proactive reservoir management policies, guidelines,

and rules whose use may reduce flood damages associated with extreme events and improve water management operations. The California Nevada River Forecast Center is currently developing the collaborative forecasting capabilities. Concurrently, the F-CO program has funded the USACE's Hydrologic Engineering Center (HEC) to enhance the HEC-ResSim model to handle collaborative reservoir inflow forecasts.

The need for congressional authorization of the F-BO program will not be definitely determined prior to development of specific modifications/ changes to the flood control manual, so the program is planned to be implemented in two steps. Step one will be to develop the program and document specific reservoir operation modifications, and consult with the USACE. During this step, the scope of the flood control manual s' required modifications and the need for congressional authorization will be identified. Step two, if required, is to seek congressional authorization for the implementation of the F-BO.

While the F-BO program has not been implemented, future F-BO can be coordinated with reservoir operational criteria changes. This coordination has the potential to improve the efficiency with which reservoir storage is managed, thereby improving flood management.

1.7.4 Central Valley Hydrology Study

DWR, under the FloodSAFE Initiative, and in cooperation with USACE, has initiated the Central Valley Hydrology Study, a comprehensive assessment of unimpaired and impaired Central Valley stream flow frequencies and magnitudes. This endeavor includes the development of a comprehensive database of historic rainfall and runoff information, the development of operation models for major Central Valley reservoirs, and an assessment of the effects on the hydrology from climate change. Previous systemwide hydrologic studies, such as the Comprehensive Study, completed a reconnaissance-level analysis of the system. These new Central Valley studies will extend the Comprehensive Study by providing the level of detail required for Federal Emergency Management Agency (FEMA) actions, feasibility planning studies, design of flood management actions, and studies and actions that will enhance operation of the existing flood management system.

The Central Valley Hydrology Study is under development and cannot be used for the 2012 CVFPP Reservoir Analysis. Once the hydrology is available, future studies can use the hydrology to perform their analyses.

1.7.5 Folsom Dam Joint Federal Project

Folsom Dam and Lake, components of the CVP, are owned and operated by the U.S. Department of the Interior, Bureau of Reclamation (Reclamation). The facility is primarily operated to maximize flood management and water supply storage benefits. It is also operated for power, fish and wildlife management, recreation, navigation, and water quality purposes (Reclamation, 2009).

To improve public safety, Folsom Dam and its appurtenant structures (collectively referred to as the Folsom Facility) must be strong enough to withstand the various types of forces and stresses created by a significant earthquake, storm, or seepage event. The authorized Folsom Dam Joint Federal Project (JFP) is a joint effort between Reclamation and USACE to address these issues at the Folsom Facility. The following three objectives are pursued as part of the Folsom Facility improvements:

- **Dam Safety** – the need for expedited action to reduce hydrologic (flood), seismic (earthquake), and static (seepage) events.
- **Flood Damage Reduction** – the need to reduce the risk of flooding in the Sacramento area, which is one of the most at-risk communities in the nation.
- **Increase Spillway Capacity** – provide improved flood protection to the lower American River watershed in conjunction with downstream levee improvements.

Construction activities began in January 2008 and will continue through 2015. These improvements will allow more water to be safely released earlier in a storm event, leaving more storage capacity in the reservoir to hold back peak inflows.

Because the Folsom Dam JFP is already authorized and under construction, this project was included as part of the No Project condition (see Section 3). San Joaquin River Restoration Program

The SJRRP is a comprehensive long-term effort to restore flows to the San Joaquin River from Millerton Lake at Friant Dam to the confluence of the Merced River and restore a self-sustaining Chinook salmon fishery in the river while reducing or avoiding adverse water supply impacts from restoration flows.

Implementation of the SJRRP would affect the timing and volume of Millerton Lake releases and potentially carryover storages. This program, while not intentionally changing flood operations, may incidentally affect

flood management benefits, especially when paired with reservoir operational criteria changes.

1.7.6 Surface Storage Investigations

To address new water resources needs in California, the State and federal governments have funded five Surface Storage Investigations, which were conceived to support at least three of CALFED's programmatic goals: water supply reliability, water quality, and ecosystem restoration.

These new projects are being designed to be adaptive and robust, and would support aquatic and riparian ecosystem restoration focused on the Delta and its tributaries, improved drinking and habitat water quality, and the water supply needs associated with California's growing population and diverse economy. Furthermore, these projects must perform well under a number of potential future conditions including changing environmental conditions and needs, climate change, alternative Delta conveyance and management, and disaster/emergency response scenarios (DWR, 2012).

The five surface storage investigations are as follows:

- Shasta Lake Water Resources Investigation (Shasta Enlargement)
- North-of-the-Delta Offstream Storage (Sites Reservoir)
- In-Delta Storage Program
- Los Vaqueros Reservoir Expansion
- Upper San Joaquin River Basin Storage Investigation (Temperance Flat Reservoir)

These surface storage investigations (with the exception of the In-Delta Storage Program) will change the configuration of the Central Valley river systems and affect how flood management operations occur. These projects are not included in the 2012 CVFPP Reservoir Analysis because they are still in their early planning stages, but are important as they may affect future operational criteria change studies.

1.7.7 Federal Energy Regulatory Commission Relicensing

FERC relicensing does not typically affect flood operating rules, which are prescribed by USACE for federal projects or as a condition of federal cost sharing on nonfederal projects. But, FERC relicensing may change how water is released and the timing and magnitude of inflow into downstream

major multipurpose reservoirs, thus having an incidental effect on flood operations.

Reservoirs that have hydropower facilities are regulated through licenses that FERC issues for given periods of time. As the expiration date of an existing license approaches, dam owners must undergo FERC relicensing, which involves reviewing operational practices of the overall facility to continue operation of the hydropower facilities.

Per the 1986 Federal Power Act, FERC is required to develop license conditions with equal consideration of development and environmental values. The FERC relicensing process provides an opportunity for public and resource agencies to evaluate project effects and balance needs from different perspectives, as well as to modify hydropower dams to meet modern environmental standards. New licenses establish new requirements for water supply, flood management, water quality, fisheries, wildlife, recreational uses, cultural resources, etc. Implementation of these requirements is unlikely to change reservoir flood management operational criteria.

The FERC relicensing process takes multiple years to complete. At least 5 years before a license expires, the licensee must file a notice of intent to file a new license and submit a preapplication document with a proposed study plan to begin the scoping process for an environmental analysis. At least 2 years before a license expires, the licensee must file an application for a new license, and FERC begins the environmental analysis.

In the Central Valley, several reservoirs are undergoing the relicensing process, including Lake Oroville, Middle Fork American River Project, New Bullards Bar Reservoir, New Don Pedro Reservoir, New Exchequer Reservoir, and Mammoth Pool. Lake Oroville, an SPFC facility, is owned by DWR and is operating under an annual license issued by FERC effective on February 1, 2007. Through the FERC relicensing process, the Oroville Facilities were to reevaluate all project purposes and to accommodate current issues that were not extant when the first 50-year license was issued in 1957. One such issue is the potential effects of the facility on spawning Chinook salmon; this will be mitigated through the use of the Oroville Facilities Chinook Salmon Fish Hatchery (DWR, 2010a).

FERC relicensing may change how water is released and the timing and magnitude of inflow into downstream major multipurpose reservoirs, thus having an incidental effect on flood operations and potentially the benefits associated with operational criteria changes.

1.8 Report Organization

Organization of this document is as follows:

- Section 1 introduces and describes the purpose of this attachment. It also provides an overview of flood management in the Sacramento and San Joaquin river systems, and past and ongoing Central Valley flood reservoir studies that affect reservoir operational criteria and form a basis for this analysis.

- Section 2 summarizes results and findings of 2012 CVFPP reservoir modeling in the Sacramento and San Joaquin river basins and future opportunities for reservoir analyses after 2012.

- Section 3 describes the methodology used in this analysis.

- Section 4 describes the current (No Project) performance of multipurpose reservoirs in the Sacramento and San Joaquin river basins.

- Section 5 describes the sensitivity of the system to reservoir operational criteria changes that were used to identify scenarios for further consideration.

- Section 6 explores two operational scenarios considered for the 2012 CVFPP.

- Section 7 summarizes the simulated flood management benefits of the scenarios considered.

- Section 8 contains references for the sources cited in this document.

- Section 9 lists acronyms and abbreviations used in this document.

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2.0 Results Summary and Findings

Model results from this preliminary analysis conducted for the 2012 CVFPP, suggest that there are potential systemwide flood management benefits that could result from allocating more space to flood storage and from modifying release schedules, especially when operational criteria changes reduce downstream peak flood stage. It is recommended that future detailed and coordinated studies occur to consider other potential effects (e.g., water supply, environmental) and to explore the feasibility of modifying operational criteria at individual reservoirs.

While this analysis does not propose any specific changes to reservoir operational criteria or suggest that these changes are the only options for modifying operational criteria, the 2012 CVFPP Reservoir Analysis does provide insight for future studies to explore operational criteria changes in more detail. This analysis highlighted the following observations:

- Operational criteria changes are generally effective in lowering downstream peak flow and, as a result, the volume of water leaving a channel through levee breaches.
- While operational criteria changes can reduce peak downstream flood flow, the changes in peak flow are not necessarily consistent for all frequency storm events or for all storm locations (centerings).
- Delaying larger reservoir releases could allow floodwater from other tributaries to pass through the Central Valley flood management systems before the modified reservoirs release their higher flow, generally resulting in lower downstream peak flows.
- The volume of additional flood storage allocation is not equal to the actual reduction in out-of-system flow volume (e.g., an additional 100 TAF of flood storage allocation does not reduce the volume of out-of-system flow by 100 TAF). Therefore, from the viewpoint of containing out-of-channel flood volume, an increase in flood storage allocation may not be as efficient as other methods.

In general, physical or operational criteria changes could reduce the need for some types of downstream actions, such as levee improvements, and could mitigate the hydraulic effects that system improvements can have on downstream reaches. Reservoir operational criteria changes can also provide greater flexibility to accommodate future hydrologic changes, (e.g., climate change), provide greater system resiliency, and benefit the ecosystem.

While changes to flood storage allocations and objective releases typically require relatively small capital costs, they could have significant water resources impacts and present regulatory challenges. Because of the interconnected nature of the multipurpose reservoirs in the Central Valley, changes to flood management operations will affect operations for other purposes (including water supply, hydropower generation, and recreation). To implement such changes would require a detailed project-level analysis and coordination to develop a comprehensive suite of analyses, followed by significant administrative actions. The 2012 CVFPP recommends an overall system reservoir analysis to holistically evaluate potential integrated solutions, such as the one DWR is currently formulating under its System Reoperation Program.

2.1 Inclusion in 2012 CVFPP Approaches

The preliminary findings from this analysis were included in the Enhance Flood System Capacity Approach. This approach includes modifications to the reservoir release schedule and flood storage allocation at Lake Oroville (equivalent to an additional 200,000 acre-feet of flood storage), and coordinated operation with New Bullards Bar Reservoir, to reduce flood stages on the Feather River during a 200-year (0.5 percent annual exceedence probability (AEP)) flood event. Also, in the San Joaquin River Basin, the State would partner with interested reservoir operators to increase the flood storage allocation at New Don Pedro, Friant, and/or New Exchequer dams by about 400,000 acre-feet to effectively manage the 100-year (1 percent AEP) flood event at these reservoirs. In combination with bypass expansion and other features of the Enhance Flood System Capacity Approach, these operational features help manage the timing and magnitude of peak floodflows before they enter the Sacramento and San Joaquin rivers.

Operational criteria changes were ultimately not moved forward into the State Systemwide Investment Approach because of: (1) the preliminary nature of this analysis; (2) uncertainty associated with the potential effects of operational criteria changes; and (3) the need to coordinate with operators and owners on more detailed, reservoir-specific analyses. An exception is the already authorized operational changes associated with the Folsom Dam Raise, which are included in both the No Project condition and State Systemwide Investment Approach.

2.2 Potential Future Studies

The 2012 CVFPP Reservoir Analysis described herein provides insight for future evaluations, and these future reservoir operational criteria studies should focus on the development of integrated solutions that consider project-specific costs as well as addressing potential effects on other reservoir purposes. The integrated solutions could include actions such as increasing downstream transitory storage, constructing setback levees, and increasing upper watershed storage to maximize flood management and other benefits.

Conjunctive use (CU), which is the cooperative management of both surface water and groundwater, is another possibility to be explored in future reservoir analyses. By diverting water from a flood management reservoir into a groundwater aquifer prior to flood season, CU could increase flood protection by providing additional flood storage allocation in the reservoir, but could still recover the prestored water if needed during the year. Combining this CU analysis, with other systemwide analyses would aid in formulating and selecting reservoir operational criteria change alternatives. These future studies should also be coordinated with ongoing studies such as DWR's existing F-CO and planned F-BO programs.

As stated above, the 2012 CVFPP Reservoir Analysis used existing data and tools to explore modifications to the reservoir operational criteria of flood storage allocation and objective release. In addition to reservoir operational criteria changes, other actions (such as increasing transitory storage, constructing setback levees, and increasing upper watershed storage) that maximize flood management benefits while providing other benefits should be explored to identify integrated flood management solutions. Various efforts have been made and others are underway to analyze the potential for reservoir operational criteria changes in further detail.

In summary, with the defined vision from the 2012 CVFPP, future reservoir analyses could include, but not be limited to, the following:

- **Hydrology Updates** – New hydrology is being developed for the Sacramento and San Joaquin river basins under the Central Valley Hydrology Study. This new hydrology will be used to prepare new inflow hydrographs for the HEC-5 (or HEC-ResSim) models.

- **Climate Change** – Current inflow hydrographs for the HEC-5 models were developed based on historical data and climate

information. Climate change may modulate the "typical" hydrology[1] and alter the timing and evacuation requirements for flood management; thus, it is necessary to incorporate climate projections into reservoir operational criteria. Once DWR identifies a standardized approach for climate change, hydrology could be updated to address climate change. In addition, a better understanding of changes in the timing and distribution of precipitation and runoff within the State would improve decisions regarding operational criteria changes, as well as the ability to assess systemwide effects of operational criteria changes.

- **Reservoir Modeling Tools** – The HEC-5 models from the Comprehensive Study, provide a basin-wide representation of Central Valley multipurpose reservoirs, and a prefeasibility tool to identify ranges of operational criteria change scenarios for future analysis. Project-specific reservoir analyses will require reservoir models with additional details for in-depth evaluations. New models could be developed or adapted for analysis in the future.

- **System Optimization** – Future analyses could aim to apply an optimization approach to identify optimal alternatives under interconnected operational criteria constraints (e.g., water supply, flood management operations, and hydropower generation constraints).

- **Headwater Reservoir Operations** – Headwater reservoirs are mainly for hydropower generation, and mostly have no formal flood management functions. However, previous studies have indicated that available storage in headwater reservoirs could significantly reduce peak inflows into lower basin reservoirs (USACE, 2002d). Changes in headwater reservoir operations could potentially reduce flood damage through spillway regulation or alteration of outlet elevations to better account for flood operations.

- **Offstream Storage Opportunity** – Diverting excess floodflows from river channels into adjacent storage areas could reduce flow rate and stage within the main channels. Refuge or agricultural areas along the mainstem Sacramento and San Joaquin rivers could provide such offstream storage for flood damage reduction. These

[1] Hydrologic impacts of climate change are uncertain, but are likely to increase hydrological variability in the future and include less frequent precipitation, more intense precipitation, increased frequency of dry and extremely wet days, and less snowpack and snow cover. Precipitation shifts would affect the origin and timing of runoff. Increases in precipitation intensity could increase flood events, and thus change the overall flood regime (such as the frequency of different sized floods) and affected areas (Brekke et al., 2009).

storage projects would provide opportunities to allocate or reallocate dedicated flood storage space or change operational criteria to meet flood damage reduction objectives.

- **Physical Reservoir Modifications** – The 2012 CVFPP Reservoir Analysis only explored the potential of altering reservoir operational criteria, not physical modifications. To minimize the effects on the other purposes of the reservoirs (e.g., water supply reliability, hydropower generation, recreational opportunities, groundwater storage, instream requirements), physical modifications to the dams and reservoirs should be considered in future analyses. For example, increasing the size/capacity of a reservoir would provide additional flood storage without reducing the current water supply storage.

- **Starting Storage Assumptions** – This analysis assumed that the starting storage for each reservoir was the top of conservation pool. Especially for lower frequency storms, starting storage may be lower than assumed in this analysis. Future analyses should explore the potential benefits and impacts of operational criteria changes under various reservoir starting storages.

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3.0 Methodology

This section first provides an overview of the approach used for the 2012 CVFPP Reservoir Analysis. Then it summarizes past reservoir studies on changes to operational criteria, whose methodology and tools were used as a starting point for the 2012 CVFPP Reservoir Analysis. The remainder of the section discusses the assumptions, model selection, and model specifications used in the 2012 CVFPP Reservoir Analysis.

3.1 2012 CVFPP Reservoir Analysis Approach

The 2012 CVFPP Reservoir Analysis was separated into two phases and conducted as five different activities. Phase 1 reviewed past studies on changes to operational criteria (Activity 1), explored the current ability of reservoirs to manage a range of flood events (Activity 2a), and identified a range of reservoir operational criteria changes that could potentially reduce peak flow along the mainstem rivers for further analysis in Phase 2 (Activity 3a). Phase 1 of the 2012 CVFPP Reservoir Analysis did not identify any reservoir-specific changes in reservoir operational criteria, but did identify potential types of operational criteria changes, such as enlargement of flood storage allocation or modifications to reservoir release criteria, for future analysis in Phase 2.

The objectives of Phase 2 were to further explore and identify the current (No Project) ability of reservoirs to manage flood events (Activity 2b), perform incremental operational criteria changes based on Phase 1 observations (Activity 3b), explore operational scenarios (Activity 4), and estimate benefits and impacts from the scenarios (Activity 5). Phase 2 explored two scenarios, one in the Sacramento River Basin and one in the San Joaquin River Basin, that have potential to help reduce downstream floodflows, thereby increasing flood management flexibility.

Figure 3-1 outlines the two phases and briefly describes the activities conducted in each phase. Each type of activity is grouped together and described in separate sections in this report.

Phase 1 Reservoir Analysis

- Activity 1 – Past Reservoir Analyses Modeling Summary: Review past studies on changes to operational criteria of multipurpose reservoirs to gain a basic understanding of effects and develop methodology.
- Activity 2a – No Project System Performance: Improve understanding of the ability of reservoirs to manage a range of flood events under their No Project operational criteria.
- Activity 3a – Basin-Wide Sensitivity to Changes in Reservoir Operational Criteria: Explore how flood management in the Central Valley would react to simultaneous operational criteria changes at multiple reservoirs and identify which reservoirs have potential to benefit the system.

Phase 2 Reservoir Analysis

- Activity 2b – No Project System Performance: Identify the ability of reservoirs to manage a range of flood events under their existing operational criteria.
- Activity 3b – Basin-Wide Sensitivity to Changes in Reservoir Operational Criteria: Make incremental operational criteria changes to identified reservoirs.
- Activity 4 –Reservoir Operational Scenarios Considered: Explore two scenarios, one in the Sacramento River basin and one in the San Joaquin River basin.
- Activity 5 – Effects of Operational Criteria Changes: Quantify simulated effects of the two scenarios considered on flood risk management.

Figure 3-1. 2012 CVFPP Reservoir Analysis Flowchart

3.2 Past Reservoir Analyses Modeling Summary

Prior to the 2012 Reservoir Analysis, one other study that analyzed flood management in both the Sacramento and San Joaquin river basins from a systemwide perspective was the Comprehensive Study. Before the Comprehensive Study, studies focused on making incremental changes to the system without fully understanding how they might affect other parts of the system and the performance of the system as a whole. Because of similar objectives and systemwide perspective, the 2012 CVFPP Reservoir Analysis used the Comprehensive Study models and data as a basis for the analysis, with updates as necessary to include modifications to flood management in the Central Valley after the Comprehensive Study was completed. The models were then used to evaluate potential systemwide flood management effects from changing reservoir operational criteria for the 2012 CVFPP.

3.2.1 Comprehensive Study Background

The goal of the Comprehensive Study was to develop a comprehensive plan for flood damage reduction and ecosystem restoration along the Sacramento and San Joaquin rivers. A major part of the study was to develop analytical tools capable of evaluating the effects of changes on performance of the system as a whole with respect to reducing flood damages, protecting public safety, and restoring degraded ecosystems.

The Comprehensive Study reservoir modeling used HEC-5 as the reservoir simulation software. Extensive efforts were made to collect data and input flood management operational criteria into HEC-5 models to accurately represent without-project conditions. Detailed HEC-5 reservoir modeling was then performed to evaluate various flood management alternatives, including the following categories (USACE, 2002d):

- Operational criteria changes to lower basin reservoirs
  - Grid analysis that varied flood storage and objective releases of individual reservoirs
  - Reservoir operational criteria changes of existing reservoirs
  - Incorporation of floodplain storage areas in the San Joaquin River Basin with reservoir operational criteria changes
- Operational criteria changes to headwater reservoirs
- Use of onstream and offstream storage

These evaluations were completed by modifying the assumptions in the HEC-5 base models (e.g., increasing available flood storage allocation, decreasing objective release criteria) and running the models for storms of various AEPs and centers. Potential effects resulting from the Comprehensive Study alternatives were evaluated by comparing peak flows at control points for the alternative conditions against without-project conditions assuming that a reduction in peak flow could decrease flood damage. Details of the reservoir operation modeling are documented in Comprehensive Study Technical Studies Documentation Appendix C (USACE, 2002d).

The rest of this subsection provides a results summary of various flood management operation alternatives. These preliminary findings from the Comprehensive Study helped guide the 2012 CVFPP Reservoir Analysis.

3.2.2 Grid Analysis

The Comprehensive Study lower basin reservoir analysis included performing a grid analysis to evaluate how incremental changes to an individual reservoir's flood management storage and/or objective release affect the ability to manage flood events of various frequencies. Both the flood storage allocation and the objective release were changed incrementally (individually and in combination) for a range of values. The flood storage allocation was changed by lowering the required top of conservation pool on the flood rule curve (see Figure 3-2 for an example). The solid and dotted lines represent the minimum amount of required space with and without flood storage allocation changes, respectively, to be kept in the reservoir at all times. For each modification, changes in peak reservoir outflow rates under different storm events were evaluated.

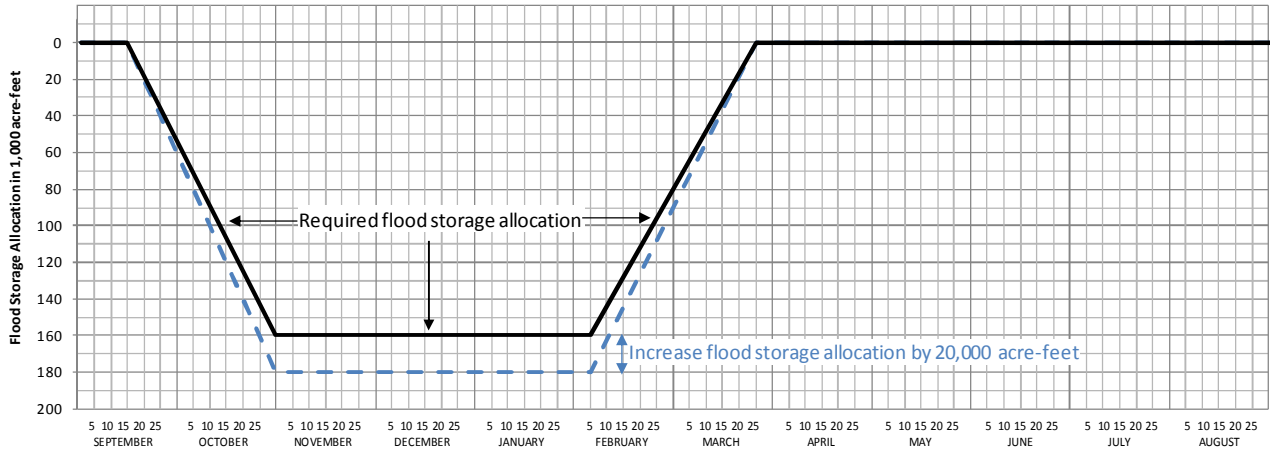


Figure 3-2. Example Flood Management Diagram

In HEC-5, the required flood storage allocation for a targeted reservoir was increased (or decreased) by lowering (or raising) the top of conservation pool; no changes to the reservoir size were made. With a larger flood storage allocation, the reservoir could store a larger volume of inflow before it reached the gross pool, thus delaying or even eliminating emergency spillway releases that were higher than the objective release. Additional storage allocation could increase flood protection and help meet objective flows (therefore maintaining flows at or below channel capacity) during larger events.

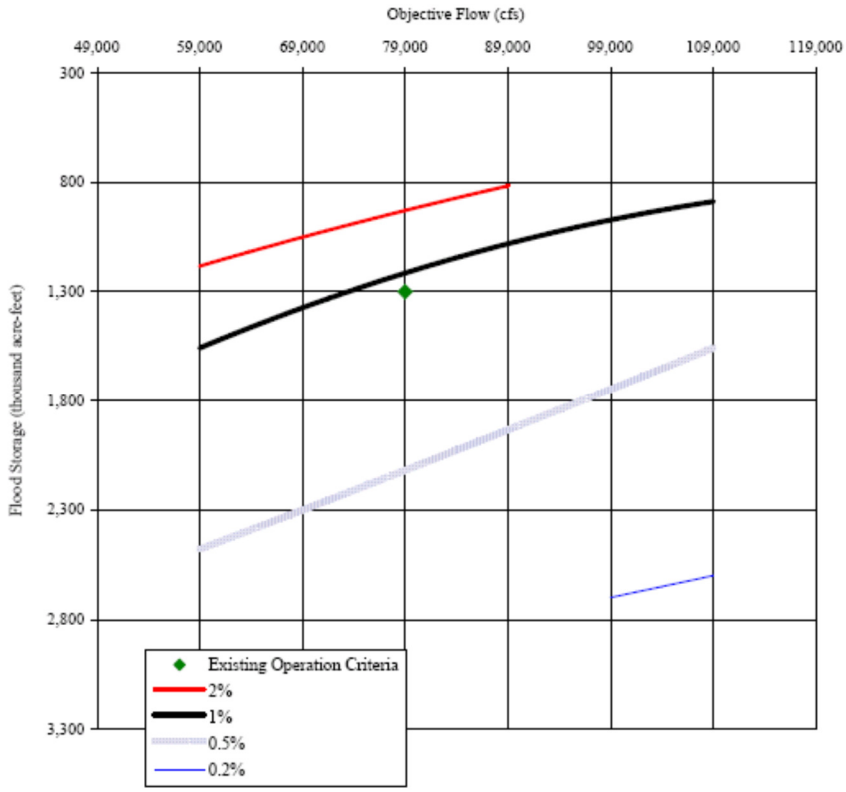
Lowering the objective release criteria could reduce reservoir outflow rates and shift the timing of the peak tributary flow to prevent coinciding with the peak flow in the mainstem. However, reducing the objective release could speed up filling of the flood pool storage and lead to earlier emergency spillway releases.

Other changes were made in the HEC-5 model for consistency between the simulations. These changes included, but were not limited to, the following:

- Starting storage of the targeted reservoir
- Gate operations
- Release ramping schedule

Figure 3-3 shows an example of grid analysis results for Shasta Lake. The curves delineate combinations of flood storage and objective flows that would pass a specified frequency event while exhausting the capabilities of the reservoir. Points above a curve indicate objective flows have been exceeded, and values below a curve indicate objective flows have not been exceeded for a particular storm event. For example, Shasta Lake is currently capable of controlling a flood event with less than a 1 percent AEP (1 percent chance of occurring in any year). Increasing the flood storage at Shasta Lake to approximately 2,100 thousand acre-feet (TAF) could enable Shasta Lake to manage up to a 0.5 percent AEP flood event without exceeding the current objective release of 79,000 cubic feet per second (cfs).

This Comprehensive Study analysis shows how changes to a reservoir's objective flow and flood storage allocation influence the level of flood protection along the mainstems and tributaries of both the Sacramento and San Joaquin rivers. Results from the grid analysis were used as a guide for the reservoir alternatives discussed below.



Source: USACE, 2002b
Notes:
1. Data representing the 50 percent and 10 percent AEP storm events are not plotted because Shasta Lake is capable of completely detaining inflows generated by events of these magnitudes.
2. Current objective flow = 79,000 cubic feet per second
3. Current maximum flood storage allocation = 1,300 thousand acre-feet

Figure 3-3. Grid Analysis Results for Shasta Lake

3.2.3 Operational Criteria Changes to Lower Basin Reservoirs

In the Comprehensive Study, the primary purpose of modifying operational criteria at lower basin reservoirs was to alter peak flows of both the mainstems and tributaries of the Sacramento and San Joaquin rivers. Alternatives included arbitrary changes in objective flow and available flood storage allocation for one or more reservoirs under different storm events. In the Sacramento River Basin, operational criteria changes were made in flood reservation and objective release to Shasta Lake, Lake Oroville, and New Bullards Bar Reservoir, and flows were limited at Cottonwood Creek (Table 3-1). In the San Joaquin River Basin, operational criteria changes were made in flood reservation and objective release at Millerton Lake (Friant Dam), Lake McClure (New Exchequer Dam), and New Don Pedro Reservoir (Table 3-2). For these alternatives, increases in flood reservation were drastic, often doubling the existing

Table 3-1. Lower Basin Reservoir Operational Criteria Changes – Sacramento River Basin Alternatives

Reservoir	Operational Criteria	Existing Condition	Alternative 1	Alternative 2	Alternative 3	Alternative 4
Shasta Lake	Flood Reservation	1,300 TAF	+1,300 TAF	-	-	-
	Objective Release	79,000 cfs	-	-	-	-
Cottonwood Creek	Flood Reservation	N/A	-	-	-	-
	Objective Release	N/A	-	Up to 15,000 cfs	-	-
Lake Oroville	Flood Reservation	750 TAF	+750 TAF	-	Incremental changes made to available storage and objective flow	Incremental changes made to available storage and objective flow
	Objective Release	150,000 cfs	-	-		
New Bullards Bar Reservoir	Flood Reservation	170 TAF	-	-	Incremental changes made to available storage and objective flow	-
	Objective Release	50,000 cfs	-	-		-

Source: Adapted from USACE, 2002d

Key:
- = no change
cfs = cubic feet per second
N/A = not applicable
TAF = thousand acre-feet

flood reservation. Note that doubling the flood storage for some reservoirs is a small portion of the total reservoir (e.g., adding 1,300 TAF of flood storage as compared to the total Shasta Lake storage of 4,552 TAF).

Table 3-3 contains example HEC-5 results from reservoir operational criteria changes. It presents peak flow reduction at six locations for an Ord Ferry-centered storm and seven return frequencies for Sacramento River Basin Alternative 1 (doubling flood reservation in both Shasta Lake and Lake Oroville).

Results from the Comprehensive Study alternatives demonstrated that operational criteria changes to existing reservoirs have the potential to reduce peak flow at various locations in the Sacramento and San Joaquin river basins.

Table 3-2. Lower Basin Reservoir Operational Criteria Changes – San Joaquin River Basin Alternatives

Reservoir	Operational Change	Existing Condition	Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5	Alt. 6	Alt. 7
Millerton Lake	Flood Reservation	170 TAF	+170 TAF	+100 TAF	-	+50 TAF	+100 TAF	-	-
	Objective Release	6,500 cfs	-	-	-	Up to 4,000 cfs	-	Up to 8,000 cfs	-
Lake McClure	Flood Reservation	350 TAF	-	+50 TAF	-	-	+50 TAF	-	-
	Objective Release	6,000 cfs	-	-	-	Up to 1,000 cfs	-	-	Up to 2,000 cfs
New Don Pedro Reservoir	Flood Reservation	340 TAF	+340 TAF	+100 TAF	-	-	+200 TAF	-	-
	Objective Release	9,000 cfs	-	Up to 2,000 cfs	Up to 6,000 cfs	Up to 6,000 cfs	-	-	-

Source: Adapted from USACE, 2002d
Key:
- = no change
Alt. = Alternative
cfs = cubic feet per second
TAF = thousand acre-feet

Table 3-3. Percent Peak Flow Reduction at Mainstem Gage Locations in Sacramento River Basin for Alternative 1

Ord Ferry Storm Runoff Centering						
AEP (percent)	Bend Bridge	Vina Bridge	Ord Ferry	Oroville	Verona	Sacramento
50	0.0	0.0	0.0	42.7	9.6	12.2
10	0.0	0.0	0.0	36.1	8.5	6.4
4	0.0	0.0	0.0	55.0	16.8	13.3
2	0.0	0.0	0.0	53.6	15.7	12.4
1	0.0	0.0	0.0	60.9	21.2	17.0
0.5	10.2	0.0	0.0	30.0	8.4	6.9
0.2	38.6	18.7	20.0	0.0	0.8	0.5

Adapted from USACE, 2002d

Notes:
1. Flow at mainstem points are estimated by HEC-5, which assumes all flows remain in channel (bypasses were treated as channels).
2. Percent Peak Flow Reduction = ((Maximum Regulated No Project Inflow)-(Maximum Regulated Alternative Inflow))/ (Maximum Regulated No Project Inflow) X 100%.
Key:
AEP = annual exceedence probability

3.2.4 Major Comprehensive Study Findings

The Comprehensive Study evaluation of potential reservoir operational criteria changes led to several important findings for flood management in the Central Valley that were used to inform operational criteria changes in the 2012 CVFPP Reservoir Analysis:

- The Central Valley flood management systems' design does not provide a uniform level of flood protection to all areas.
- The Central Valley flood management systems cannot safely convey the flows that it was formerly considered capable of accommodating.
- All of the preliminary systemwide evaluations indicated that some amount of new flood storage is needed in the Sacramento River Basin, regardless of the type of flood management improvements implemented.
- Weirs and bypasses in the Sacramento River Basin tend to dampen the effects of changes to the flood management systems.
- Under existing conditions, flow out of the Tuolumne River system overwhelms flow in the San Joaquin River downstream from the Tuolumne River confluence.
- During floods, water leaves the Central Valley foothills and moves through the different rivers and channels in the Central Valley at different rates. Thus, flood peak from one tributary might reach the mainstem hours or days before the peak from another tributary.
- If levee reliability were improved systemwide, substantial increases in flood storage capacity could offset hydraulic impacts in downstream areas because of improved upstream reliability.
- A comprehensive solution to improve public safety, reduce flood damages, and restore degraded ecosystems in the Central Valley will require a combination of measures that increase conveyance capacity and flood storage, and improve floodplain management.

3.3 2012 CVFPP Reservoir Analysis Assumptions

Using the preliminary findings and methodology from the Comprehensive Study, reservoir operational criteria changes were considered for the 2012 CVFPP Reservoir Analysis if a reservoir met the following conditions:

- Reservoir is multipurpose (i.e., flood management, water supply, recreation)
- Gross pool is greater than 100 TAF
- Reservoir is located within the analysis area
  - Reservoir is located within the Sacramento and San Joaquin river basins
  - Reservoir is located on mainstem or tributaries that connect directly to the mainstem
  - Reservoir is not located on eastside tributaries or within the Delta

Operational criteria at reservoirs that are solely or mostly operated for flood management (i.e., less than 100 TAF of storage is dedicated for nonflood management purposes) were not changed because insufficient flexibility existed in operations since nearly all of the storage is already dedicated to flood management. Similarly, if a reservoir had a gross pool smaller than 100 TAF, it was not considered because there is little flexibility in operations. Reservoirs located outside the Sacramento and San Joaquin river basins were not considered (i.e., Pine Flat Lake, located on the Kings River) because they are outside the area of analysis. Reservoirs located on tributaries that do not enter the Sacramento or San Joaquin rivers directly were also not included because most of the effects of operational criteria changes would not affect the mainstems. For example, Indian Valley Reservoir, on the North Fork Cache Creek, was not analyzed because Cache Creek drains into the Yolo Bypass, not directly to the Sacramento River. Reservoirs on the eastside tributaries (Cosumnes, Mokelumne, and Calaveras rivers and Littlejohns Creek), which drain into the San Joaquin River within the Delta boundary, were also not included because they are at the downstream end of the system, thus having less potential for systemwide benefits.

Of the 24 lower basin reservoirs included in the existing HEC-5 models (refer to Section 3.5.1), 9 fit these conditions; therefore, operational criteria changes at these reservoirs were explored further in this analysis (Table 3-4).

Table 3-4. Reservoirs Considered for Operational Criteria Changes

Sacramento River Basin	San Joaquin River Basin
Shasta Lake	New Melones Reservoir
Lake Oroville	New Don Pedro Reservoir
New Bullards Bar Reservoir	Lake McClure
Folsom Lake	H.V. Eastman Lake
	Millerton Lake

The following decisions were made for tool and methodology selection:

- Because the 2012 CVFPP Reservoir Analysis was based on the Comprehensive Study, which primarily focused on the Sacramento and San Joaquin river basins, effects on the Delta were not directly explored.
- The 2012 CVFPP Reservoir Analysis used the best available existing tools for the analysis. New reservoir simulation models (e.g., DWR and USACE HEC-ResSim models) and new hydrologic information are under development, but they were not available for this analysis.
- Operational criteria changes were made to maximize systemwide flood management benefits.
- Other effects, including water resources benefits, and hydropower and environmental impacts, were not considered when making operational criteria changes.
- No climate change or environmental analyses were conducted.

3.4 2012 CVFPP Reservoir Analysis Model Selection

Three computer models were used to conduct this analysis: HEC-5, HEC-ResSim, and UNET. As described above, the 2012 CVFPP Reservoir Analysis was divided into five different activities. The first activity, review of past reservoir analyses modeling, did not require any additional modeling as part of the 2012 CVFPP. The corresponding models used for each of the remaining four activities of the analysis are as follows:

Activity 2. No Project System Performance – HEC-5 and HEC-ResSim

Activity 3. Basin-Wide Sensitivity to Reservoir Operational Criteria Changes – HEC 5

Activity 4. Reservoir Operational Scenarios Considered – HEC-5

Activity 5. Effects of Operational Criteria Changes on Flood Risk Management – UNET

Figure 3-4 shows an overview of how the models relate to each other and their inputs and outputs.

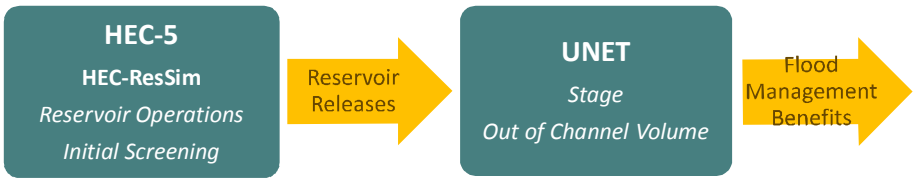


Figure 3-4. Models Process Overview

3.4.1 HEC-5 Hydrologic Reservoir Operations Model

Preliminary flood management benefits were compared using the hydrologic reservoir operations model HEC-5. This is a reservoir operations model that simulates rule curves and other operational criteria based on reservoir inflow. HEC-5 provided preliminary estimates for the reduction in peak flow, duration, and magnitude of channel capacity exceedence, and contribution of reservoir flood releases to downstream flow at index point locations (i.e., key locations of interest to observe effects of operational criteria changes) for a wide range of scenarios.

The HEC-5 model implementation developed for the Comprehensive Study and simulating all of the major reservoirs in the Sacramento and San Joaquin river basins was selected for use in this analysis because it is currently the best available systemwide model. While new tools are being developed, they were not available for use in this analysis.

The HEC-5 Comprehensive Study models represent Year 2000 reservoir operational criteria within the current flood management systems of the Sacramento and San Joaquin river basins. These models were updated for the 2012 CVFPP Reservoir Analysis to include changes to reservoir operations since completion of the Comprehensive Study (see Section 3.5.1).

3.4.1 HEC-ResSim Hydrologic Reservoir Operations Model

HEC-ResSim supplemented HEC-5 to simulate current reservoir operations and screen various reservoir operational criteria changes. HEC-5 is a

legacy program; HEC-ResSim, developed by USACE, is its successor and includes a graphical user interface and the ability to better simulate some types of operational rules.

HEC-ResSim was used to simulate American River and Folsom Lake operational criteria, including the new Folsom Dam JFP modifications, because it would be difficult to simulate these operations in HEC-5. While the preferred method for incorporating Folsom Dam JFP changes would be to modify HEC-5, doing so would not accurately reflect the Folsom Dam JFP. HEC-5 was unable to accurately simulate the variable release diagram and design targets associated with the Folsom Dam JFP. As a result, the USACE HEC ResSim model of the American River was used to simulate releases from Folsom Lake. Results from the HEC-ResSim model were used as input into the HEC-5 model.

Although HEC-ResSim demonstrates more advanced features and improvements than HEC-5, it was only used to simulate reservoir operations in the American River Basin because systemwide HEC-ResSim models were not available at the time of this analysis.

3.4.1 UNET Hydraulic Model

Once the two potential scenarios for consideration were identified, UNET was run to assess in more detail the effects of operational criteria changes on flood management. UNET used the time series of reservoir releases from HEC-5 to compute the stage and out-of-channel volume of water throughout both basins. UNET is an unsteady-state riverine hydraulic flow model that simulates the one-dimensional (1-D) flow in a network of streams. The UNET model used in this analysis was first developed as part of the Comprehensive Study to simulate floods in the Sacramento and San Joaquin river basins, including levee breaks.

New river hydraulic models are currently under development by DWR, but were not available for the 2012 CVFPP. Therefore, the available UNET model and data, with some updated information, were used for analyses required for the CVFPP.

3.5 2012 CVFPP Reservoir Analysis Model Specifications

The following describes model specifications for the three models used in this analysis. Because the majority of the 2012 CVFPP Reservoir Analysis used HEC-5 to explore operational criteria changes, additional detail is provided regarding the HEC-5 model, its model limitations, and available storm event inputs.

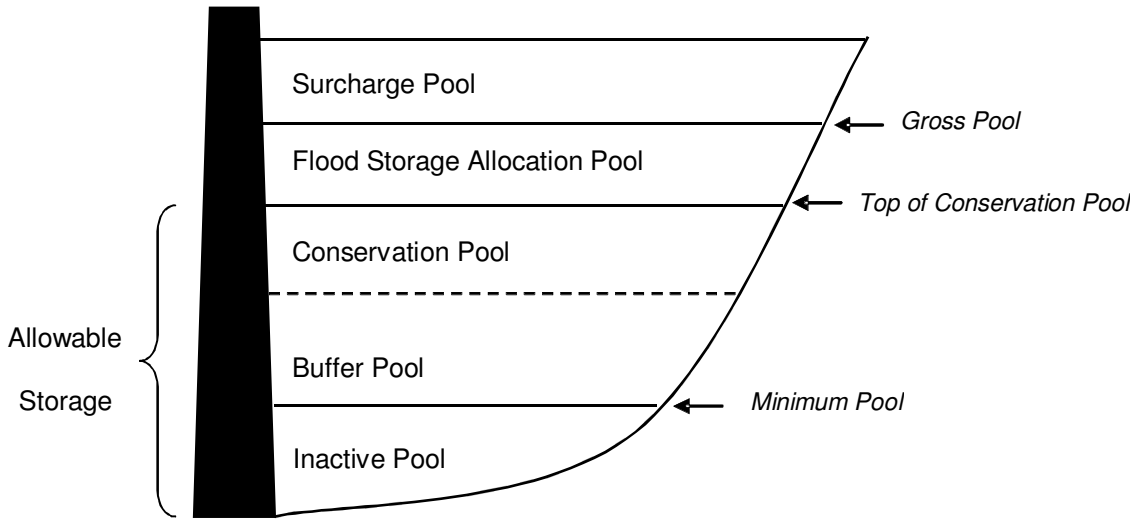
3.5.1 HEC-5 Model Specifications

HEC-5, a computer program first developed and distributed in 1973, was designed by USACE HEC to offer guidance in real-time reservoir release decisions and to aid in planning studies for proposed reservoirs, operation alternatives, and flood space allocation based on specified project demands and constraints. HEC-5 can simulate a dendritic reservoir system configuration of streams, weirs, bypasses, and storage areas. The program accepts criteria related to flood operations, hydropower generation, river routings, diversions, and low-flow operations. Simulations can be performed using time steps ranging from 5 minutes to 1 month.

With support from the USACE Water Management Section of the Sacramento District, HEC constructed working HEC-5 models for flood damage reduction reservoirs within the Central Valley. The Water Management Section began detailed modeling in 1999 to expand the working models into calibrated models capable of performing reservoir simulations for an entire watershed under hydrologic conditions of differing return frequencies and storm centerings.

HEC-5 routes flow through reservoirs based on operational criteria provided by the modeler. Operational criteria in the No Project HEC-5 models strictly observe guidelines established within each reservoir's water control manual and focus on flood damage reduction operations, as well as winter operations for water supply and hydropower. Figure 3-5 shows the basic operational zones of a reservoir in HEC-5.

Under normal conditions, when reservoir storage begins to encroach into the flood storage allocation pool (i.e., storage exceeds the top of conservation pool), reservoir outflow is ramped up to match the inflow, but not to exceed the objective release to evacuate water from the flood storage allocation pool. The objective release is based on downstream channel capacity and reservoir outlet capacity. If inflow into a reservoir is greater than outflow, the volume of water in the reservoir continues to increase, and emergency spillway releases (which are greater than objective releases) begin when storage reaches the gross pool.



Adapted from Hickey et al., 2003
Inactive Pool – Storage in this pool may be zero or a minimum pool.
Buffer Pool – This is part of the conservation pool; when the water level drops into the buffer pool, only essential demands will be met.
Conservation Pool – Space is reserved for various water demands on the reservoir (e.g., agricultural, municipal).
Flood Storage Allocation Pool – Water is stored in this pool when it cannot be safely passed downstream within objective flow targets.
Surcharge Pool – Water in this pool is above the emergency spillway; outflows are determined by the spillway capacity or Emergency Spillway Release Diagram.

Figure 3-5. Basic Operational Zones of a Reservoir in HEC-5

Four separate HEC-5 models were used for the 2012 CVFPP Reservoir Analysis: two for the Sacramento River system and two for the San Joaquin River system. Each system has one model that represents the headwater reservoirs and a second model for the lower basin flood management facilities. The headwater model for each basin generally contains reservoirs located upstream from flood damage reduction projects. Lower basin models contain flood reduction projects as well as water supply, recreation, and hydropower facilities. Reservoirs simulated in the HEC-5 models either currently have flood damage reduction functions or maintain an active storage of greater than 10,000 acre-feet and regulate a significant natural drainage area. The operations of lower basin reservoirs are based on their respective water control manuals. Water control manuals are prepared by USACE for each reservoir that has variable allocations for flood control during the year. Water control manuals also specify reservoir inflow parameters, and contain notes prescribing the use of storage space in terms of release schedules, runoff, nondamaging or other controlling flow rates downstream from the damsite, and other major factors as appropriate.

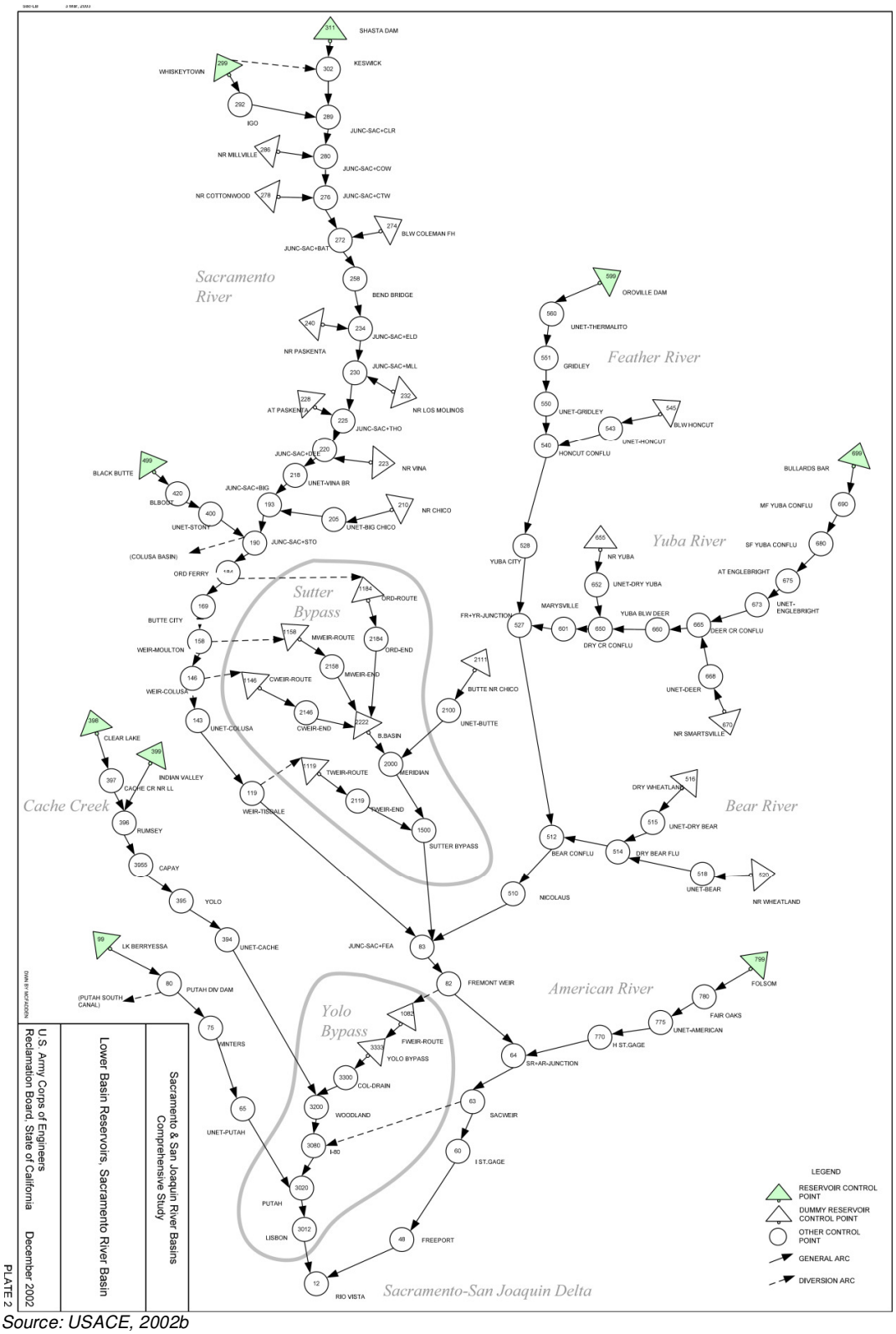
These models can be run for various storm centerings. As described above, 1 storm centering for each basin was used for the 2012 CVFPP Reservoir Analysis. Storm centerings are defined according to the location in the

basin where the highest intensity floodflows occur, although a storm may occur throughout the basin. The process used to analyze each storm centering is described in Attachment 8A: Hydrology. An overview of the storm centerings is provided later in this section.

In the lower basin models, HEC-5 applies Muskingum routing (hydrologic routing) to simulate river routing that delays and attenuates flows as water travels downstream from a reservoir through river reaches. Travel times and attenuation factors were determined through past studies, comparison with historical flood hydrographs, communication with local water agencies, and channel characteristics. The routing coefficients were assumed to be the same for all storm AEPs.

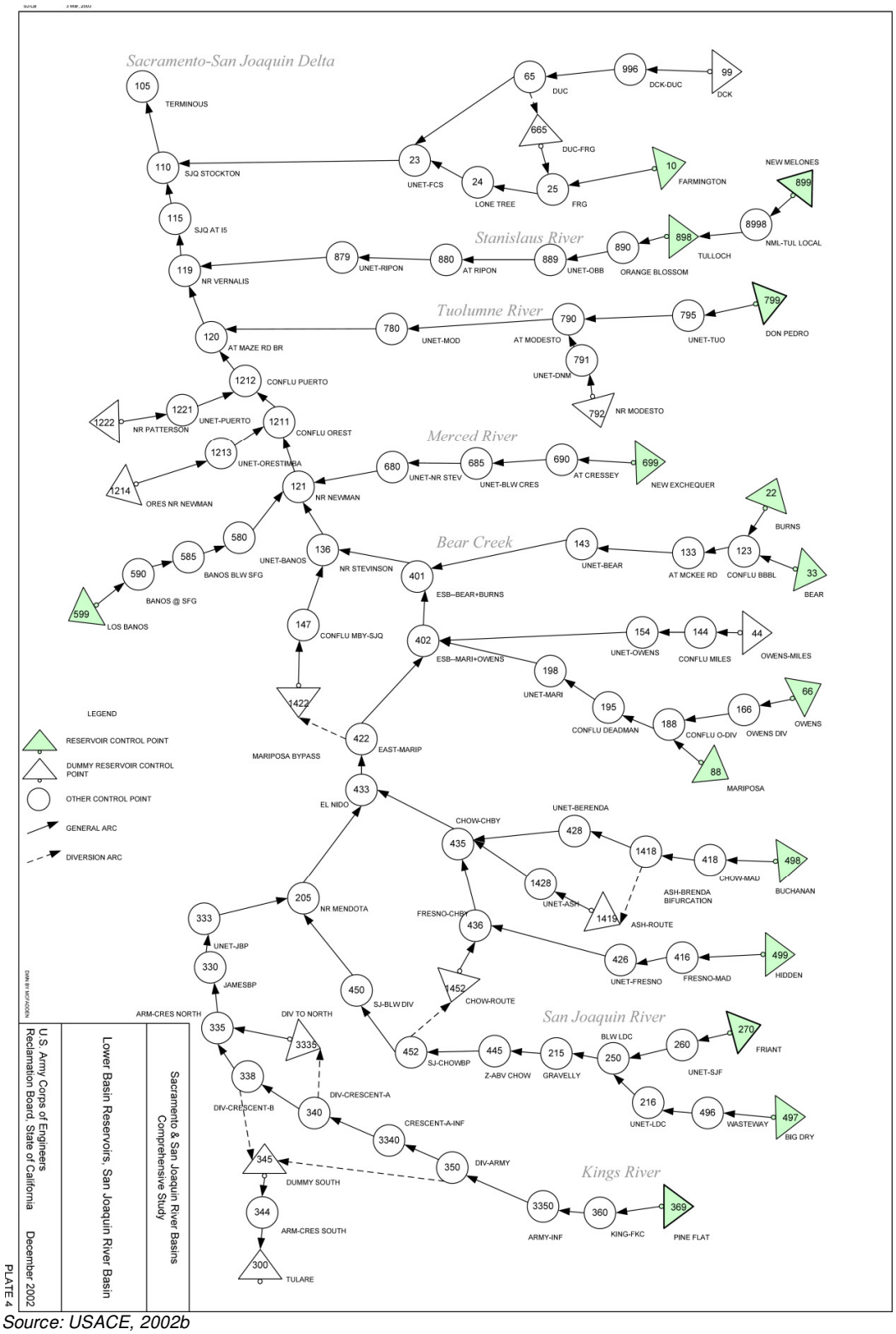
Figures 3-6 and 3-7 are HEC-5 lower basin model schematics for the Sacramento and San Joaquin river basins, respectively. The triangle symbols represent reservoirs and riverine control points; circles represent other control points.

HEC-5 requires a reservoir to be located at the most upstream location in a subreach; hence, riverine control points are represented as pseudo reservoirs (also known as dummy reservoirs). Pseudo reservoirs do not model physical reservoirs, nor do they have any storage. They are a modeling artifact for locations that receive diverted flows; flows simply pass through these locations without any regulation. Table 3-5 lists reservoirs, as well as important notes and assumptions, simulated in the HEC-5 lower basin model for the Sacramento River Basin. Table 3-5 also shows a similar list for the San Joaquin River Basin.



Source: USACE, 2002b

Figure 3-6. HEC-5 Schematic for Sacramento River Basin



Source: USACE, 2002b

Figure 3-7. HEC-5 Schematic for San Joaquin River Basin

HEC-5 Model Limitations

The HEC-5 models represent Year 2000 reservoir operational criteria within the current flood management systems of the Sacramento and San Joaquin river basins. HEC-5 simulates the regulated flow time series for hydraulic models (UNET) to perform detailed downstream hydraulic routing. These models, developed for the Comprehensive Study, were updated as necessary for the 2012 CVFPP Reservoir Analysis.

The hydrologic routing of HEC-5 allows modeling of floodflow conditions along the river mainstem below the reservoirs. More detailed hydraulic models are required to predict site-specific flow conditions. UNET models are the appropriate hydraulic tools to predict flow rates and water stages at various riverine locations inside the Sacramento and San Joaquin river basins. However, the HEC-5 models provide reconnaissance-level flow evaluation of river mainstems for prefeasibility studies.

These HEC-5 models have the following key assumptions and limitations:

- Models were developed for use only with synthetic hourly hydrographs from January 1 through February 4. To simulate other time steps or series, adjustments may need to be made.

- FEMA requires that the starting storage of any headwater reservoir be established as that reservoir's gross pool for floodplain studies. However, the Comprehensive Study simulations established starting storages of the headwater reservoirs as an average of their storages during the 1997, 1995, and 1986 Central Valley storm events. If the average storage thus computed was greater than gross pool, gross pool was used as the starting storage.

- For the lower basin reservoirs, the starting storage was at the top of conservation pool. This assumes a maximum basin wetness and thus, the required maximum available flood space.

Table 3-5. HEC-5 Lower Basin Reservoirs in Sacramento and San Joaquin River Basins

Reservoir	River	Owner	Objective Flow	Gross Pool Storage (TAF)	Maximum Flood Space (TAF)	Notes
Sacramento River Basin						
Shasta Dam and Lake	Sacramento River	Reclamation	Below dam – 79,000 cfs Bend Bridge – 100,000 cfs	4,552	1,300	
Whiskeytown Dam and Lake	Clear Creek	Reclamation	N/A	241	N/A	No formalized flood space
Black Butte Dam and Lake	Stony Creek	USACE	Below dam – 15,000 cfs	144	136	Up to 40 TAF of storage can be transferred based on storage in East Park and Stony Gorge
Oroville Dam and Lake Oroville	Feather River	DWR	Below dam – 150,000 cfs Gridley – 150,000 cfs Yuba City – 180,000 cfs Feather-Yuba River Junction – 300,000 cfs Nicolaus – 320,000 cfs	3,538	750	
New Bullards Bar Dam and Reservoir	Yuba River	YCWA	Below dam – 50,000 cfs Marysville at Yuba River – 180,000 cfs	970	170	
Folsom Dam and Lake	American River	Reclamation	Below dam – 115,000 cfs	975	670	Up to 200 TAF of storage can be transferred based on storage in French Meadows, Hell Hole, and Union Valley
Clear Lake and Cache Creek Dam	Cache Creek (South Fork)	YCFC&WCD	N/A	314	150	No formalized flood space, but YCFC&WCD holds appropriative rights for up to 150 TAF per year.
Indian Valley Dam and Reservoir	Cache Creek (North Fork)	YCFC&WCD	Below dam – 10,000 cfs Rumsey – 20,000 cfs	301	40	
Monticello Dam and Lake Berryessa	Putah Creek	Reclamation	Below dam – 16,000 cfs	1,564	N/A	No formalized flood space

Table 3-5. HEC-5 Lower Basin Reservoirs in Sacramento and San Joaquin River Basins (contd.)

Reservoir	River	Owner	Objective Flow	Gross Pool Storage (TAF)	Maximum Flood Space (TAF)	Notes
San Joaquin River Basin						
Pine Flat Dam and Lake	Kings River	USACE	Kings River North – 4,750 cfs Kings River South – 3,200 cfs	1,000	475	Up to 162 TAF of storage can be transferred based on storage in Courtright and Wishon
Big Dry Creek Dam and Reservoir	Dry Creek	FMFCD	Wasteway – 700 cfs	30	30	Has been historically used for flood management, but cannot always be relied on
Friant Dam and Millerton Lake	San Joaquin River	Reclamation	Little Dry Creek – 8,000 cfs Mendota Gage – 6,500 cfs	521	170	Up to 85 TAF of storage can be transferred based on storage in Mammoth Pool
Hidden Dam and Hensley Lake	Fresno River	USACE	Fresno River at Madera Canal – 5,000 cfs	90	65	
Buchanan Dam and H.V. Eastman Lake	Chowchilla River	USACE	Below dam – 7,000 cfs Chowchilla River at Madera Canal – 7,000 cfs	151	45	
Mariposa Dam and Reservoir	Mariposa Creek	USACE	N/A	15	15	
Owens Dam and Reservoir	Owens Creek	USACE	N/A	4	4	
Bear Dam and Reservoir	Bear Creek	USACE	N/A	8	8	
Burns Dam and Reservoir	Burns Creek	USACE	N/A	7	7	
New Exchequer Dam and Lake McClure	Merced River	MID	Cressey – 6,000 cfs	1,025	350	
Los Banos Dam and Detention Reservoir	Los Banos Creek	Reclamation	Los Banos – 1,000 cfs	35	14	
New Don Pedro Dam and Reservoir	Tuolumne River	TID	Modesto (Tuolumne River below Dry Creek) – 9,000 cfs	2,030	340	
New Melones Dam and Lake	Stanislaus River	Reclamation	Orange Blossom Bridge – 8,000 cfs	2,420	450	

Table 3-5. HEC-5 Lower Basin Reservoirs in Sacramento and San Joaquin River Basins (contd.)

Reservoir	River	Owner	Objective Flow	Gross Pool Storage (TAF)	Maximum Flood Space (TAF)	Notes
Tulloch Dam and Reservoir	Stanislaus River	Oakdale, So. San Joaquin ID	Orange Blossom Bridge – 8,000 cfs	68	10	Flow-through reservoir; generally releases are the same as New Melones except in high flows
Farmington Dam and Reservoir	Littlejohns Creek	USACE	Town of Farmington – 2,000 cfs	52	52	

Key:
cfs = cubic feet per second
DWR = California Department of Water Resources
FMFCD = Fresno Metropolitan Flood Control District
ID = Irrigation District
MID = Merced Irrigation District
N/A = Not applicable, no specified objective releases or flood storage allocation
Reclamation = U.S. Department of the Interior, Bureau of Reclamation
TAF = thousand acre-feet
TID = Turlock Irrigation District
USACE = U.S. Army Corps of Engineers
YCFC&WCD = Yolo County Flood Control & Water Conservation District

- Guidelines established within each reservoir's water control manual were strictly observed.

- Some reservoirs with stepped release schedules rely on both the percentage of required flood control space used and peak inflow in determining flood releases. For these reservoirs, fixed percentages of required flood control space used were assumed.

- Muskingum routing parameters were fixed for all simulated exceedence frequencies.

- Local flows were either synthetically produced or were assumed to be a ratio of the short duration maxima of a nearby natural flow hydrograph. These ratio multipliers were not scaled for each simulated exceedence frequency. For more detailed studies, variable ratio multipliers based on floodflow frequency should be examined.

- Calibration and verification were accomplished using Central Valley flood events in 1995 and 1997 and by comparing these to manual routings published in water control manuals.

- It was assumed that all river channels have infinite capacity (i.e., all flows would be routed through the channels even if channel capacity was exceeded). No losses, such as evaporation, seepage, and overbank flow due to levee breaks, were simulated.

- HEC-5 cannot integrate concisely some of the operating criteria for some reservoirs. The multiparameter "Release Schedules" for Black Butte, Shasta, and Oroville lakes had to be written into the model by assuming one of the variable parameters to be constant. Similar difficulties with Black Butte Dam (Ord Ferry) required that an operational point be excluded from the simulations. Complications with the forecast capabilities of HEC-5 required that one of the operating points of Friant Dam be located outside the program's forecast window.

- The simulation program assumed near certainty in flow contributions from downstream tributaries when operating facilities for flows at that location or downstream from that location. Uncertainty in forecasting downstream flow contributions should be addressed in a risk analysis along with other variables affecting the operational efficiency of a reservoir.

For more information about the capabilities of the HEC-5 simulation program and its basic assumptions and limitations, refer to the October

HEC-5 Simulation of Flood Control and Conservation Systems User's Manual (USACE, 1998) and the December *Comprehensive Study Reservoir Simulation Models User's Guide* (USACE, 2002b).

Updates to Models

Changes were made to the Comprehensive Study HEC-5 models to include the Feather-Yuba F-CO program and Folsom Dam JFP modifications. It was assumed that implementing the SJRRP had no effect on flood operational criteria at Millerton Lake.

Feather-Yuba F-CO Program The goal of the F-CO program is to improve flood protection for communities along and downstream from the Yuba and Feather rivers without impacting the water supply of Lake Oroville and New Bullards Bar Reservoir. This was accomplished through reducing peak floodflows via improved river flow forecasting and improved operational coordination between Lake Oroville and New Bullards Bar Reservoir (YCWA, 2008).

To incorporate these changes into the model, the following two downstream control points for which New Bullards Bar Reservoir is operated were added to the HEC-5 model (as specified in the Reservoir Operations (RO) Points record): confluence of Yuba and Feather rivers, and Feather River at Nicolaus. Adding these operational criteria points means that when channel capacity is close to being exceeded at these control points, Lake Oroville and New Bullards Bar Reservoir will modify their releases based on available flood storage space to maintain channel capacity. To meet downstream channel capacities, the reservoir with the largest percentage of allocated flood storage still available would lower its releases more than the other reservoir.

Folsom Dam JFP As mentioned, the Folsom Dam JFP is a collaborative effort by Reclamation and USACE to address dam safety hydrologic risk at Folsom Lake and improve flood protection. Among other modifications, this project will include a new auxiliary spillway, a change in Folsom Lake operational criteria capabilities provided by the new auxiliary spillway, improved weather forecast products, and alternative variable storage options. The following text briefly summarizes key changes to Folsom Lake operational criteria. Note that all routing assumptions documented in support of design decisions are subject to further refinement or optimization efforts via the Folsom Dam Permanent Operations (FPO) Study. For more information on the changes to Folsom Lake, refer to the *Folsom Dam Auxiliary Spillway Control Structure Draft Design Documentation Report* (USACE, 2009) and http://www.usbr.gov/mp/jfp/ (Reclamation, 2009).

While the preferred method for incorporating Folsom Dam JFP changes would be to modify HEC-5, this did not accurately reflect the Folsom Dam JFP. HEC-5 was not capable of accurately simulating the variable release diagram and design targets associated with the Folsom Dam JFP. As a result, the HEC-ResSim model of the American River, developed by USACE, was used to simulate releases from Folsom Lake. More details on incorporating the Folsom Dam JFP into the model are provided in the following HEC-ResSim subsections.

Storm Events

There were seven AEP storm events developed for the Comprehensive Study and were available to use for the 2012 CVFPP Reservoir Analysis (Table 3-6). Another way of representing AEP is to use the inverse of the percent exceedence to describe the exceedence probability of a storm or flood using a return period, which is the long-term expected return period for a given exceedence.

Table 3-6. Comprehensive Study Simulated Frequency Events

AEP (percent)	Return Period
50	2-year
10	10-year
4	25-year
2	50-year
1	100-year
0.5	200-year
0.2	500-year

Key:
AEP = annual exceedence probability

In the HEC-5 Sacramento River Basin model, the following storm centers were developed for the Comprehensive Study:

- Shasta Lake to Ord Ferry (Shasta centered)
- Sacramento River at latitude of Ord Ferry[2] (Ord Ferry centered)
- Yuba River near Marysville (Yuba centered)
- Feather River at Oroville (Oroville centered)
- Sacramento River at latitude of Sacramento (Sacramento centered)

[2] All “at latitude” locations represent mainstem storm runoff centerings.

- American River at Fair Oaks (American centered)

In the HEC-5 San Joaquin River Basin model, the following storm centers were developed for the Comprehensive Study:

- San Joaquin River at Friant (Friant centered)
- San Joaquin River at latitude of El Nido (El Nido centered)
- San Joaquin River at latitude of Newman (Newman centered)
- San Joaquin River at latitude of Vernalis (Vernalis centered)
- Merced River at Exchequer (Exchequer centered)
- Tuolumne River at Don Pedro (Don Pedro centered)

According to Phase 1 objectives, which were to gain a high-level understanding of the two basins and run preliminary reservoir operational criteria change simulations, the storm events applied are essentially the same as those described above, except the following:

- The 50 percent AEP events were not evaluated because it was anticipated that both the Sacramento and San Joaquin river basins can safely pass flows resulting from such frequent events.
- It was recognized that while individual tributary storm centers could generate very different flow conditions for local tributaries, from a basin-wide perspective (which is the focus of CVFPP), tributary storm centers that are relatively close together would likely result in similar peak flow conditions along the Sacramento and San Joaquin rivers. Storm centers for the Feather River at Oroville in the Sacramento River Basin and the Tuolumne River at Don Pedro in the San Joaquin River Basin were not evaluated because of the proximity of the storm centers to the Yuba River near Marysville and the Merced River at Exchequer, respectively.

For Phase 2, fewer AEPs and storm centerings were selected to efficiently analyze a wide range of operational criteria changes while gaining a better understanding of how the system would react to these specific operational criteria changes. Storm frequencies for the Phase 2 analysis were selected based on the ability of the reservoirs in the basin to manage floodflows, and of the Sacramento and San Joaquin rivers to convey flows within the channel capacity. For the Sacramento River Basin, the 1 and 0.5 percent AEP storms were chosen to compare reservoir operational criteria

scenarios. These AEPs were chosen because the channel capacity was generally not exceeded for the No Project condition in the Feather River Basin (which was the focus of Phase 2 changes) for storms that occurred more frequently than a 1 percent AEP. If channel flows were within channel capacity, it was assumed that the system can safely convey the water without flooding adjacent areas. Because flow was within channel capacity, operational criteria changes would not affect the volume of flooding. While the 0.5 percent AEP storm occurs infrequently, and any benefit derived from operational changes would be minimal when distributed over the frequency of occurrence of large floods, it was included in this analysis because reservoir operational criteria changes have the potential to noticeably lower the channel flow rate in the Sacramento River Basin for a 0.5 percent AEP storm. The 0.2 percent AEP storm was not used in the comparison because of the storm's extremely infrequent nature.

For the San Joaquin River Basin, the 2 and 1 percent AEP storms were chosen for preliminary comparisons of the reservoir operational criteria change scenarios. Because of the generally lower channel capacity of this basin, storms that occur more frequently were selected. The channel capacity was exceeded for the No Project condition in the downstream portion of the San Joaquin River for storms that occurred more frequently than a 2 percent AEP. The 0.5 and 0.2 percent AEP storms were not used in the comparison because, as seen during the Phase 1 analysis, the magnitudes of these storms were so large that reservoir operational criteria changes alone would not be sufficient to keep flows within the channel capacity of most streams in the basin.

The storm centerings used in Phase 2 to compare the No Project condition with reservoir operational criteria changes for the Sacramento and San Joaquin river basins are the Sacramento and Vernalis storm centerings, respectively. These storm centerings were selected because they resulted in the highest simulated river stages (as determined using UNET) basin-wide for a majority of the AEPs (refer to Attachment 8C: Riverine Channel Evaluations for more details regarding UNET modeling). Selecting one centering for each basin allowed the simulated effects of reservoir operational criteria changes throughout the basin to be easily compared.

Locations

In the Sacramento River Basin, observations at index points throughout the basin were used to demonstrate potential peak flow reduction from reservoir operational criteria changes (Table 3-7 and Figure 3-8).

The Sacramento River at Ord Ferry was used in Phase 1 to indicate the effects of changes to Shasta Lake operational criteria. Yuba City and

Marysville were selected because they had both previously experienced serious flooding, and river flows at these two locations are indicative of the effects of Oroville and New Bullards Bar reservoirs' operational criteria changes, respectively. The confluence of the Feather and Yuba rivers and Nicolaus were chosen to better describe the additive effect of the changes in operational criteria to Oroville and New Bullards Bar reservoirs. Changes in operation at Folsom Lake would affect the American River at the H and I Street gages. Locations on the Sacramento River downstream from the Fremont Weir and at Freeport were selected to describe the collective effects to the Sacramento River from operational criteria changes for multiple upstream reservoirs.

Table 3-7. Sacramento River Index Point Locations for HEC-5 Analysis

Index Point	Phase 1	Phase 2
Sacramento River Downstream from Ord Ferry	X	N/A
Feather River at Yuba City	X	N/A
Yuba River at Marysville	X	N/A
Confluence of Feather and Yuba Rivers	X	X
Feather River at Nicolaus	X	X
Sacramento River Downstream from Fremont Weir	X	N/A
Confluence of Sacramento and Feather Rivers	N/A	X
Sacramento River at I Street Gage	N/A	X
Sacramento River at H Street Gage	X	N/A
At Lake Oroville	N/A	X
Sacramento River at Freeport	X	N/A

Key:
N/A = not applicable

Phase 2 mainly focused on flow at the confluence of the Feather and Yuba rivers to observe the preliminary effects of reservoir operational criteria changes. Flow at the confluence of the Feather and Yuba rivers is the farthest upstream location influenced by coordinated operations of both Lake Oroville and New Bullards Bar Reservoir, the two reservoirs analyzed in this phase. Once the scenarios for further consideration were identified, flow effects at four additional index point locations were observed (Table 3-7).

In the San Joaquin River Basin, observations at index points throughout the basin were used to demonstrate potential peak flow reduction from reservoir operational criteria changes (Table 3-8 and Figure 3-9).

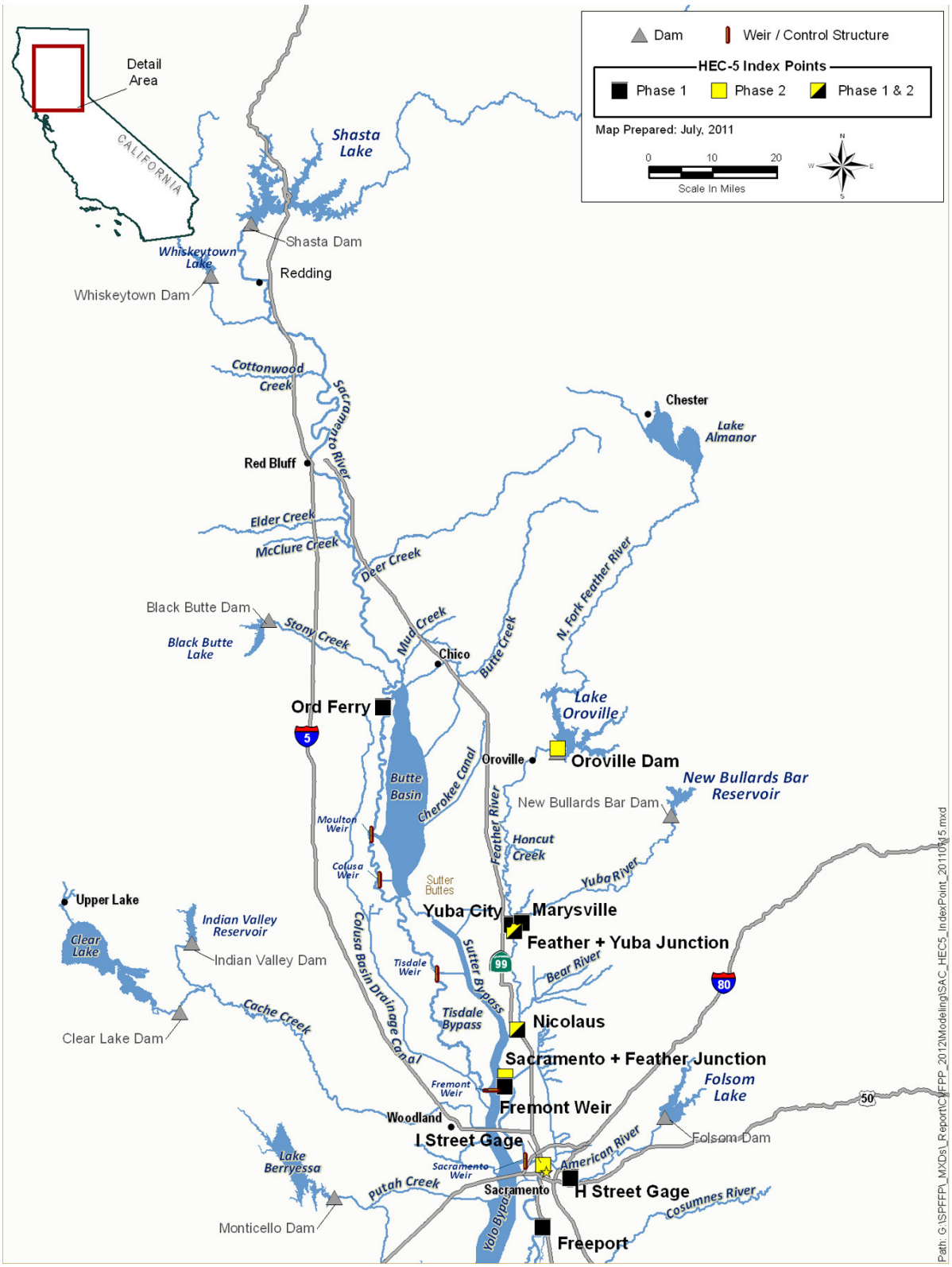


Figure 3-8. Sacramento River Basin HEC-5 Index Point Locations

Table 3-8. San Joaquin River Index Point Locations for HEC-5 Analysis

Index Point	Phase 1	Phase 2
Near Mendota	X	N/A
Chowchilla Bypass near Fresno River	N/A	X
El Nido	X	X
Near Newman	X	X
At Maze Road Bridge	N/A	X
Near Vernalis	X	X
Stockton	X	X

Key:
N/A = not applicable

The index point near Mendota was selected because it is downstream from Millerton Lake. For Phase 2, the Chowchilla Bypass near the Fresno River was selected because most of the floodflows would be routed through the Chowchilla Bypass. El Nido, near Newman, at Maze Road Bridge, and near Vernalis are located on the mainstem of the San Joaquin River and were chosen because they are located downstream from the confluences of tributaries with major multipurpose reservoirs. Stockton was selected because it is the most downstream location in the HEC-5 model and would show the collective effects of multiple reservoir operational criteria changes.

Similar to the Sacramento River Basin, Phase 2 mainly focused on flow at one location in the San Joaquin River Basin, at Stockton, to observe the preliminary effects of reservoir operational criteria changes. Once the scenarios for further consideration were identified, flow effects at five additional index point locations were observed (Table 3-8).

Operational Criteria Changes

Changes in reservoir operational criteria were incorporated into HEC-5 for multipurpose reservoirs within the Central Valley. Similar to the Comprehensive Study, this 2012 CVFPP Reservoir Analysis assumed that the most likely operational criteria changes would be as follows:

- Changes to the flood management rule curves (i.e., increasing the amount of space dedicated to flood storage)
- Changes to the objective flow to which the reservoir is operated

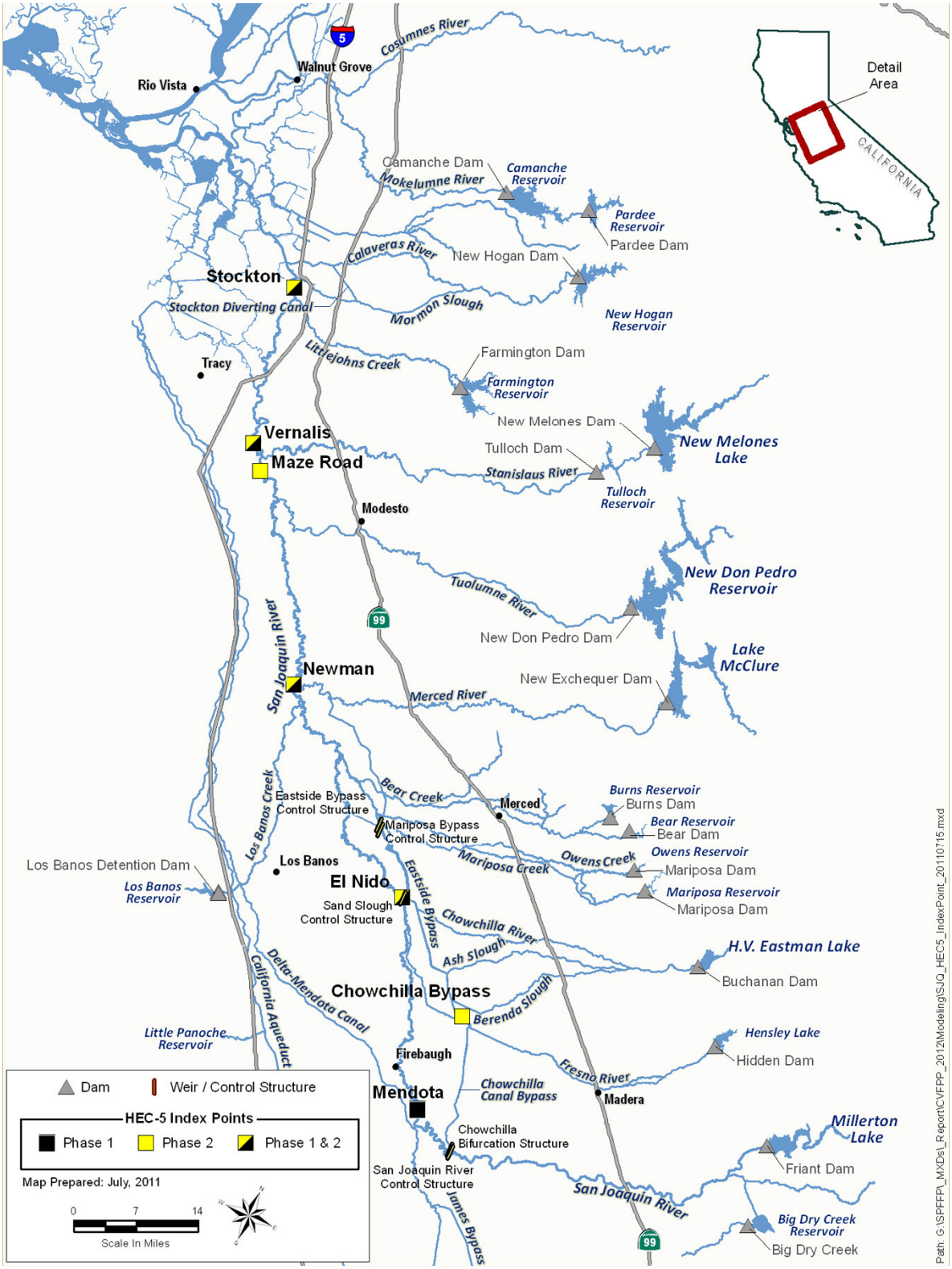


Figure 3-9. San Joaquin River Basin HEC-5 Index Point Locations

The flood management rule curve used in HEC-5 was modified through increasing the amount of required flood space in a reservoir by lowering the parameters in the model that represent the top of conservation pool (see Figure 3-5 for a simple flood rule curve). While increasing the required flood space could also be achieved through physical changes, no modifications to the total reservoir capacity or appurtenances were made for this analysis, but some reservoirs required modification of spillway operation parameters in HEC-5 for operational criteria consistent with the new flood storage level.[3]

For each scenario, it was assumed that the starting storage for all lower basin reservoirs was at the top of conservation pool; hence, increasing the available flood storage decreased the starting storage for each reservoir.

Decreasing the objective release in the HEC-5 models would lower the magnitude of flows being released from a reservoir until reservoir storage reached gross pool and emergency spillway operations began. Objective releases were decreased by lowering the maximum flow limit at downstream operating points and downstream channel capacities based on reservoir level. Reservoir diversions and gate regulations associated with flow rates were also modified, when applicable.

Systemwide Peak Flow Reduction

As described, HEC-5 was used to observe the effect of changes to reservoir operational criteria on peak flow at key index point locations throughout the basins. The peak flows are not the exact flows that would occur in an actual flood because the channel routing in HEC-5 simulates attenuation and travel time, but not losses from the channel. As a result, levee breaks are not included in the model, but for downstream locations and large storm events, it is possible, or even likely in some cases, that levee breaks would have occurred upstream, thereby reducing flows in the downstream reaches of the river. This analysis focuses on the relative change in downstream peak flows resulting from scenarios that simulated changes in reservoir operational criteria, and not absolute simulated peak flows.

[3] Scenarios that lower the top of conservation pool become 50 percent encroached at a lower volume, causing emergency spillway operations to begin at an earlier time. For example, if the original top of conservation pool is at 100 TAF and the gross pool is at 200 TAF, the reservoir is 50 percent encroached when the volume is 150 TAF. If the top of conservation pool is lowered by 50 TAF, emergency spillway operations would begin at 125 TAF (50 percent encroached). Instead of gate operations being related to the percentage encroached, this analysis assumed that emergency spillway operations began at the same volume as for the No Project condition. As a result, for this example, the HEC-5 data file was modified such that emergency spillway operations occurred at 150 TAF in both cases (i.e., at 67 percent encroached in the scenario).

The timing, magnitude, and duration of flow into rivers and tributaries varies, depending on the storm centering and AEP; hence, one operational criteria change would not always have the same effect at every index point location. As a result, for each basin, Phase 2 focused on only one storm centering (Sacramento storm centering for the Sacramento River Basin and Vernalis storm centering for the San Joaquin River Basin) and two AEPs during the basin-wide sensitivity analysis to better compare the effects of operational criteria changes.

3.5.2 HEC-ResSim Model Specifications

USACE has been developing new HEC-ResSim models as part of the DWR and USACE Central Valley Hydrology Study. USACE has completed the calibration of the new HEC-ResSim models for the Sacramento and San Joaquin river basins using HEC-5 Comprehensive Study hydrology. These HEC-ResSim models are currently undergoing quality assurance and quality control; the models have not yet been released to the public (USACE, 2010).

As described above, HEC-ResSim was used to supplement the HEC-5 model because the HEC 5 model (developed during the Comprehensive Study) does not include the Folsom Dam JFP modifications. To simulate Folsom Dam JFP operational criteria effects on Folsom Lake, the following changes to Folsom Lake operational criteria were incorporated into HEC-ResSim: (1) updated model inputs (i.e., spillway ratings and capacity curve), (2) modified flood space requirements, (3) updated Emergency Spillway Release Diagram (ESRD), and (4) changed operational criteria to reflect new design targets.

The updated model inputs include 1997 outlet ratings and new auxiliary spillway with a capacity of 138,519 cfs at elevation 418 feet. Flood space requirements were modified in accordance with the new water control diagram for Folsom Lake. This will reduce the variable flood storage allocation from the current operating range of 400 TAF to 670 TAF to 400 TAF to 600 TAF once improvements to Folsom Dam are completed (according to the federal Water Resources Development Act of 1999). Also, emergency spillway operations were modified to reflect the updated ESRD. Operational criteria for Folsom Dam and Lake were changed to reflect new design targets. These targets included limiting the discharge for the 1 percent AEP storm event to 115,000 cfs, and discharge for the 0.5 percent AEP storm event to 160,000 cfs.

The HEC-ResSim model used to establish the No Project condition at Folsom Lake was developed by USACE and is in draft form with an unknown completion date. USACE is currently refining the HEC-ResSim model used in this analysis, which will include all of the Folsom Dam JFP

modifications listed above. While incomplete, this HEC-ResSim model was selected because it is the best available model and, in general, it accurately simulates the changes to Folsom Dam and Lake. Once all storms were routed through HEC-ResSim, the time series of Folsom Dam and Lake releases were input into HEC-5, and the rest of the Sacramento River Basin was simulated.

For more information about the capabilities of this model, refer to the April 2007 *HEC-ResSim Reservoir System Simulation User's Manual* (USACE).

3.5.3 UNET Model Specifications

UNET is designed to simulate 1-D, fully unsteady flow through a full network of open channels, weirs, bypasses, and storage areas. It is a fixed-bed analysis and does not account for sediment movement, scour, or deposition. UNET assumes no exchange with groundwater, but is capable of simulating levee breaks and breaches (USACE, 2002c). For more information about the capabilities of this model, refer to the August 1997 *UNET: One-Dimensional Unsteady Flow Through a Full Network of Open Channels User's Manual* (USACE) and Comprehensive Study Technical Studies Documentation, Appendix C – Hydraulic Technical Documentation (USACE, 2002d).

Separate UNET models were developed for the Sacramento River system and San Joaquin River system. The UNET models can be used to determine river flow, stage, velocity, and depth, as well as breakout and return flows from overbank areas.

Changes made to the UNET model for the 2012 CVFPP studies are documented in Attachment 8C: Riverine Channel Evaluations.

Storm Events

Inputs to the UNET model come from the HEC-5 model; therefore, the same storm centerings were used as for the Phase 2 HEC-5 hydrologic modeling, Sacramento and Vernalis.

Because only two scenarios were validated using UNET, all six (10, 4, 2, 1, 0.5, and 0.2 percent) AEP storms were run to assess the simulated effects of these scenarios on flood management. This enabled a thorough comparison of simulated effects for a range of channel flow magnitudes.

Locations

In the Sacramento River Basin, four index point locations were used to demonstrate the potential stage reduction from the two scenarios (Figure 3-10):

- Feather River and Yuba River confluence
- Feather River at Nicolaus
- Yolo Bypass at Lisbon
- Sacramento River at the I Street gage

The first two locations were selected because they are common flood management operation objectives for both Lake Oroville and New Bullards Bar Reservoir. The Yolo Bypass at Lisbon and Sacramento River at I Street gage are two of the most downstream locations and would show the systemwide effects of reservoir operational criteria changes.

In the San Joaquin River Basin, four index point locations were used to demonstrate potential stage reduction from reservoir operational criteria changes (Figure 3-11):

- San Joaquin River near Newman
- San Joaquin River at Maze Road Bridge
- San Joaquin River near Vernalis
- San Joaquin River at Stockton

The San Joaquin River Basin index points are all located downstream from the Merced River. These locations were selected because they are on the mainstem and would reflect changes to each of the five identified reservoirs' operational criteria (see Table 3-4).

Out-of-channel flow was aggregated for most reaches throughout the Sacramento and San Joaquin river basins.

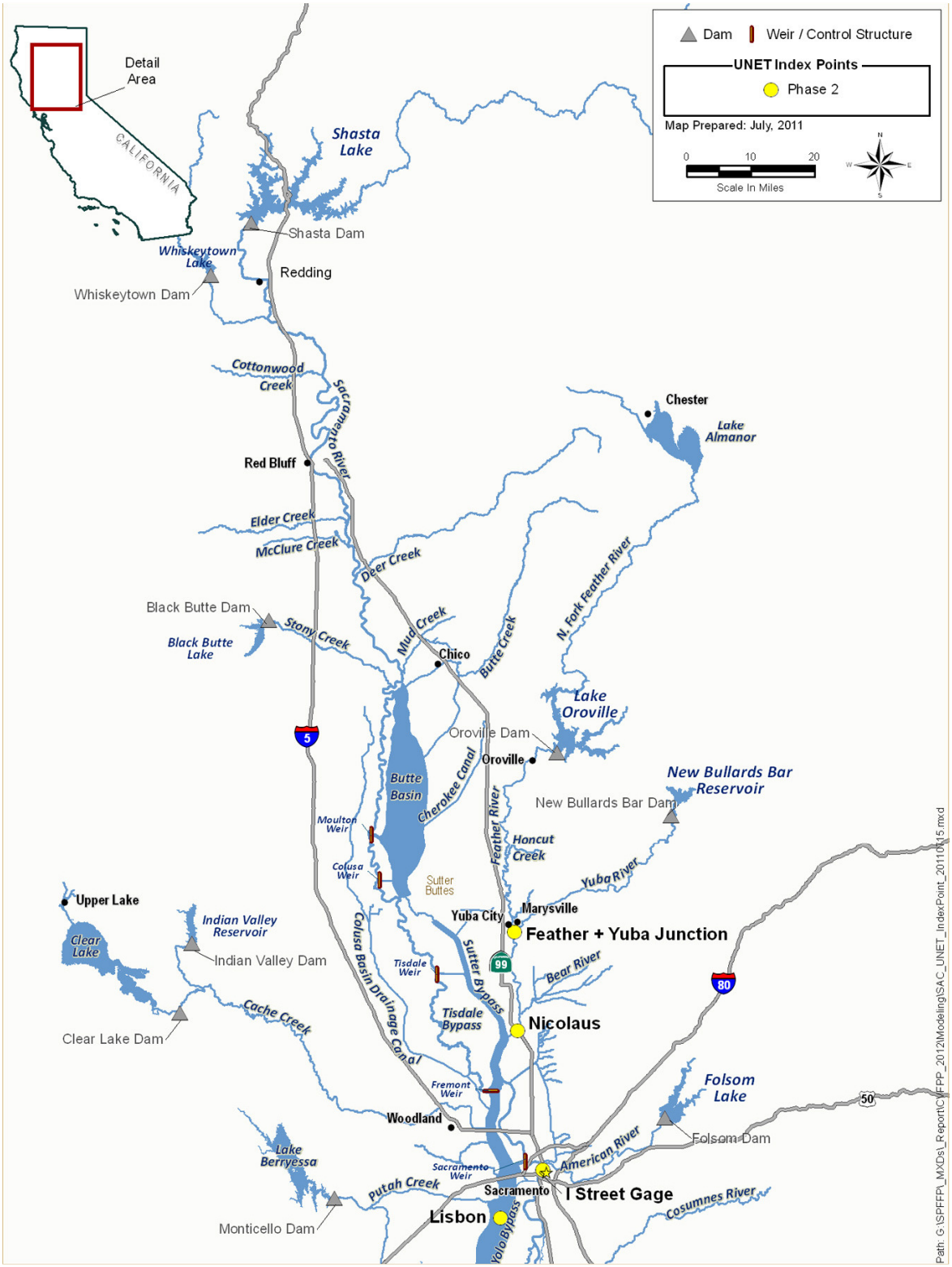


Figure 3-10. Sacramento River Basin UNET Index Point Locations

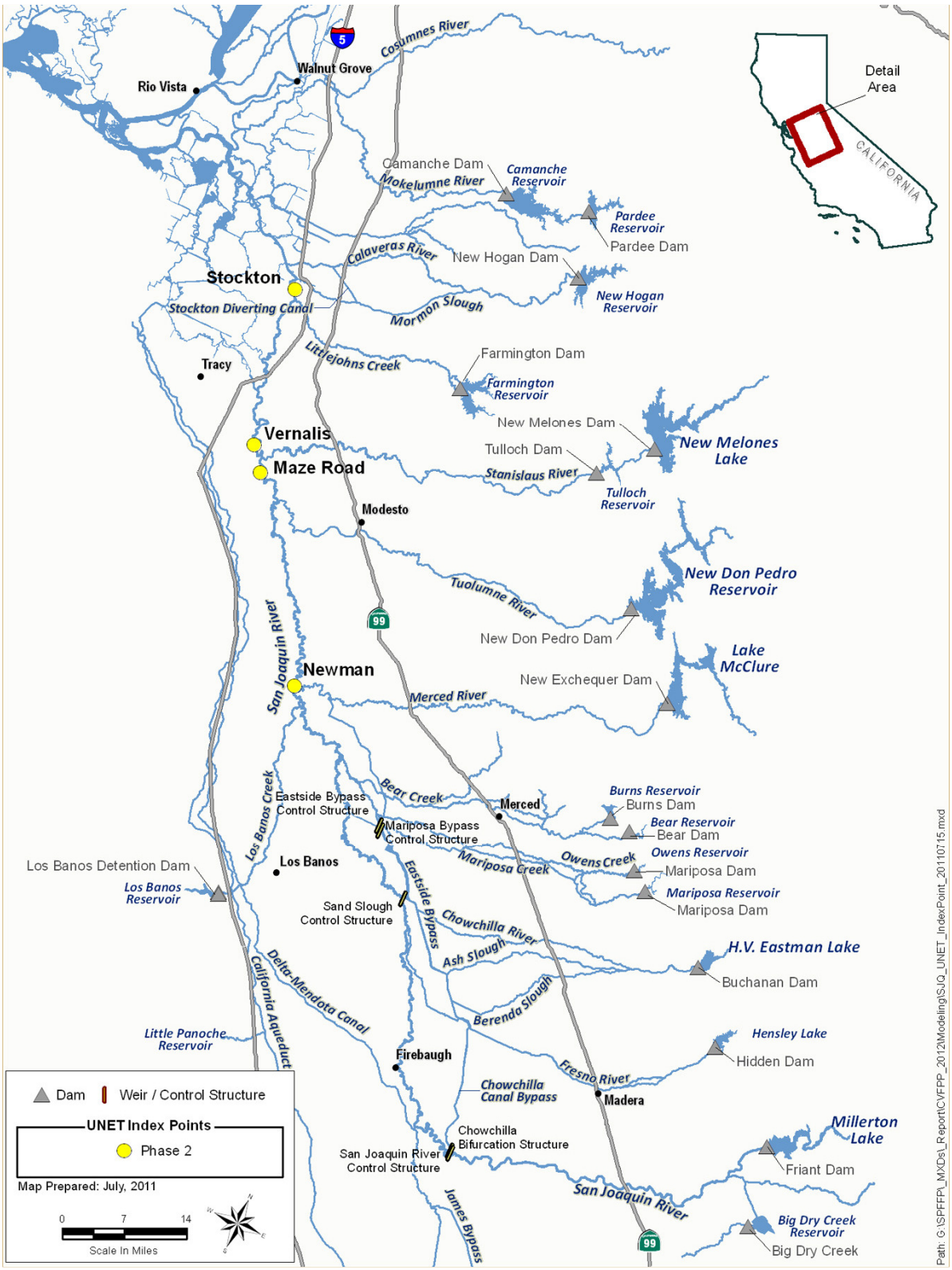


Figure 3-11. San Joaquin River Basin UNET Index Point Locations

4.0 No Project System Performance

This section provides an overview of No Project system performance of the Sacramento and San Joaquin river basins as simulated in HEC-5 (and supplemented by HEC-ResSim for the American River). The ability of reservoirs to manage a range of flood events under their No Project operational criteria is described.

4.1 Sacramento River Basin

As described in Section 3, Sacramento River Basin No Project condition includes the original Comprehensive Study HEC-5 assumptions plus the modifications associated with the Folsom Dam JFP and F-CO program.

Table 4-1 shows HEC-5 simulated results for the No Project condition compared to stated channel capacities. Striped cells in the table indicate peak flows in excess of, but within 3 percent of the channel capacity. Shaded cells in the table indicate peak flows in excess of the channel capacity. The table also shows that the current Sacramento River system can withstand different frequencies of storms, depending on location. For example, on the Feather River, system flood protection would be slightly below a 2 percent AEP storm. At the I Street gage, the objective flow was within 3 percent of its channel capacity for storms with a 1 percent or more frequent AEP.

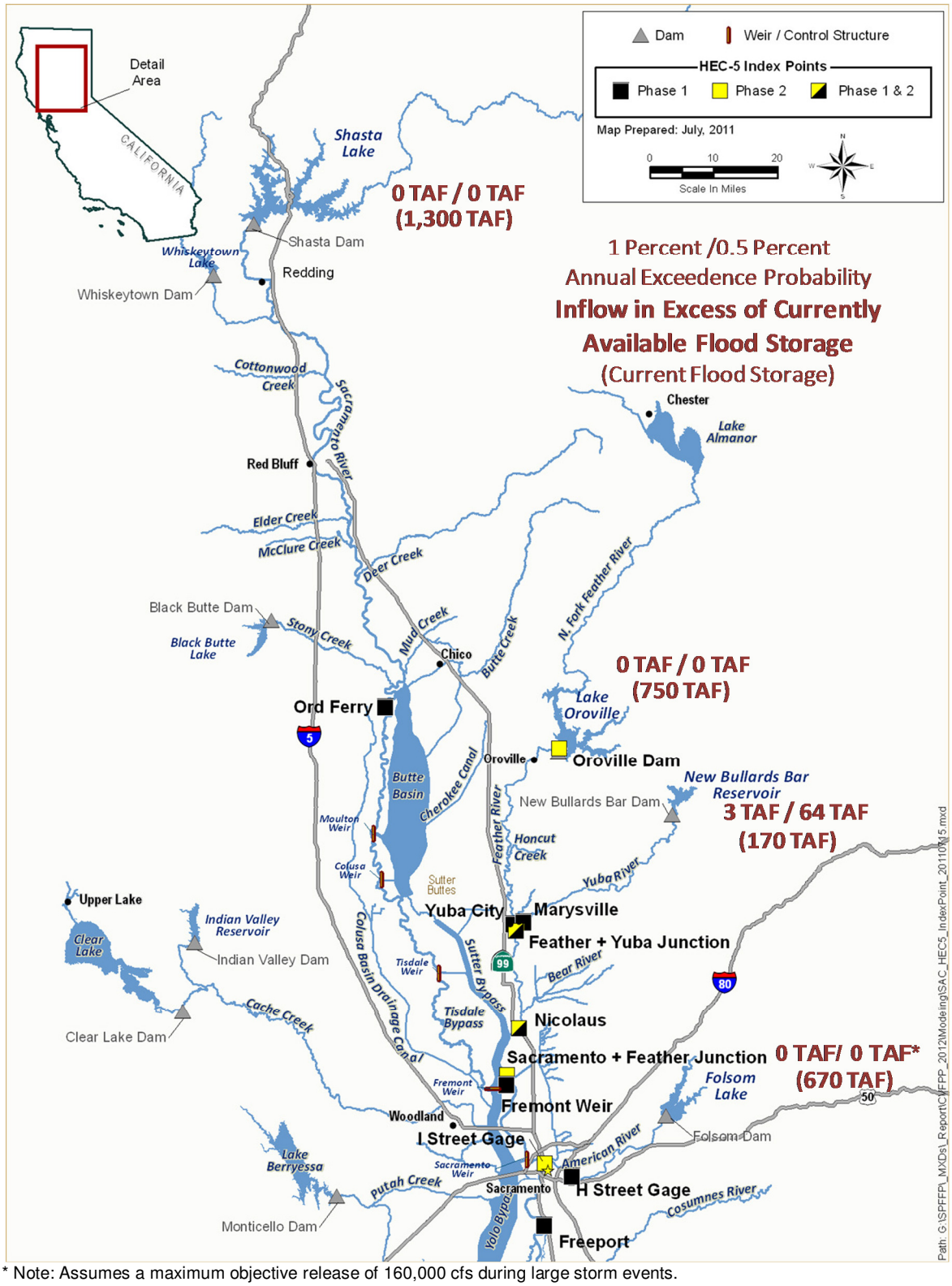
The ability of reservoirs to operate within their objective release also varies depending on storm magnitude. For both 1 and 0.5 percent AEP Sacramento-centered storms, Shasta Lake, Lake Oroville, and Folsom Lake can operate within their objective releases. Unlike other major multipurpose reservoirs, New Bullards Bar Reservoir has a simulated inflow of 3 TAF and 64 TAF in excess of available flood storage that could not be managed for 1 and 0.5 percent AEP Sacramento-centered storms, respectively (Figure 4-1).

Table 4-1. Simulated Sacramento River Basin Objective Flow Exceedence for No Project Condition for Sacramento-Centered Storm

Index Point Location	Peak Flow of Flood Event (cfs)						
	Channel Capacity (cfs)	10 percent AEP	4 percent AEP	2 percent AEP	1 percent AEP	0.5 percent AEP	0.2 percent AEP
Feather and Yuba River Junction	300,000	179,717	270,028	276,070	276,712	359,036	587,901
Feather River at Nicolaus	320,000	208,764	309,737	320,129	327,445	420,103	656,064
Sacramento and Feather River Junction	410,000	323,838	444,372	473,955	499,559	614,891	877,461
Sacramento River near I Street Gage	110,000	95,224	111,611	112,268	112,167	130,042	224,649

Model: HEC-5
Note:
Striped cells indicate peak flows in excess of the channel capacity, but within 3 percent of the channel capacity.
Shaded cells indicate peak flows in excess of the channel capacity.

Key:
AEP = Annual Exceedence Probability
cfs = cubic feet per second
Sacramento-Centered = Storm centered at Sacramento River at latitude of Sacramento



* Note: Assumes a maximum objective release of 160,000 cfs during large storm events.

Figure 4-1. Volume of Inflow in Excess of Currently Available Flood Storage for 1 and 0.5 Percent AEP Sacramento-Centered Storms for No Project Condition

Figures 4-2 and 4-3 show the No Project condition for the Feather and Yuba rivers during 1 and 0.5 percent AEP storms. The Yuba River contributes nearly half of the flow at the confluence of the Feather and Yuba rivers, but less than half of Yuba River flow is regulated by New Bullards Bar Reservoir on the North Fork Yuba River. The figures also show that while Lake Oroville stays within its objective release of 150,000 cfs below the dam for both AEP storms, this high objective release substantially contributes to peak downstream flows.

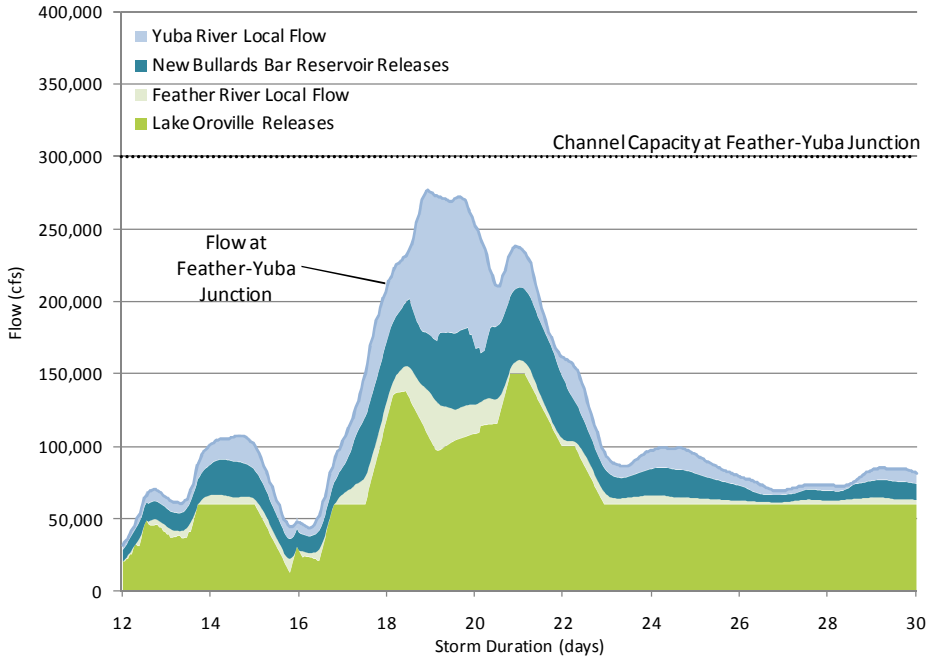


Figure 4-2. Simulated No Project Condition for 1 Percent AEP Sacramento-Centered Storm

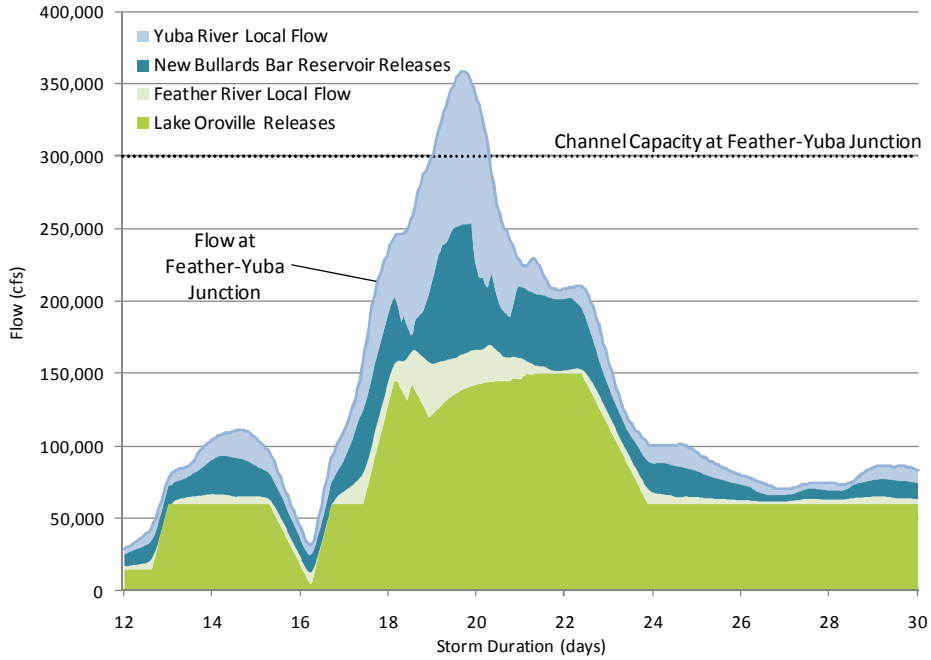


Figure 4-3. Simulated No Project Condition for 0.5 Percent AEP Sacramento-Centered Storm

Observations of system performance for the No Project condition during 1 and 0.5 percent AEP Sacramento-centered storms include the following:

- Lake Oroville is appropriately sized to manage at least a 0.5 percent AEP Sacramento-centered storm.
- Lake Oroville's objective flow downstream from Oroville Dam is half of the channel capacity at the confluence of the Feather and Yuba rivers.
- New Bullards Bar Reservoir exceeds its objective release during 1 and 0.5 percent AEP Sacramento-centered storms.
- Less than half of Yuba River flow is regulated by New Bullards Bar Reservoir.
- The Yuba River contributes to half or more of the peak flow at the Feather-Yuba river junction.

4.2 San Joaquin River Basin

No Project flow conditions in the San Joaquin River Basin were simulated using HEC-5. No changes to Comprehensive Study HEC-5 assumptions

for reservoir operational criteria for the San Joaquin River Basin were made for this analysis. Table 4-2 shows simulated peak flows in the San Joaquin River Basin at various locations on the mainstem under the six flood events resulting from a Vernalis-centered storm. Shaded cells in the table indicate peak flows in excess of the channel capacity

Table 4-2. Simulated San Joaquin River Basin Objective Flow Exceedence for No Project Condition for Vernalis-Centered Storm

Index Point Location	Channel Capacity (cfs)	Peak Flow of Flood Event (cfs)					
		10 percent AEP	4 percent AEP	2 percent AEP	1 percent AEP	0.5 percent AEP	0.2 percent AEP
Chowchilla Bypass near Fresno River	10,000	7,447	9,485	12,577	24,024	37,405	55,805
El Nido	16,500	12,070	16,566	22,262	36,672	62,441	98,012
Newman	45,000	21,713	27,575	32,494	62,665	98,090	154,357
At Maze Road Bridge	46,000	30,407	37,097	55,020	92,051	135,191	214,299
Near Vernalis	52,000	35,564	44,856	62,342	98,864	150,109	250,309
Stockton[1]	52,000*	36,883	46,582	63,128	98,194	150,627	250,132

Model: HEC-5
Note:
[1] HEC-5 models Stockton as downstream from the confluence of the San Joaquin River and Littlejohns Creek, and no flow is diverted to other tributaries. Assumed channel capacity would remain the same as at Vernalis.
Shaded cells indicate peak flows in excess of the channel capacity.

Key:
AEP = annual exceedence probability
cfs = cubic feet per second
Vernalis-Centered = Storm centered at San Joaquin River at latitude of Vernalis

According to the HEC-5 simulation, at most locations, the San Joaquin River system capacity is only sufficient for storms at or more frequent than a 4 percent AEP. For a 2 percent AEP Vernalis-centered storm, only three of the five major multipurpose flood reservoirs are able to operate without exceeding objective releases. Millerton Lake and New Don Pedro Reservoir have a simulated 2 TAF and 86 TAF, respectively, of inflow in excess of available flood storage (Figure 4-4).

For a 1 percent AEP Vernalis-centered storm, Millerton Lake and New Don Pedro Reservoir are unable to stay within their objective releases, and have 61 TAF and 224 TAF more inflow, respectively, than they can manage (Figure 4-4). Lake McClure also has a simulated inflow of 99 TAF in excess of available flood storage.

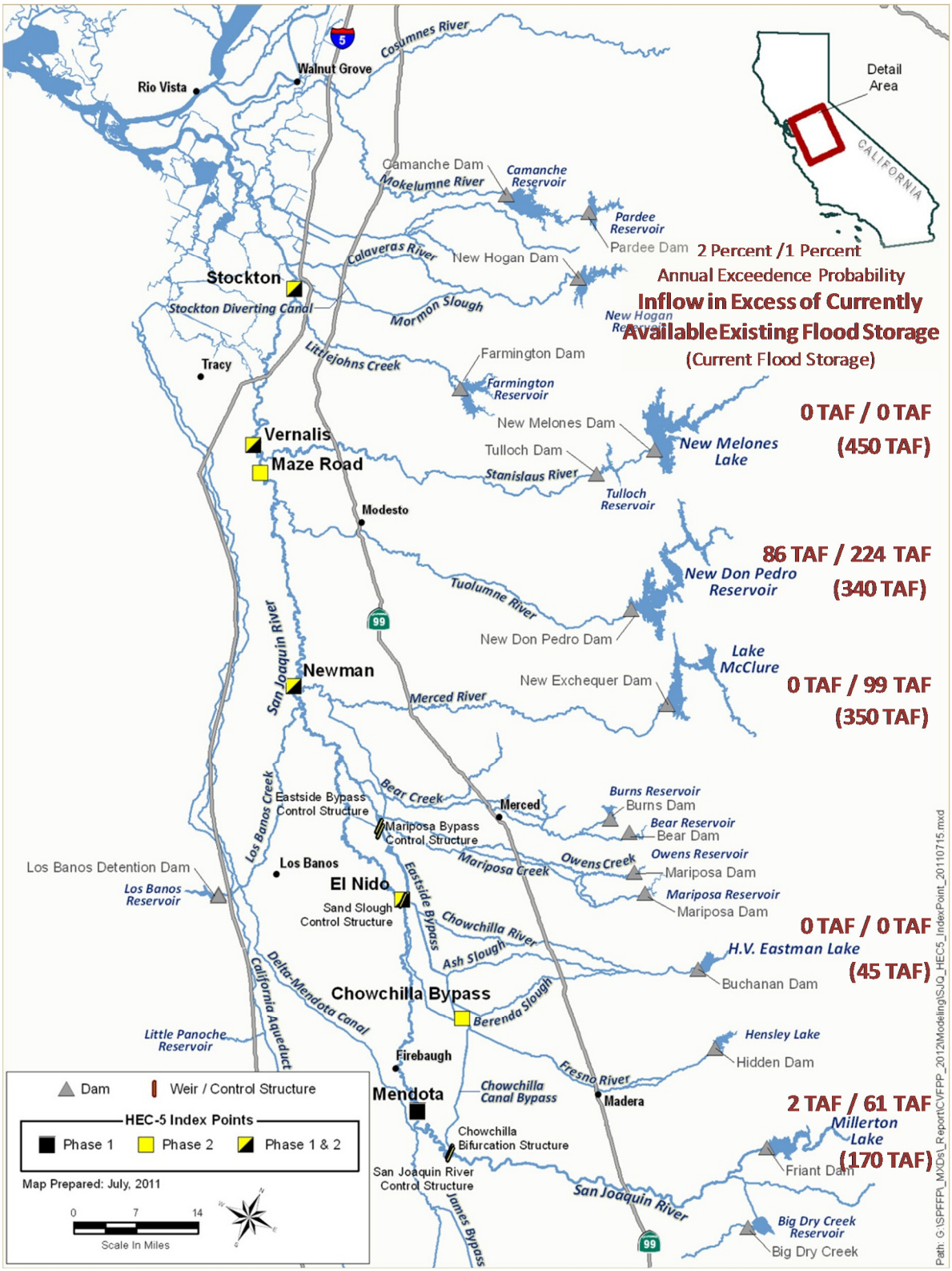
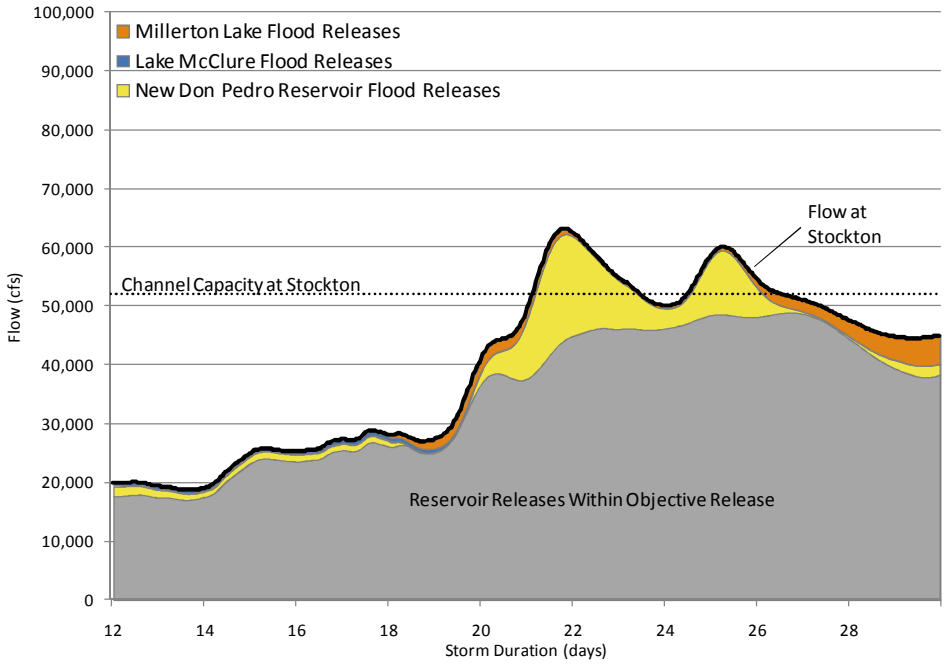


Figure 4-4. Volume of Inflow in Excess of Currently Available Flood Storage for 2 and 1 Percent AEP Vernalis-Centered Storms for No Project Condition

H.V. Eastman and New Melones reservoirs are able to operate within their objective releases for both the 2 and 1 percent AEP storms.

When a reservoir makes releases in excess of objective release targets, it almost always exceeds the channel capacity just downstream from the reservoir and also has a higher potential to contribute to exceeding channel capacity downstream in the river system. Unlike the Sacramento River Basin, which has a complex system of weirs and bypasses, the majority of reservoir releases in the San Joaquin River Basin flow directly into the mainstem San Joaquin River. As a result, it is possible to evaluate the impact of reservoir releases above objective flow targets on the system at a reconnaissance level.

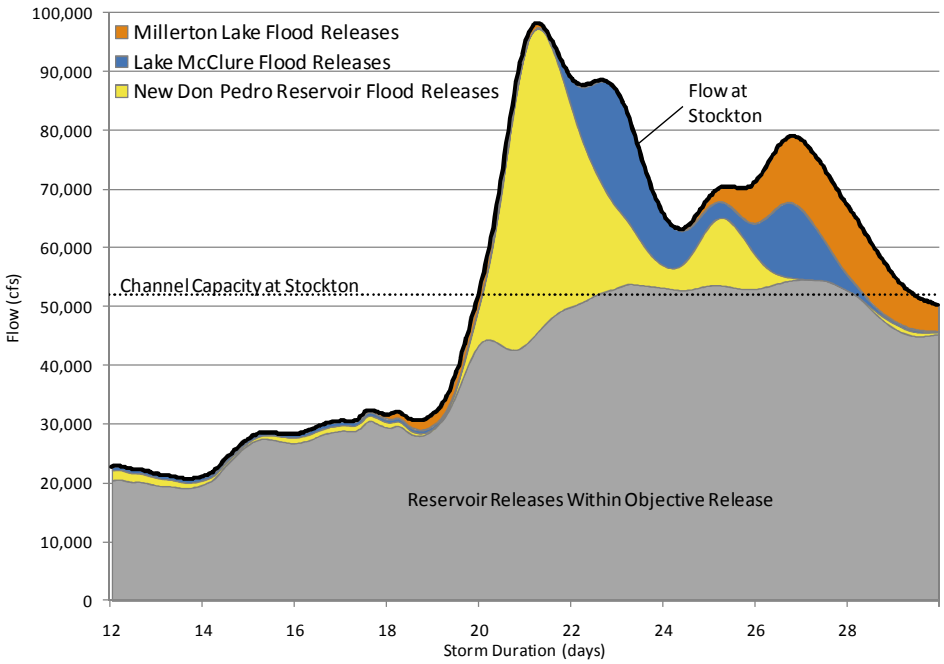
For a 2 percent AEP Vernalis-centered storm, channel capacity in the San Joaquin River at Stockton is exceeded under the No Project condition (Figure 4-5). If all of the multipurpose reservoirs operated within their objective releases, channel capacity at Stockton would not be exceeded, as shown by the grey shaded area. H.V. Eastman and New Melones reservoirs are not shown in the figure because they operate within the objective release (i.e., no flood releases). Releases from New Don Pedro Reservoir above its Tuolumne River flow objective were the main contributor to channel capacity in Stockton being exceeded.



Note:
Reservoir flood releases mean reservoir releases are above their objective releases.

Figure 4-5. Simulated Reservoir Contributions to Flow at Stockton for 2 Percent AEP Vernalis-Centered Storm for No Project Condition

As shown in Figure 4-6, for a 1 percent AEP storm, the highest peak flow in the San Joaquin River at Stockton is predominantly influenced by New Don Pedro Reservoir. Lake McClure and Millerton Lake also release flows above their objective releases and contribute to high flows at Stockton, but their contributions occur later in the storm event and do not affect the highest peak flow at Stockton. If the reservoirs were operated to not exceed their objective releases, flows at Stockton would be close to staying within the channel capacity (as shown by the top of the grey shaded area being close to the dotted channel capacity line).



Note:
Reservoir flood releases mean reservoir releases are above their objective releases.

Figure 4-6. Simulated Reservoir Contributions to Flow at Stockton for 1 Percent AEP Vernalis-Centered Storm for No Project Condition

New Don Pedro Reservoir contributes the largest volume of floodflow into the system. However, even if all reservoirs operate within their objective releases, flows at Stockton would remain well above channel capacity for storms of greater magnitude.

The following observations were made regarding the reservoirs' current operational criteria and were used to guide the magnitude and location of strategic reservoir operational criteria changes based on review of current reservoir operational criteria during 2 and 1 percent AEP storms:

- New Don Pedro Reservoir has the largest volume of inflow that cannot be managed for the hydrology used in this analysis.

- New Don Pedro Reservoir is the sole contributor to peak flow at Stockton for a 1 percent AEP Vernalis-centered storm.
- H.V. Eastman and New Melones reservoirs do not exceed their respective objective release targets for either Vernalis-centered storm frequency.
- Lake McClure is appropriately sized to manage a 2 percent AEP Vernalis-centered storm.
- The effect of Millerton Lake exceeding its objective release for San Joaquin River flows in Stockton is not observed until late in the simulated storm because of the long travel distance.

5.0 Basin-Wide Sensitivity to Changes in Reservoir Operational Criteria

This section summarizes the sensitivity of reservoir operational criteria changes on individual and basin-wide bases. First, multiple changes were made to reservoirs' operational criteria to determine how the reservoirs and the system would react to operational criteria changes. Next, the operational criteria changes were incrementally refined to determine which modifications were most effective in yielding flood risk management benefits. Lastly, as described in Section 6, the operational criteria changes that yielded high flood benefits, as simulated in HEC-5, were used to identify the two scenarios considered for the Enhance Flood System Capacity Approach.

Changes in reservoir operational criteria were simulated in HEC-5 for multipurpose reservoirs within the Central Valley. Operational criteria changes explored in this analysis included the following:

- Changes to the flood management rule curves (i.e., increasing the amount of space dedicated to flood storage)
- Changes to the objective flow to which a reservoir is operated
- Changes to the reservoir release diagram
- Addition of coordinated reservoir operating locations

5.1 Sacramento River Basin Operational Criteria Changes

The basin-wide sensitivity analysis was completed in two phases (as described in Section 3). Phase 1 explored how the system would react to simultaneous operational criteria changes at multiple reservoirs and identified which reservoirs have the greatest potential to benefit the system. Phase 2 made incremental operational criteria changes to the identified reservoirs.

The 16 scenarios from Phase 1 and Phase 2 in the Sacramento River Basin are summarized in Table 5-1. During the Phase 1 analysis, six scenarios with modified operational criteria at Lake Oroville, New Bullards Bar Reservoir, Folsom Lake, and Shasta Lake were run.

5.1.1 Phase 1

Main findings and recommendations from Phase 1 of the 2012 CVFPP Reservoir Analysis in the Sacramento River Basin are summarized as follows:

- The Feather-Yuba River Basin is potentially sensitive to operational criteria changes. Modifications to Lake Oroville and New Bullards Bar Reservoir resulted in peak flow reduction in the Feather-Yuba River Basin. Although attenuated, similar effects were observed on the Sacramento River and in the Yolo Bypass.

- There was no noticeable effect from operational criteria changes to Shasta Lake and Folsom Lake.

- Phase 2 of the 2012 CVFPP Reservoir Analysis should focus on Lake Oroville and New Bullards Bar Reservoir.

5.1.2 Phase 2

For Phase 2, 10 scenarios with modified Lake Oroville and New Bullards Bar Reservoir operational criteria were run. One scenario that modified Lake Oroville operational criteria was identified during Phase 2 for the Sacramento River Basin and is discussed in further detail in Section 7.

Shasta Lake operational criteria changes were not explored in Phase 2 because of the large magnitude of unregulated flows entering from tributaries downstream from Shasta Lake that overwhelms changes made to Shasta Lake operational criteria. For example, the simulated 1 percent AEP storm peak flow for the No Project condition from Shasta Lake was 74,000 cfs, while its downstream tributary, Cottonwood Creek, had a larger peak flow of 97,400 cfs for the same storm.

Folsom Lake operational criteria changes were not explored in Phase 2 because Folsom Lake operational criteria have recently been changed through the Folsom Dam JFP. These modifications were included in the No Project condition model.

Table 5-1. Sacramento River Basin Reservoir Operational Criteria Changes and Peak Flow Reduction for 1 Percent AEP Sacramento-Centered Storm

	Scenario	Flood Storage Added by Reservoir[1] (1,000 acre-feet)					Shasta Lake Objective Release Changes (cfs)	Lake Oroville Release Schedule Changes[2]	Description	Peak Flow Reduction (percent)[3]	
		Lake Oroville	New Bullards Bar Reservoir	Folsom Lake	Shasta Lake	Total				At Nicolaus	At Freeport
Phase 1	SAC-1	100	50			**150**			Evaluate the sensitivity of each reservoir to flows in the Sacramento River Basin.	2.1	0.1
	SAC-2	150	50			**200**				1.0	0.1
	SAC-3	250	100			**350**				0.2	0.2
	SAC-4	250	100	107		**457**				0.2	0.2
	SAC-5	250	100		500	**850**				0.2	0.2
	SAC-6	250	100		500	**850**	79,000 to 75,000			0.2	0.2
Phase 2	SAC-7	100	100			**200**			Combine Lake Oroville and New Bullards Bar Reservoir.	2.1	0.1
	SAC-8	250	150			**400**				0.2	0.2
	SAC-9	500	200			**700**				2.2	0.5
	SAC-10		100			**100**			Isolate New Bullards Bar Reservoir effects.	0.0	0.1
	SAC-11		150			**150**		**C**		2.5	-0.5
	SAC-12	100				**100**		**A**	Isolate Lake Oroville effects.	1.7	0.4
	SAC-13	200				**200**				-0.1	0.1
	SAC-14	200				**200**		**B**		-0.4	0.6
	Sacramento Scenario	200				**200**		**C**		3.8	-1.0
	SAC-15	200	50			**250**		**C**	Combine Lake Oroville and New Bullards Bar Reservoir operational criteria changes.	4.4	-0.8

Note:
[1] Blank cells represent no changes to operational criteria.
[2] See Lake Oroville Release Schedule Modifications Table (Table 5-2) for more details.
[3] Negative peak flow reductions correspond to an increase in peak flow.

Key:
AEP = annual exceedence probability
cfs = cubic feet per second
SAC = Sacramento

5.1.3 Oroville Dam and Lake Oroville

The Feather River is sensitive to changes in Lake Oroville's operational criteria. Currently, Lake Oroville can operate near its objective release for up to a 0.5 percent AEP Sacramento-centered storm. Despite both Lake Oroville and New Bullards Bar Reservoir operating within their objective releases, downstream channel capacities at some locations are exceeded during a 1 percent AEP Sacramento-centered storm.

During the basin-wide sensitivity analysis, Lake Oroville's flood storage allocation was increased, but this did not produce a noticeable reduction in reservoir releases. The release schedule of a reservoir is not only a function of storage in the reservoir, but also inflow into the reservoir. As a result, the release schedule at Lake Oroville was modified such that the maximum objective release of 150,000 cfs would not occur until there was a higher reservoir inflow than under current conditions. This change was made in conjunction with an increase in flood storage to allow the reservoir to manage more water while still permitting releases to be governed by inflow rather than operational criteria for flood pool. Three release schedule modifications were explored (Table 5-2).

Table 5-2. Lake Oroville Release Schedule Modifications

No Project		A		B		C	
Inflow (cfs)	Release Capacity (cfs)	Inflow (cfs)	Release Capacity (cfs)	Inflow (cfs)	Release Capacity (cfs)	Inflow (cfs)	Release Capacity (cfs)
1	15,000	1	15,000	1	15,000	1	15,000
30,000	15,000	30,000	15,000	30,000	15,000	30,000	15,000
30,005	60,000	30,005	70,000	30,005	60,000	30,005	60,000
120,000	60,000	120,000	70,000	120,000	60,000	120,000	60,000
120,005	100,000	120,005	100,000	120,005	80,000	120,005	80,000
175,000	100,000	175,000	100,000	200,000	80,000	300,000	80,000
175,005	150,000	175,005	150,000	200,005	150,000	300,005	150,000
900,000	150,000	900,000	150,000	900,000	150,000	900,000	150,000

Key:
cfs = cubic feet per second

5.1.4 New Bullards Bar Dam and Reservoir

For the No Project condition, New Bullards Bar Reservoir is generally able to operate within its objective release criteria for 1 percent AEP or more frequent storms. Operational criteria changes to New Bullards Bar Reservoir could lower its peak releases, but its effect on the system is minimal. As shown in Figure 5-1, less than half of the Yuba River flow at

Marysville is regulated by New Bullards Bar Reservoir on the North Yuba River; the remaining flow comes from the unregulated Middle and South Yuba rivers. Because New Bullards Bar Reservoir regulates less than half of the Yuba River flows, operational criteria changes did not produce large downstream flood risk management benefits.

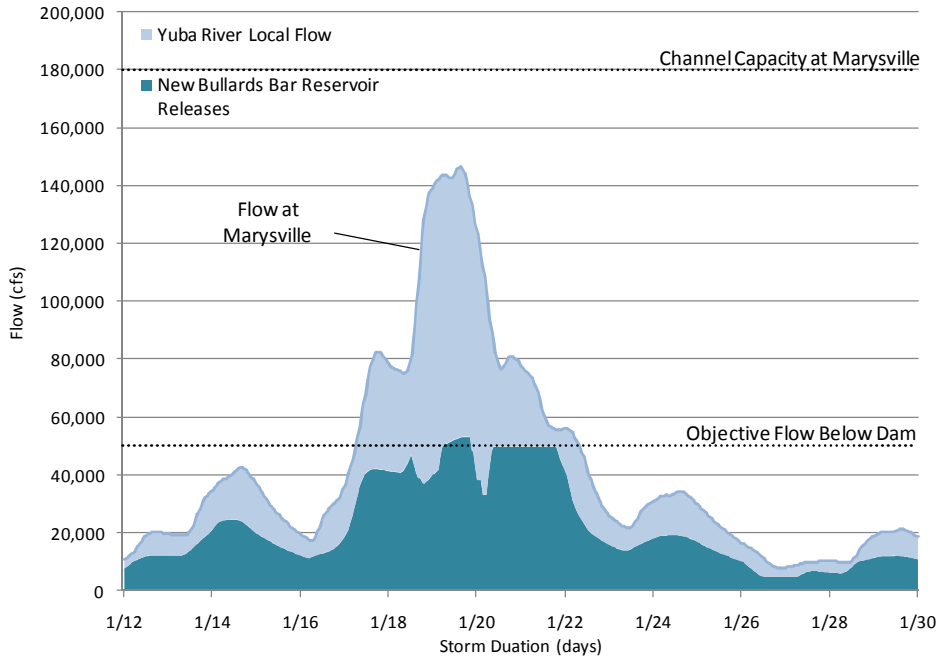


Figure 5-1. Yuba River Flow for 1 Percent AEP Sacramento-Centered Storm – No Project Condition

For more infrequent storms (0.5 and 0.2 percent AEPs), when New Bullards Bar Reservoir would be forced to make releases in excess of objective release targets, additional flood storage does improve downstream channel flow conditions. Adding flood storage would allow the reservoir to release flows closer to its objective release targets. For example, adding 100 TAF of storage decreases flow at Marysville from approximately 195,800 cfs to 186,500 cfs for a 0.5 percent AEP Sacramento-centered storm.

5.2 San Joaquin River Basin Operational Criteria Changes

The 33 scenarios from Phase 1 and Phase 2 in the San Joaquin River Basin are summarized in Table 5-3. During the Phase 1 analysis, 17 scenarios were run that modified operational criteria at Millerton Lake, H.V. Eastman Lake, Lake McClure, New Melones Reservoir, and New Don Pedro Reservoir.

Table 5-3. San Joaquin River Basin Reservoir Operational Criteria Changes and Peak Flow Reduction for 1 Percent AEP Vernalis-Centered Storm

	Scenario	Flood Storage Added by Reservoir[1] (1,000 acre-feet)						Objective Release Changes by Reservoir (cfs)	Description	Peak Flow Reduction at Stockton (percent)[2]
		Millerton Lake	H.V. Eastman Lake	Lake McClure	New Don Pedro Reservoir	New Melones Reservoir	Total			
Phase 1	A-1		45			100	**145**		No Millerton Lake changes.	0
	B-1	25					**25**		Isolate Millerton Lake effects.	0
	C-1	50					**50**			0
	C-2	50					**50**	New Don Pedro Reservoir: 9,000 to 11,000	Combine upper San Joaquin River reservoirs.	9
	C-3	50		25		50	**125**			6
	C-4	50		25	25		**100**			11
	C-5	50		25	25		**100**	New Melones Reservoir: 8,000 to 6,000		13
	C-6	50		50			**100**			6
	C-7	50		50			**100**	Lake McClure: 6,000 to 5,000		6
	D-1	85					**85**			1
	D-2	85	45				**130**		Isolate Millerton Lake effects.	1
	D-3	85	45				**130**	New Melones Reservoir: 8,000 to 6,000	Combine upper San Joaquin River reservoirs.	8
	D-4	85	45		100		**230**			10
	D-5	85	45			100	**230**			1
	D-6	85	45	25		100	**255**			6
	D-7	85	45	100			**230**			1
	D-8	85	45	100		150	**380**			6

Table 5-3. San Joaquin River Basin Reservoir Operational Criteria Changes and Peak Flow Reduction for 1 Percent AEP Vernalis-Centered Storm (contd.)

	Scenario	Flood Storage Added by Reservoir[1] (1,000 acre-feet)						Objective Release Changes by Reservoir (cfs)	Description	Peak Flow Reduction at Stockton (percent)[2]
		Millerton Lake	H.V. Eastman Lake	Lake McClure	New Don Pedro Reservoir	New Melones Reservoir	Total			
Phase 2	SJQ-1	300					**300**		Assume construction of Temperance Flat Dam.	3
	SJQ-2			25			**25**		Isolate Lake McClure effects.	0
	SJQ-3			50			**50**			0
	SJQ-4			100			**100**			0
	SJQ-5			150			**150**			0
	SJQ-6				25		**25**		Isolate New Don Pedro Reservoir effects.	5
	SJQ-7				100		**100**			10
	SJQ 7a				230		**230**			20
	SJQ-8				275		**275**			20
	SJQ-9				300		**300**			20
	SJQ-10						**0**	New Don Pedro Reservoir: 9,000 to 12,000		7
	SJQ-11				160		**160**	New Don Pedro Reservoir: 9,000 to 12,000		16
	San Joaquin Scenario	60		100	230		**390**		Combine effect of reservoirs based on volume of unmanageable inflow.	34
	SJQ-12	100		150	300		**550**			44
	SJQ-13			100	230		**330**			25
	SJQ-14[3]			100	230		**330**		Include coordination operations.	25

Note:
[1] Blank cells represent no changes to operational criteria.
[2] Rounded to the nearest percent.
[3] Added coordinated operation point at Maze Road for Lake McClure and New Don Pedro Reservoir.

Key:
AEP = annual exceedence probability
cfs = cubic feet per second

5.2.1 Phase 1

The main findings and recommendations from Phase 1 of the 2012 CVFPP Reservoir Analysis in the San Joaquin River Basin are summarized as follows:

- Even after operational criteria changes, simulated peak flows at some locations exceeded assumed channel capacity in all storm events, except events of 4 percent AEP and smaller. Peak flows at these locations were results of hydrologic routing, which does not reflect levee breaches as in hydraulic models. These results are indicative but not predictive of how flow could change.

- Further hydraulic modeling is recommended as necessary to better understand changes to mainstem flow through reservoir operational criteria changes.

5.2.2 Phase 2

For Phase 2, 16 scenarios with modified Millerton Lake, Lake McClure, and New Don Pedro Reservoir operational criteria were run.

H.V. Eastman Lake operational criteria changes were not made in Phase 2 because increasing the volume of H.V. Eastman Lake's flood reservation space did not provide any additional benefits in peak flow reduction in the San Joaquin River at Stockton (as shown in the Phase 1 analysis).

No additional simulations were run that included New Melones Reservoir in Phase 2 because the reservoir has a large storage volume compared to the volume of inflow into the reservoir. The sensitivity of increasing the flood storage allocation among the three upper San Joaquin River reservoirs (including New Melones Reservoir) is briefly discussed later in this section. As a result, one scenario that modified a combination of Millerton Lake, Lake McClure, and New Don Pedro Reservoir operational criteria changes was identified during Phase 2 for the San Joaquin River Basin and is discussed in further detail in Section 6.

5.2.3 Friant Dam and Millerton Lake

As described earlier, Millerton Lake is almost capable of operating within its objective release for a 2 percent AEP Vernalis-centered storm but is unable to manage all of the 1 percent AEP storm inflow with its current 170 TAF allocation of flood storage. Figures 5-2 and 5-3 show the effects of adding three increments of flood storage to Millerton Lake.

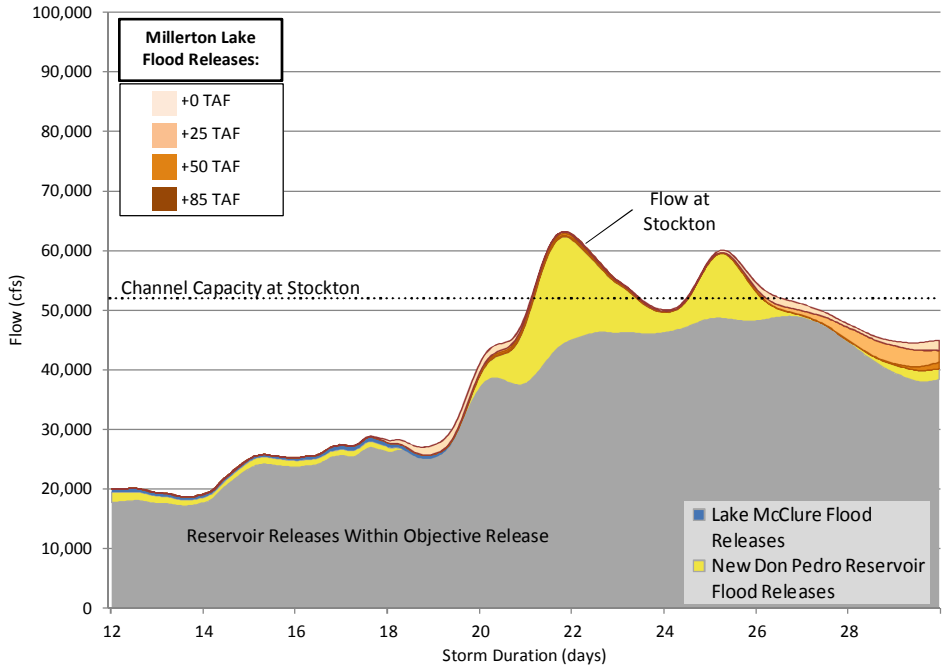


Figure 5-2. Comparison of Effects at Stockton from Additional Flood Storage Increments at Millerton Lake for 2 Percent AEP Vernalis-Centered Storms

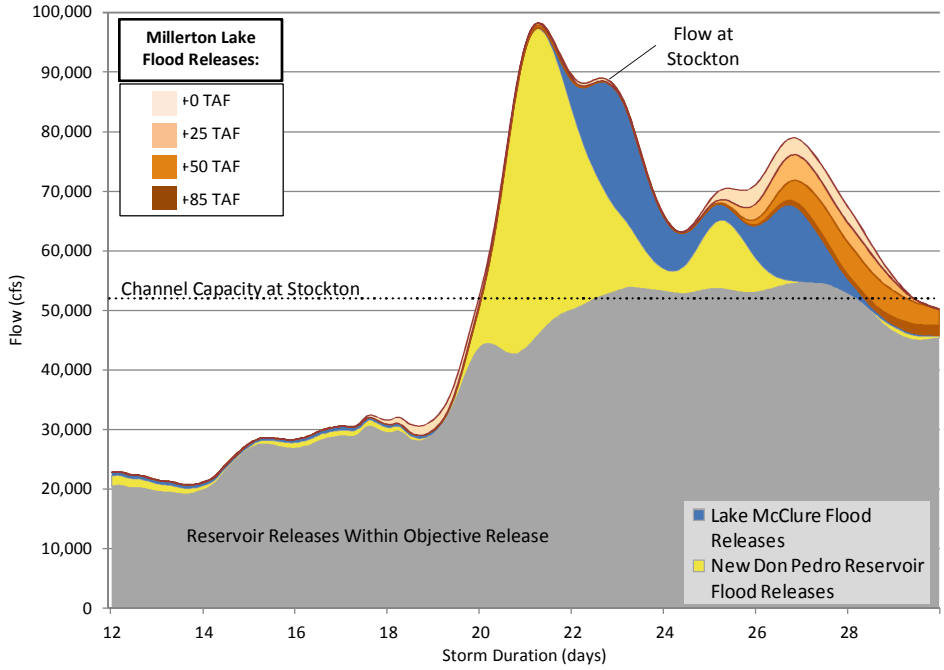


Figure 5-3. Comparison of Effects at Stockton from Additional Flood Storage Increments at Millerton Lake for 1 Percent AEP Vernalis-Centered Storms

The largest simulated effects occurred when flood storage was increased by 50 percent (from 170 TAF to 255 TAF). For the 1 percent AEP storm, simulated peak flow decreased by a maximum of 5,703 cfs, but flow remained above channel capacity for nearly the same duration for the No Project condition at Stockton; peak flow decreased by only 3 hours for a 1 percent AEP storm.

Table 5-4 shows that of the three scenarios, the largest benefit relative to the increase in flood storage allocation was when 85 TAF of flood storage was added to Millerton Lake flood storage allocation.

5.2.1 San Joaquin River Reservoirs

The sensitivity of allocating the same magnitude of additional flood storage at different reservoirs was further explored using HEC-5 runs from Phase 1. Increasing the flood storage allocation by 100 TAF at Lake McClure and New Don Pedro and New Melones reservoirs had different effects on the system. Under the No Project condition, both New Melones Reservoir and Lake McClure can manage a 2 percent AEP storm, and New Melones Reservoir can manage a 1 percent AEP storm. Hence, it was expected and confirmed that adding more flood storage allocation would have limited downstream effects. Reservoir operational criteria changes have less effect on the flood management systems if a reservoir is already capable of managing flood inflows (i.e., the objective release is not exceeded).

Table 5-5 shows that because New Don Pedro Reservoir has the largest volume of floodflow that cannot be managed, this reservoir showed the greatest downstream benefit from an increased flood storage allocation. Changes to the objective releases of the reservoirs, in combination with increased flood storage allocations, were explored in Phase 1, but did not noticeably affect peak downstream flows.

Table 5-4. Effects of Additional Flood Storage Allocation at Millerton Lake for 2 and 1 Percent AEP Vernalis-Centered Storm

Total Added Storage (TAF)	Peak Flow at Stockton				Stockton Channel Capacity Exceeded				At Stockton	
	Rate (cfs)		Percent Reduction		Duration (hours)		Percent Reduction		Peak Flow Reduction Index[1]	Unit Performance Index[2] (percent)
	2 percent	1 percent	2 percent	1 percent	2 percent	1 percent	2 percent	1 percent		
0	63,128	98,194	N/A	N/A	106	227	N/A	N/A	N/A	N/A
25	63,232	98,285	0	0	97	225	8	1	-0.1[3]	-0.4
50	62,930	97,902	0	0	95	222	10	2	0.1	0.1
85	62,532	97,548	1	1	92	202	13	11	0.3	0.4

Model: HEC-5

Notes:

[1] Peak Flow Reduction Index = Σ [(Percent Reduction_AEP, i) x (AEP, i)] / [Σ (AEP, i)] x 100

[2] Unit Performance Index = Peak Flow Reduction Index / Total TAF

[3] Indices are negative because for some AEPs, peak flow increased at Stockton because the shift in flows at Millerton Lake, combined with the peak flows from other tributaries, resulted in greater downstream peak flows.

Key:

AEP = annual exceedence probability

cfs = cubic feet per second

N/A = not applicable

TAF = thousand acre-feet

5.2.1 New Don Pedro Dam and Reservoir

Because operational criteria changes to New Don Pedro Reservoir yielded the greatest downstream benefit, three types of operational criteria changes were preliminarily explored: required flood storage allocation increases, objective release increases, and coordinated operations with Lake McClure.

Increase in Flood Storage Allocation

Increasing flood storage allocation at New Don Pedro Reservoir resulted in flood management benefits. Varying allocations of flood storage were added to New Don Pedro Reservoir to observe their effects on the system.

As shown in Figure 4-4, New Don Pedro Reservoir has a simulated 224 TAF of inflow during a 1 percent AEP Vernalis-centered storm in excess of available storage. To determine whether increasing the flood storage allocation by an equivalent amount would yield flood risk management benefits, 230 TAF of flood storage allocation was added. Figure 5-4 shows that this has a substantial impact on the magnitude of flows and the duration of time that channel capacity is exceeded. To confirm that the volume of flood inflow exceeding available storage is directly related to changes in downstream peak flow, a suite of additional flood storage allocation scenarios were simulated. Reduction in flow and the duration of time that channel capacity is exceeded occurs as more flood storage is allocated to New Don Pedro Reservoir, but this relationship is not linear. The largest benefit is realized when 230 TAF of flood storage is added; flow remains within channel capacity for a 2 percent AEP Vernalis-centered storm and peak flows decrease by nearly 20,000 cfs for a 1 percent AEP storm. The incremental benefit tapers off as additional flood storage is allocated.

The peak flow reduction index and unit performance index are lower for these scenarios compared to operational criteria changes for other reservoirs, such as at Millerton Lake (Table 5-6). Because these indices are weighted by storm AEP, and the largest benefit from peak flow reduction occurs for less frequent storms, the benefit derived from New Don Pedro Reservoir operational criteria changes may be considered understated.

Objective Release Changes

To minimize the volume of additional flood storage allocation while still reducing downstream flow, an increase in the objective release from New Don Pedro Reservoir was also explored. Effects of changes to the objective release on the system varied, depending on the frequency of the storm.

Table 5-5. Effects of Additional Flood Storage Allocation for 2 and 1 Percent AEP Vernalis-Centered Storm

Add 100 TAF of Storage to	Total Added Storage (TAF)[1]	Peak Flow at Stockton				Stockton Channel Capacity Exceeded				At Stockton	
		Rate (cfs)		Percent Reduction		Duration (hours)		Percent Reduction		Peak Flow Reduction Index[2]	Unit Performance Index[3] (percent)
		2 percent	1 percent	2 percent	1 percent	2 percent	1 percent	2 percent	1 percent		
N/A	130	62,617	97,669	0	0	87	201	0	0	0	0
McClure	230	62,617	97,583	1	1	87	201	18	11	0.5	0.2
New Don Pedro	230	55,740	87,892	12	10	27	184	75	19	1.9	0.8
New Melones	230	62,617	97,669	1	1	87	201	18	11	0.7	0.3

Model: HEC-5

Notes:

[1] Includes increasing flood storage allocation by 85 TAF and 45 TAF to Millerton Lake and H.V. Eastman Lake, respectively.

[2] Peak Flow Reduction Index = Σ [(Percent Reduction_AEP, i) x (AEP, i)] / [Σ (AEP, i)] x 100

[3] Unit Performance Index = Peak Flow Reduction Index / Total TAF

Key:

AEP = annual exceedence probability

cfs = cubic feet per second

N/A = not applicable

TAF = thousand acre-feet

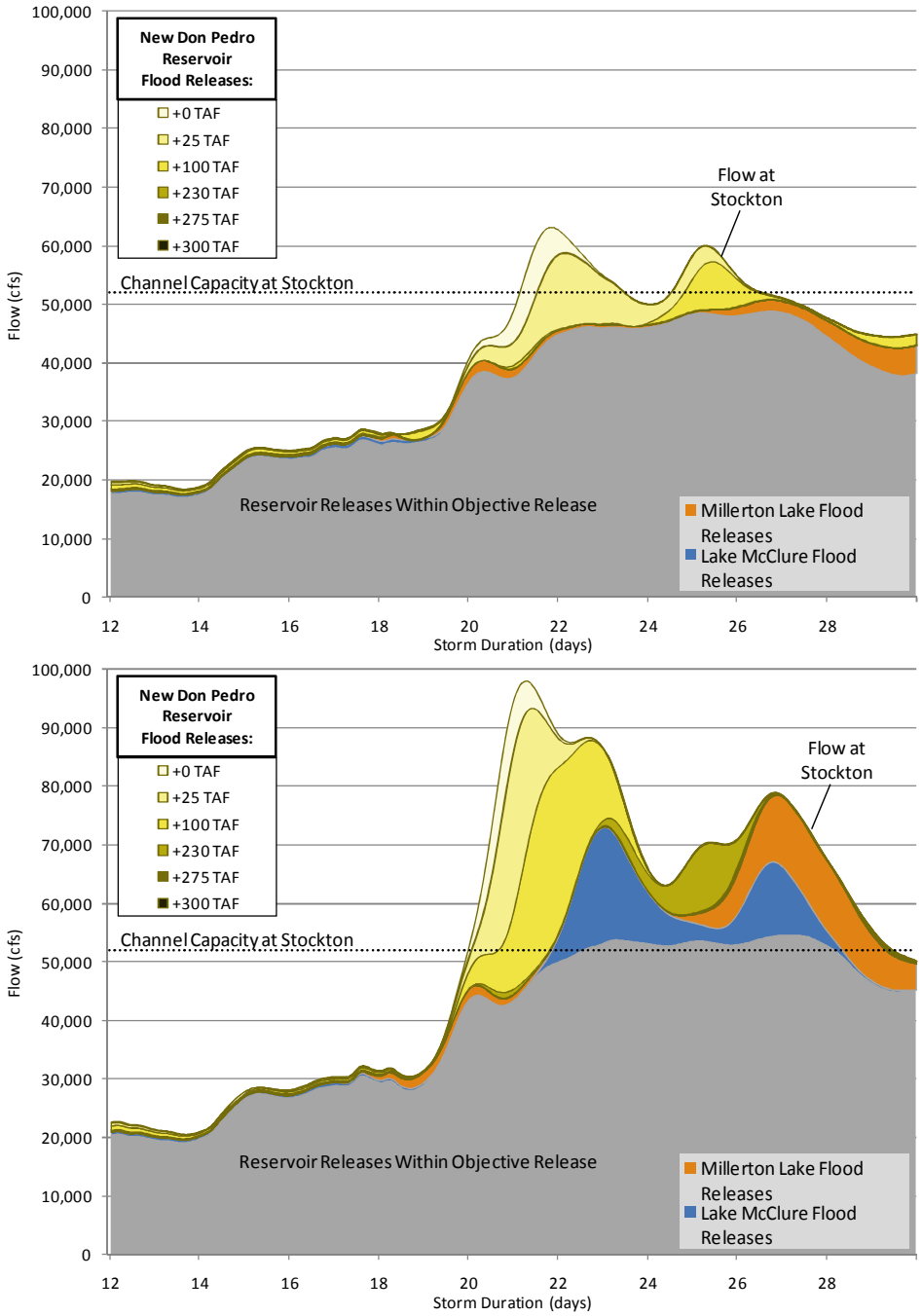


Figure 5-4. Comparison of Effects at Stockton from Additional Flood Storage Allocation Increments at New Don Pedro Reservoir for 2 and 1 Percent AEP Vernalis-Centered Storm

Table 5-6. Effects of Operational Criteria Changes at New Don Pedro Reservoir for 2 and 1 Percent AEP Vernalis-Centered Storm

Total Added Storage (TAF)	Peak Flow at Stockton				Stockton Channel Capacity Exceeded				At Stockton	
	Rate (cfs)		Percent Reduction		Duration (hours)		Percent Reduction		Peak Flow Reduction Index[1]	Unit Performance Index[2] (percent)
	2 percent	1 percent	2 percent	1 percent	2 percent	1 percent	2 percent	1 percent		
0	63,128	98,194	N/A	N/A	106	227	N/A	N/A	N/A	N/A
25	60,066	93,525	5	5	97	226	8	0	0.9	3.6
100	57,401	87,943	9	10	42	212	60	7	1.9	1.9
230	50,878	78,972	19	20	0	184	100	19	3.7	1.6
275	50,878	78,770	19	20	0	184	100	19	3.8	1.4
300	50,878	78,589	19	20	0	180	100	21	3.9	1.3

Model: HEC-5

Notes:

[1] Peak Flow Reduction Index = Σ [(Percent Reduction_AEP, i) x (AEP, i)] / [Σ (AEP, i)] x 100

[2] Unit Performance Index = Peak Flow Reduction Index / Total TAF

Key:

AEP = annual exceedence probability
cfs = cubic feet per second
N/A = not applicable
TAF = thousand acre-feet

Increasing objective releases allows a reservoir to release higher volumes of water earlier in a storm, increasing the available reservoir storage in anticipation of high inflows later on. This change would ideally evacuate enough storage that the reservoir would not have to exceed its objective release targets. It is important to note that objective release targets are often based on channel capacity; increasing the objective release would likely require improving the channels to increase channel capacity.

New Don Pedro Reservoir currently operates within its objective release, and channel capacity is not exceeded at Stockton for more frequent storms (10 and 4 percent AEPs). As a result, increasing the objective release had negative effects on downstream channel flow. Increasing the objective release by 3,000 cfs resulted in the average release from New Don Pedro Reservoir increasing by 3,000 cfs, and an associated higher downstream channel flow.

For larger storm events (2 percent AEP and less frequent AEPs), New Don Pedro Reservoir exceeds its objective release under current operating rules. Increasing the objective release slightly lowered the peak flow, but increased the duration of time that the downstream channel capacity was exceeded.

Increase in Flood Storage Allocation and Objective Release

The basin-wide sensitivity analysis also considered simultaneously increasing both flood storage allocation and objective release at New Don Pedro Reservoir to lower the peak release and decrease the volume of unmanageable flood inflow into the reservoir.

In summary, increasing the flood storage allocation by 160 TAF had two effects:

1. **Lowered peak flow** – More space was available to capture flood inflow and, hence, the reservoir could make lower releases.

2. **Decreased duration of flow above downstream channel capacity** – The duration of time the New Don Pedro Reservoir releases were in excess of objective release targets was much shorter than under current operational criteria, and reservoir releases were lower. Lower peak releases, when combined with mainstem flows, decreased the duration of time that downstream flows were greater than capacity.

Increasing the objective release by 3,000 cfs had two effects:

1. **Lowered peak flow** – More space could be maintained to capture high flood inflow and, hence, the reservoir could make lower releases throughout a storm event.

2. **Increased duration of time above downstream channel capacity** – Higher objective reservoir releases, when combined with mainstem flows, increased the duration of time that downstream flows were higher.

Increasing the objective release by 3,000 cfs and flood storage allocation by 160 TAF had two effects:

1. **Lowered peak flow** – More space was available to capture high flood inflow; hence, the reservoir could make lower releases. Increasing the flood storage allocation kept the downstream flow entirely within the channel capacity.

2. **Decreased duration of time above downstream channel capacity** – The duration of time that New Don Pedro Reservoir made releases in excess of objective release targets was much shorter; hence, peak reservoir releases were also lower. However, higher releases resulting from an increase in the objective release, when combined with mainstem flows, would offset some of the benefit of lower peak releases.

Similar to other storm frequencies, increasing the objective release lowered the peak flow for large infrequent storms (0.5 and 0.2 percent AEPs), but increased the duration of time that channel capacity would be exceeded. Peak flow would be slightly lowered because a small amount of storage would be evacuated before the large inflow. However, because the inflow was of such a high magnitude, the benefit of additional flood storage allocation would be almost negligible.

Overall, downstream channel benefits were lower when compared to only the allocation of additional flood storage for large storm events.

New Don Pedro Reservoir and Lake McClure Coordinated Operations

Another operational criteria change explored during the basin-wide sensitivity analysis was operating both New Don Pedro Reservoir and Lake McClure for the same downstream location, the San Joaquin River at Maze Road. This change allowed the flow in the San Joaquin River to remain within the channel capacity slightly longer (by a few hours), but peak flows were higher. This was because the reservoirs held back their releases longer to keep the mainstem within the channel capacity for the earlier parts of a storm; thus, the reservoirs filled their allocated flood storage sooner and had to release more water later in the storm. As a result, this operational criteria change was not further explored in this analysis.

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6.0 Reservoir Operational Scenarios Considered

Using preliminary observations from the Phase 1 and 2 analyses, several reservoir operational scenarios were considered for inclusion in the Enhance Flood System Capacity approach. These scenarios were considered based on No Project flood management performance in the Central Valley and basin-wide sensitivity observations, and are described in more detail in the following subsections. Because of the preliminary nature of this analysis, the uncertainty associated with the effects of operational criteria changes, and the needed coordination, operational criteria changes were not moved forward into the State Systemwide Investment Approach aside from changes associated with the Folsom Dam Raise, which is already authorized.

The reservoir operational scenarios considered in the Enhance Flood System Capacity approach includes modification to the reservoir release schedule and flood storage allocation at Lake Oroville (equivalent to an additional 200,000 acre-feet of flood storage), and coordinated operation with New Bullards Bar Reservoir, to reduce flood stages on the Feather River during a 0.5 percent AEP (200-year) flood event. Also, in the San Joaquin River Basin, the State would partner with interested reservoir operators to increase the flood storage allocation at New Don Pedro, Friant, and New Exchequer dams by about 400,000 acre-feet to effectively manage the 1 percent AEP (100-year) flood event at these reservoirs.

6.1 Scenarios Considered

As stated above, the 2012 CVFPP Reservoir Analysis is a preliminary analysis and future studies will need to assess the feasibility of changes in reservoir operational criteria, with consideration of effects on other reservoir purposes, and determine the best method for implementing these changes. The goal of the analysis is the see if there are potential flood management benefits associated with making operational criteria changes; it is not to propose specific changes to any reservoir or to preclude other options in modifying operational criteria.

To demonstrate the potential of reservoir operational criteria changes in the Central Valley, the following scenarios were considered for modeling purposes only:

- Sacramento Scenario
  - Increase Lake Oroville flood storage allocation by 200 TAF
  - Modify Lake Oroville's release schedule (see Table 6-1)
- San Joaquin Scenario
  - Increase Millerton Lake flood storage allocation by 60 TAF
  - Increase Lake McClure flood storage allocation by 100 TAF
  - Increase New Don Pedro Reservoir flood storage allocation by 230 TAF

These scenarios were considered because they yielded large flood management benefits systemwide. Potential changes reduced peak downstream flow, lowered downstream flow within or near channel capacity for more AEP storms, and decreased the duration of time that flow exceeded the downstream channel capacity.

Table 6-1. Simplified Lake Oroville Release Schedule Modifications

No Project Conditions		Scenario Considered	
Reservoir Inflow (cfs)	Required Release (cfs)	Reservoir Inflow (cfs)	Required Release (cfs)
0 – 30,000	15,000	0 – 30,000	15,000
30,000 – 120,000	60,000	30,000 – 120,000	60,000
120,000 – 175,000	100,000	120,000 – 300,000	80,000
> 175,000	150,000	> 300,000	150,000

Key:
cfs = cubic feet per second

6.2 Sacramento River Basin

The Sacramento Scenario targeted modifying reservoir operational criteria in the Sacramento River Basin. Because more than half of Yuba River flow is uncontrolled, the Sacramento Scenario modified the operational criteria at Lake Oroville, on the Feather River. Because Lake Oroville is able to manage 1 and 0.5 percent AEP Sacramento-centered storms, the operational criteria changes focused on lowering reservoir releases by modifying the release schedule. Modifying the release schedule lowered the required reservoir release for a given inflow, thus storing more of the inflow in the reservoir. To offset the increase in stored water, an additional

200 TAF of flood storage was allocated to Lake Oroville's flood storage allocation. Table 6-1 details the changes to the release schedule for Lake Oroville that were considered.

As stated above, modifications to the release schedule focused on lowering average maximum reservoir releases. Under the No Project condition, Lake Oroville releases 100,000 cfs when inflow into the reservoir is between 120,000 cfs and 175,000 cfs, and increases its release to 150,000 cfs when inflow exceeds 175,000 cfs. The Sacramento Scenario proposes changing the specified release from 100,000 cfs to 80,000 cfs for the same inflow range, and delaying the maximum release of 150,000 cfs until inflow exceeds 300,000 cfs. The additional flood storage allocation would be used to store the additional volume of floodflow in the reservoir resulting from decreased releases.

This scenario resulted in not only a lower simulated peak release, but also an overall average lower release during the height of a storm. Inflow into Lake Oroville exceeds 175,000 cfs for 4 percent AEP and less frequent, larger storms (Table 6-2). Hence, under the No Project condition, Lake Oroville could release up to 150,000 cfs during a 4 percent AEP storm. With the Sacramento Scenario, the maximum outflow is limited to 80,000 cfs for up to a 1 percent AEP storm. A maximum outlet capacity of 150,000 cfs would not occur until a 0.5 percent AEP storm.

Table 6-2. Peak Inflow into Lake Oroville for Sacramento-Centered Storm

AEP	Peak Inflow[1] (cfs)
50 percent	125,000
10 percent	190,000
4 percent	237,000
2 percent	295,000
1 percent	353,000
0.5 percent	441,000

Note:
[1] Peak inflow is rounded to the nearest thousand.

Key:
AEP = annual exceedence probability
cfs = cubic feet per second

Figure 6-1 shows that the simulated peak release from Lake Oroville decreased by nearly 70,000 cfs (from 150,000 cfs to 81,182 cfs) for a 1 percent AEP Sacramento-centered storm. In addition, average reservoir releases above 60,000 cfs decreased from approximately 111,000 cfs to 78,000 cfs. This resulted in lower flow at the confluence of the Feather and Yuba rivers and the number of channel flow peaks decreasing from two to one. The Sacramento Scenario also lowered the simulated peak flow farther downstream at Nicolaus (downstream from the confluence of the Bear and Feather rivers) by 40,000 cfs. The simulated peak flow, however, remained above the 320,000 cfs channel capacity at Nicolaus, at 380,026 cfs, for a 1 percent AEP storm.

The Sacramento Scenario also lowered peak downstream flows for a 0.5 percent AEP storm (Figure 6-2). While downstream channel capacity on the Feather River was still exceeded, the simulated peak flow rate decreased by 40,000 cfs at the confluence of the Feather and Yuba rivers.

Downstream from the confluence of the Feather River with the Sacramento River at the Fremont Weir, the effect of the Sacramento Scenario on Sacramento River flows was minimal (approximately a 1 percent change in flow). Flow in the mainstem slightly increased in some locations (e.g., I Street gage). This was because the volume of water diverted from the Sacramento River to the Yolo Bypass depends on the flow upstream from the bypass. If there is less flow upstream from the bypass, then less water is diverted into the Yolo Bypass; hence, more water could remain in the mainstem.

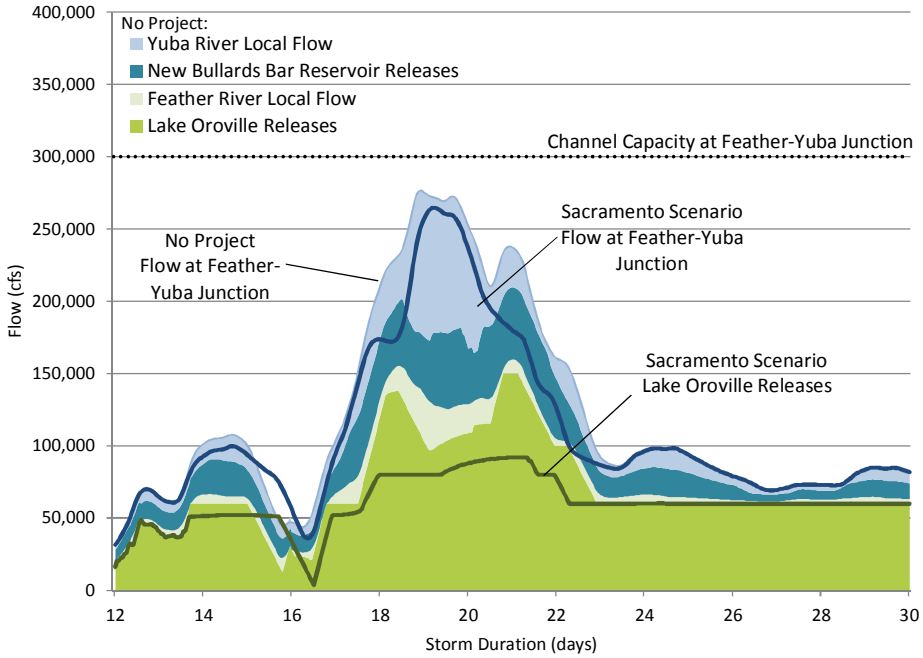


Figure 6-1. Simulated Effects of Lake Oroville Operational Criteria Changes at Feather-Yuba River Junction for 1 Percent AEP Sacramento-Centered Storm

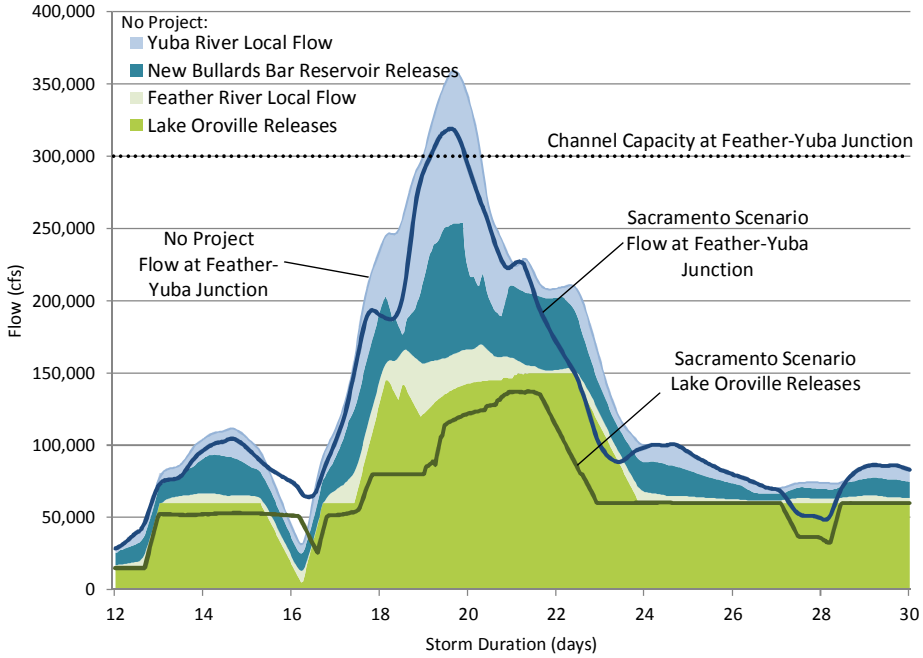


Figure 6-2. Simulated Effects of Lake Oroville Operational Criteria Changes at Feather-Yuba River Junction for 0.5 Percent AEP Sacramento-Centered Storm

Table 6-3 summarizes simulated effects on the Sacramento River Basin as a result of the Sacramento Scenario operational criteria changes to Lake Oroville.

While this scenario has flood management benefits, operational criteria changes to Lake Oroville may affect its other purposes (i.e., water supply, fisheries). Potential effects of reservoir operational criteria are discussed in Section 8.

Table 6-3. Simulated Effects of Sacramento Scenario on Peak Flow for Sacramento-Centered Storm

Index Point	Overall Effect on Peak Flow	Simulated Decrease in Peak Flow (cfs) (percent)			
		1 Percent AEP		0.5 Percent AEP	
Lake Oroville	Decrease	57,922	(39)	12,711	(8)
Feather and Yuba River Junction	Decrease	12,031	(4)	40,091	(11)
Feather River at Nicolaus	Decrease	12,551	(4)	40,077	(10)
Sacramento and Feather River Junction	Decrease	13,480	(3)	43,016	(7)
Sacramento River near I Street Gage	Increase	-638	(-1)	-1,291	(-1)

Model: HEC-5
Key:
AEP = annual exceedence probability
cfs = cubic feet per second

6.3 San Joaquin River Basin

The San Joaquin Scenario explored modifying required storage for flood management at Millerton Lake, Lake McClure, and New Don Pedro Reservoir. These three reservoirs were modified because they exceed their objective release during 1 percent AEP Vernalis-centered storms. Increasing the allocated volume of flood storage enabled the reservoirs to operate within their objective releases more frequently, decreasing channel flow downstream.

While New Don Pedro Reservoir experiences the largest amount of inflow in excess of available current flood storage, Millerton Lake and Lake McClure also contribute to above-channel-capacity flows at Stockton for 1 percent AEP and less frequent storms. To reduce both the magnitude and duration of time that channel capacity would be exceeded at Stockton, the San Joaquin Scenario increased the flood storage allocation at Millerton

Lake, Lake McClure, and New Don Pedro Reservoir by 60 TAF, 100 TAF, and 230 TAF, respectively.

The volume of additional flood storage allocation selected for the San Joaquin Scenario was based on the volume of inflow in excess of available current flood storage that could not be managed for a 1 percent AEP Vernalis-centered storm (see Figure 4-4), and the basin-wide sensitivity analysis showed that the largest benefit occurred with this volume of additional storage (see Figure 5-4).

Figure 6-3 shows that the San Joaquin Scenario changes enabled the reservoirs to operate within their objective release throughout the duration of the 2 percent AEP Vernalis-centered storm. As a result, the flow at Stockton was within its channel capacity.

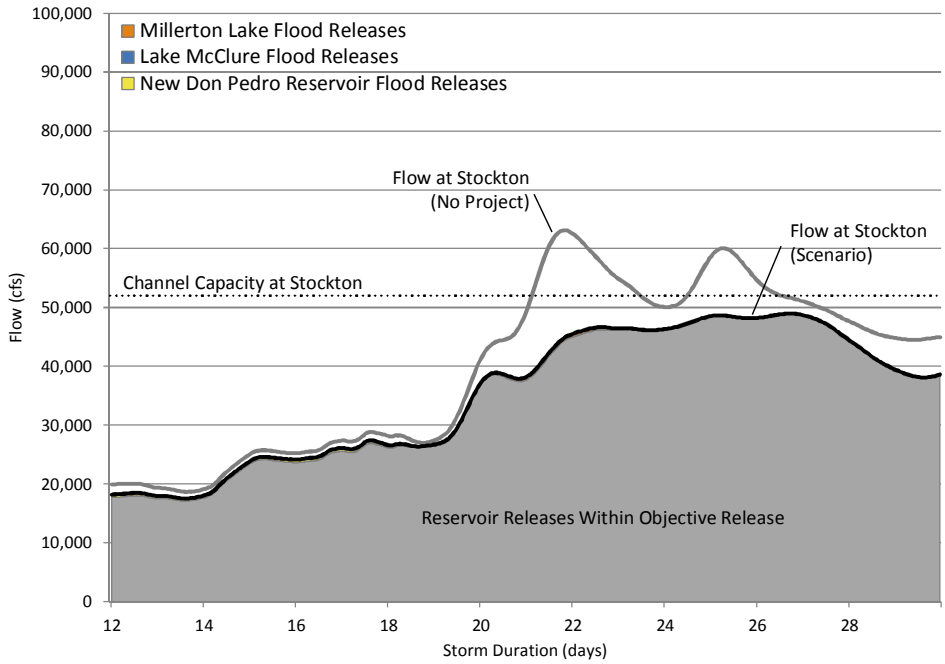


Figure 6-3. San Joaquin Scenario Simulated Reservoir Contributions to Flow at Stockton for 2 Percent AEP Vernalis-Centered Storm

For a 1 percent AEP Vernalis-centered storm, the three reservoirs generally operated within their objective releases during the beginning of the storm, removing the large first peak under the No Project condition (Figure 4-6). Nevertheless, the additional flood storage allocation was insufficient to prevent all flood releases. With changes in San Joaquin Scenario operational criteria, the highest peak flow at Stockton was reduced to 64,000 cfs. New Don Pedro Reservoir was the only reservoir that contributed flood releases during this highest peak flow at Stockton,

although a similar peak 2 days later was caused by flows from Millerton Lake and Lake McClure.

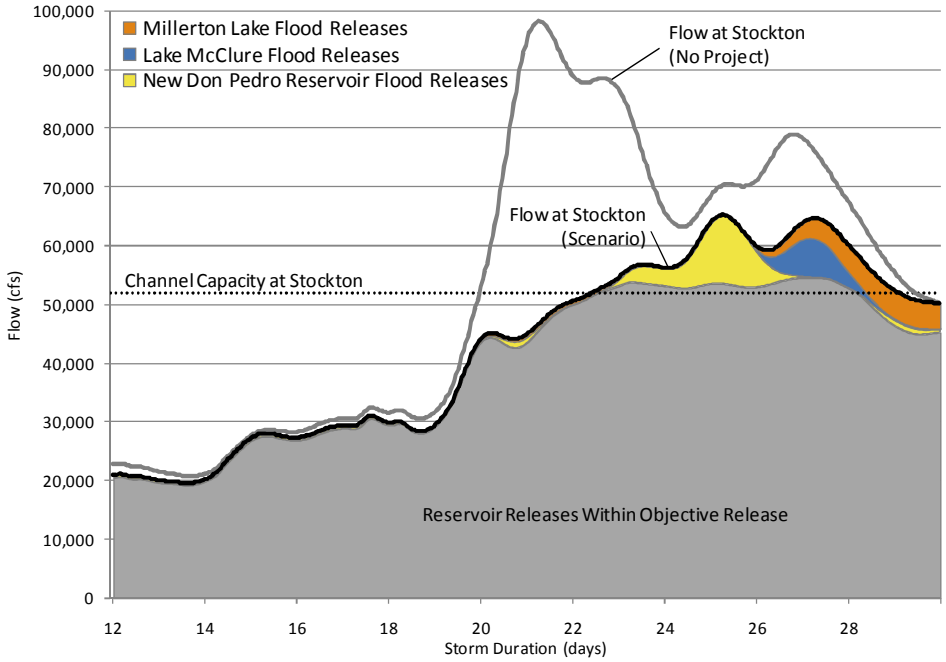


Figure 6-4. San Joaquin Scenario Simulated Reservoir Contributions to Flow at Stockton for 1 Percent AEP Vernalis-Centered Storm

The simulated effects of the San Joaquin Scenario on peak flows at various locations throughout the San Joaquin River Basin are summarized in Table 6-4.

Table 6-4. Simulated Effects of San Joaquin Scenario on Peak Flow for Vernalis-Centered Storm

Index Point	Overall Effect on Peak Flow	Simulated Decrease in Peak Flow (cfs) (percent)			
		2 Percent AEP		1 Percent AEP	
Chowchilla Bypass near Fresno River	Decrease	1,967	(16)	7,260	(30)
El Nido	Decrease	2,121	(10)	8,753	(24)
Near Newman	Decrease	1,993	(6)	15,402	(25)
At Maze Road Bridge	Decrease	15,733	(29)	34,918	(38)
Near Vernalis	Decrease	15,241	(24)	34,377	(35)
Stockton	Decrease	14,173	(22)	32,924	(34)

Model: HEC-5

Key:
AEP = annual exceedence probability
cfs = cubic feet per second

7.0 Effects of Operational Criteria Changes

This section discusses simulated flood management effects of the three reservoir operational scenarios considered, and then briefly discusses qualitatively other reservoir water uses and purposes.

7.1 Flood Management Benefits

The main objective of the 2012 CVFPP Reservoir Analysis was to determine whether changes to reservoir operational criteria could improve coordination among the reservoirs in the Central Valley flood management systems, thereby lowering downstream peak stage. Because HEC-5 does not take into account hydraulic conditions (e.g., unsteady flow, levee breaks), UNET was used to provide a more realistic estimate of riverine flow conditions resulting from reservoir operational criteria changes. Changes in the peak water surface elevation (stage) and volume of out-of-system flow were used to compare the simulated effects of reservoir operational criteria changes.

To compare the stage reduction, stage-frequency curves were generated at a series of locations throughout the Central Valley flood management systems. Peak stages for each storm AEP were connected to generate a stage-frequency curve for a given location. While not done in this reconnaissance-level analysis, stage-frequency curves can be used as inputs into an economic model, such as HEC-FDA, to quantify economic benefits associated with stage reduction.

A decrease in stage could result from (1) less water being released from reservoirs, or (2) an increase in water leaving a channel through an increase in levee failures. As a result, the volume of overland flow was quantified to better compare the effects of reservoir operational criteria changes.

The following flood management benefits resulting from the operational criteria scenarios considered were observed:

- In the Sacramento River Basin (Sacramento Scenario):
  - The largest decreases in peak stage occurred for 1 percent AEP or more frequent storm events.

- For the 1 percent AEP Sacramento-centered storm, the total volume of out-of-channel flow decreased by 13 percent (146 TAF).
- The largest flood management benefit was realized in small to midsized storm events (4, 2, and 1 percent AEP storms).

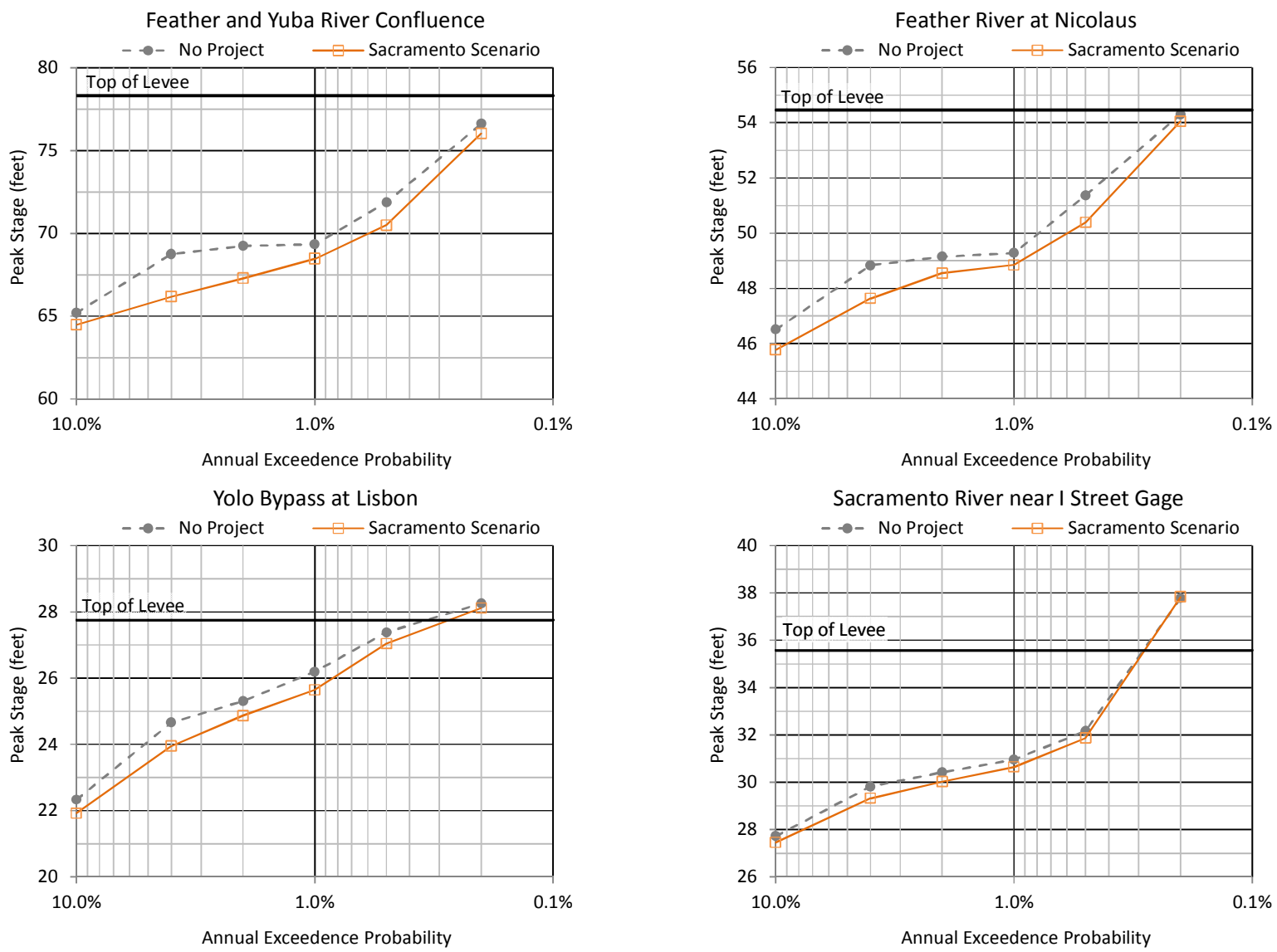
- In the San Joaquin River Basin (San Joaquin Scenario):
  - The largest decreases in peak stage occurred for 2 percent AEP or less frequent storm events.
  - The decrease in out-of-channel volume ranged from 15 percent to 39 percent (40 TAF to 206 TAF) for midsized to large-sized storm events (2, 1, and 0.5 percent AEP storms).

7.1.1 Sacramento River Basin

The Sacramento Scenario lowered the peak stage in the Feather River Basin and lower Sacramento River Basin (Figure 7-1).

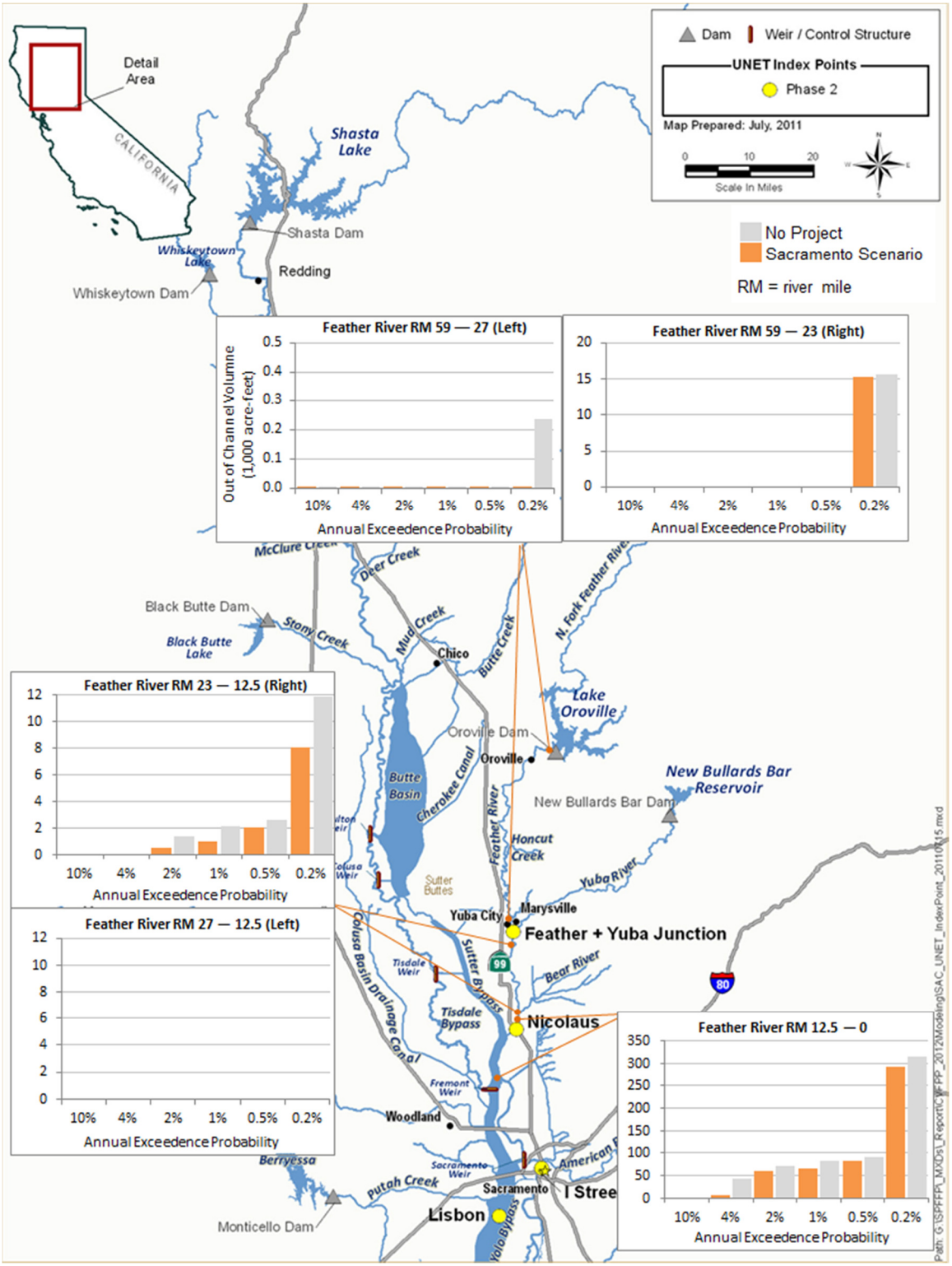
Changing Lake Oroville's operational criteria lowered the peak stage at the Feather-Yuba River confluence, the Feather River at Nicolaus, and the Sacramento River at the I Street gage by 1 percent (nearly 1 foot) for a 1 percent AEP storm. The peak stage at the Yolo Bypass near Lisbon decreased by 2 percent (0.5 foot) for a 1 percent AEP storm.

In addition to decreases in stage, the volume of out-of-channel flow decreased. Figures 7-2 and 7-3 show out-of-channel flow by reach. Throughout the Feather River, overall out-of-channel flow decreased for all storms. In the 60-mile reach of the Sacramento River downstream from the Sacramento Weir, out-of-channel flow was nearly eliminated for the 2 percent AEP Sacramento-centered storm. Figure 7-4 shows how the volume of out-of channel flow decreased throughout the entire Sacramento River Basin.



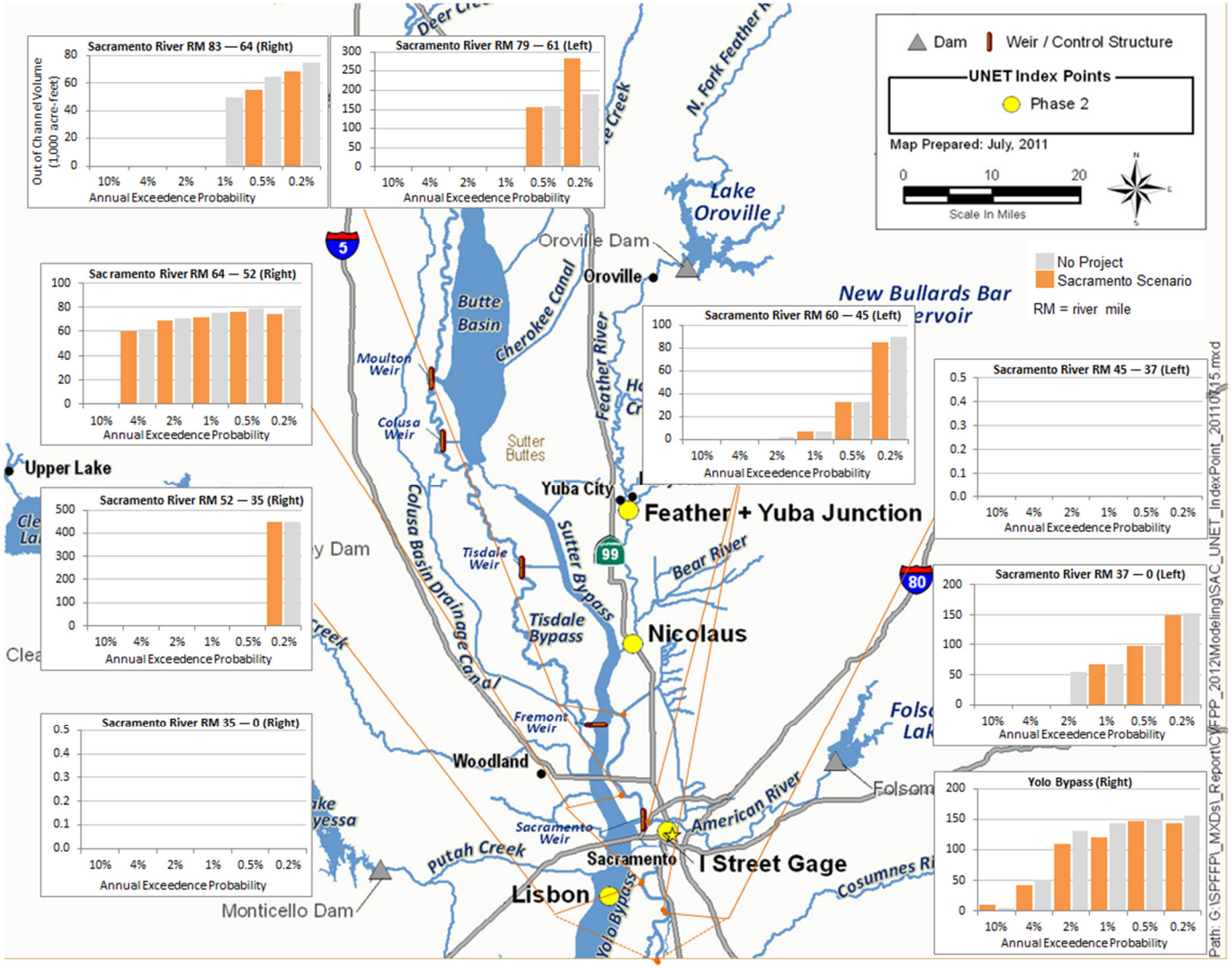
Model: UNET

Figure 7-1. Sacramento Scenario Stage-Frequency Curves for Sacramento-Centered Storm at Feather-Yuba River Confluence, Feather River at Nicolaus, Yolo Bypass at Lisbon, and Sacramento River at I Street Gage



Model: UNET

Figure 7-2. Sacramento Scenario Volume of Out-of-Channel Flow for Sacramento-Centered Storm Along Feather River (1,000 acre-feet)



Model: UNET

Note: Dotted lines represent that river miles extend past the map extents.

Figure 7-3. Sacramento Scenario Volume of Out-of-Channel Flow for Sacramento-Centered Storm Along Lower Sacramento River (1,000 acre-feet)

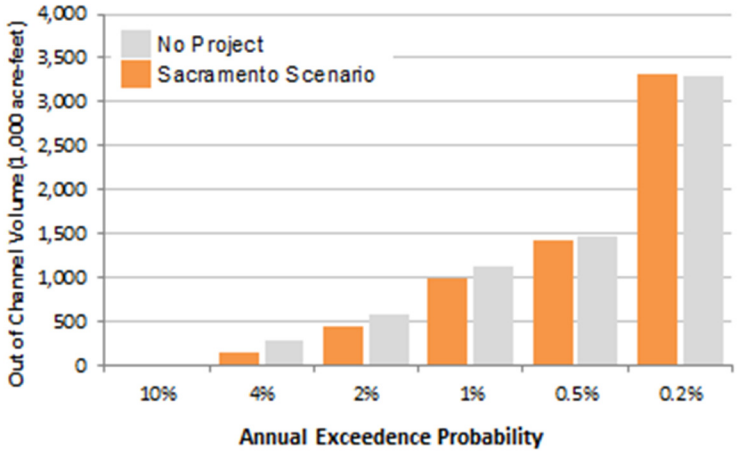


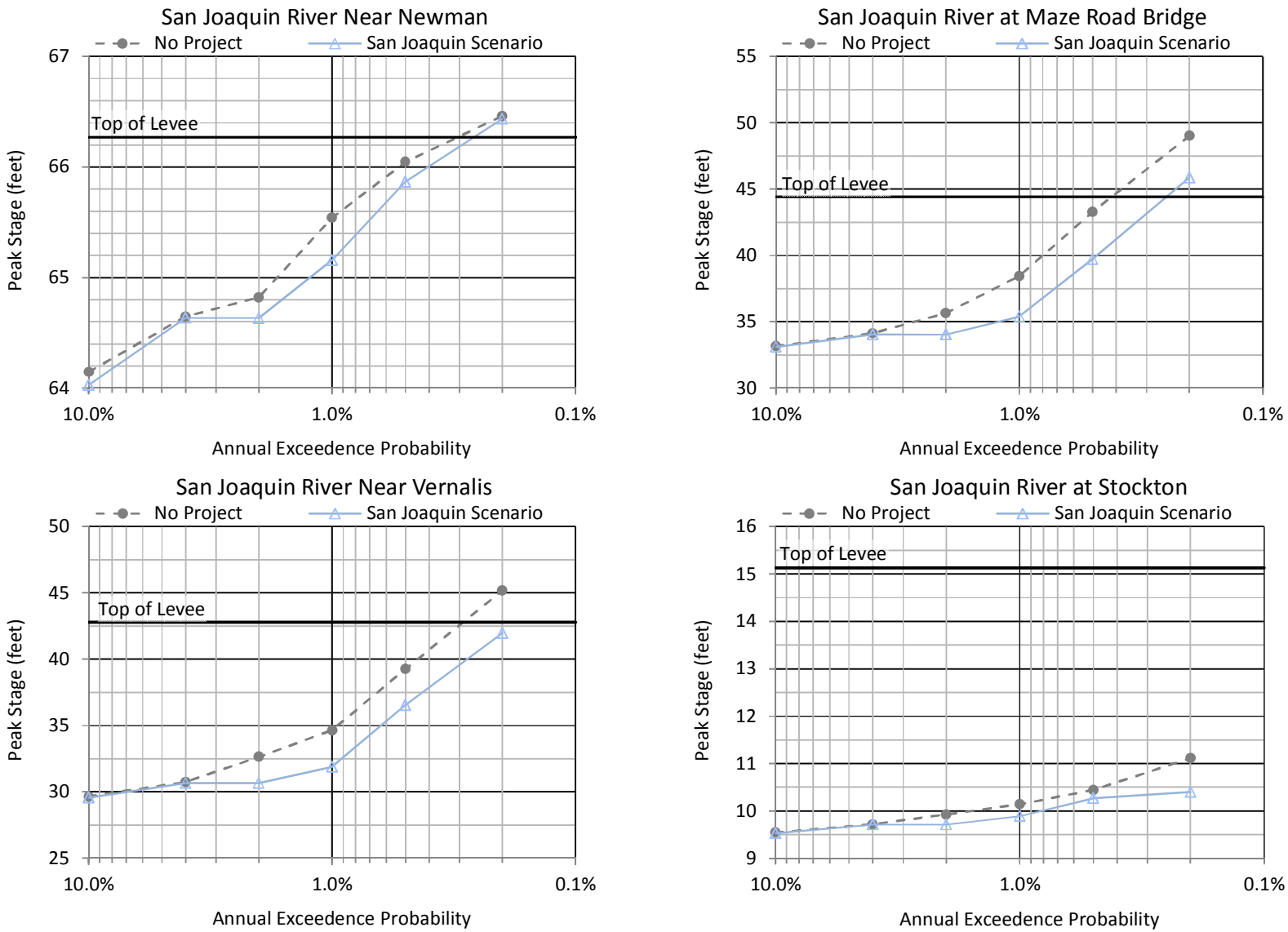
Figure 7-4. Sacramento Scenario Total Sacramento River Basin Out-of-Channel Flow Reductions

7.1.2 San Joaquin River Basin

The San Joaquin Scenario decreased the peak stage throughout the San Joaquin River Basin. Figure 7-5 shows the simulated decrease in stage at various locations along the lower San Joaquin River.

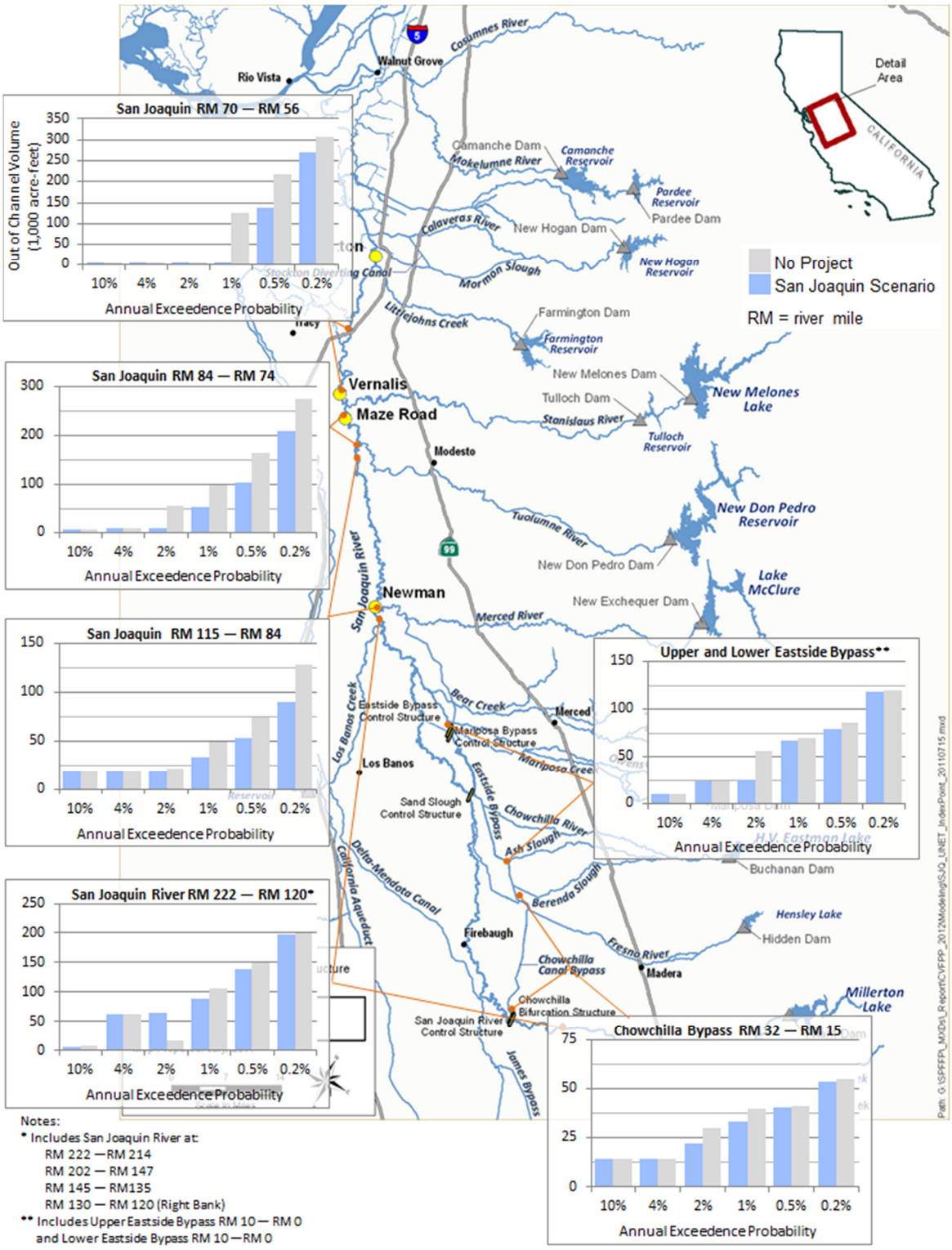
The peak stage on the San Joaquin River at Newman was slightly decreased by an average 0.2 percent from No Project conditions for all Vernalis-centered AEP storms because of influences from increased flood storage allocation at Millerton Lake and Lake McClure. At Stockton, the simulated peak stage for the 0.5 and 0.2 percent AEP storms was nearly the same (less than 0.03-foot difference).

In addition to decreases in stage, the volume of out-of-channel flow throughout the entire San Joaquin River Basin also decreased. Figure 7-6 shows the out-of-channel flow by reach and Figure 7-7 shows the total out-of-channel flow. In the 14-mile reach downstream from Vernalis, the out-of-channel flow was nearly eliminated for the 1 percent AEP Vernalis-centered storm. For the 0.5 percent AEP storm, out-of-channel flow decreased by 77 TAF for the San Joaquin Scenario. The volume of out-of-channel flow did increase for in the downstream portion of the San Joaquin River for some AEP storms, but the volume decreased in the Chowchilla and Eastside bypasses; overall, the net change in out-of channel flow was a decrease.



Model: UNET

Figure 7-5. San Joaquin Scenario Stage-Frequency Curves for Vernalis-Centered Storm at San Joaquin River near Newman, at Maze Road Bridge, near Vernalis, and at Stockton



Notes:
* Includes San Joaquin River at:
RM 222 — RM 214
RM 202 — RM 147
RM 145 — RM135
RM 130 — RM 120 (Right Bank)
** Includes Upper Eastside Bypass RM 10 — RM 0 and Lower Eastside Bypass RM 10 — RM 0

Model: UNET

Figure 7-6. San Joaquin Scenario Volume of Out-of-Channel Flow for Vernalis-Centered Storm

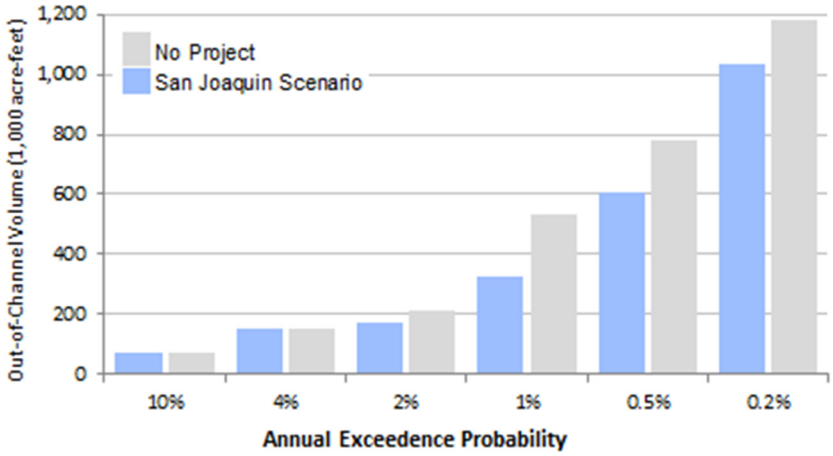


Figure 7-7. San Joaquin Scenario Total San Joaquin River Basin Out-of-Channel Flow

7.2 Other Reservoir Water Uses

Aside from providing flood management benefits, changing operational criteria for flood damage reduction could affect a multitude of other reservoir water uses and purposes. Adjusting the amount of flood storage and magnitude of objective releases may alter the volume of reservoir storage available for peak season water uses. This may result in economic effects on the following:

- Water supply reliability
- Hydropower generation
- Recreational opportunities
- Groundwater storage
- Instream requirements

7.2.1 Water Supply Reliability

In addition to flood management, water supply is one of the major purposes for multipurpose reservoirs in the Central Valley. The majority of precipitation in California falls between October and March; therefore, changes to reservoir operational criteria for peak flow reduction are focused on that period. Changes in reservoir flood space allocation and objective release during the wet season could alter the ability of a reservoir to fill by the end of the wet season and to be ready to meet water supply demands, which generally peak in summer months. On the basis of a high level appraisal, the impacts to water supply reliability resulting from operational criteria changes considered in this analysis could possibly be

effectively mitigated; a more detailed analysis to better quantify benefits to flood management and potential adverse impacts and associated mitigation is needed.

7.2.2 Hydropower Generation

Hydropower generation depends on elevation of the water in a reservoir (i.e., head). Changes to reservoir operational criteria would alter reservoir storage and available head in a reservoir during flood season and possibly during other times of the year (if the reservoir does not fill as a result of operational criteria changes), and thus decrease power generation and revenue. In addition, alternative sources of energy may be needed to account for any changes. The magnitude of the economic cost to hydropower could be determined from factors such as net generation of power and power market prices.

7.2.3 Recreational Opportunities

Many of the Central Valley multipurpose reservoirs are major recreational venues. A study performed by DWR on recreational sites in Northern California estimated that 2.5 million people visit Northern California lakes and reservoirs per year (DWR, 2004). Recreational opportunities are proportional to reservoir water surface area. In general, the greater the surface area, the more recreational activities are available. Changes to reservoir operational criteria would alter reservoir storage during flood season and other times of the year (if the reservoir did not fill as a result of operational criteria changes), and thus change water surface area. Aquatic recreational activity is especially sensitive to such changes. The value of economic effects would depend on season, type of recreational activities, etc.

7.2.4 Groundwater Storage

Changes in water supply availability from a reservoir could vary the use of other water supplies, such as groundwater. A change in groundwater pumping would affect regional groundwater storage conditions and, thus, access to groundwater by other parties could change. Also, interaction between surface water and groundwater could differ. Modifying the amount of space required for flood storage may alter the timing and magnitude of flows released from a reservoir. Reservoir water and groundwater could be used conjunctively to increase water supply while keeping space available in the reservoir for flood retention.

7.2.5 Instream Requirements

Reservoirs are also often operated to meet various requirements for fisheries, vegetation, wildlife, water quality, etc. Changes to reservoir

operational criteria during the wet season could alter water availability to meet these requirements and, thus, have an economic impact.

Modifying reservoir operational criteria may affect anadromous fish survival and reproduction rates by altering seasonal water flows and temperatures in the Sacramento and San Joaquin rivers. For example, altering river hydraulics may affect the flows required to move juvenile salmonids through the system. Changes in water temperatures, potentially resulting from a reduction in surface storage during critical periods, may affect salmon production. This change may also have an economic effect on recreational and commercial fishing for certain species.

Vegetation and wildlife may be affected if implementing any of these scenarios changes riparian habitat, modifies sensitive natural communities, affects federally protected wetlands, or conflicts with local policies, ordinances protecting biological resources, and adopted habitat conservation plans. For example, native riparian and wetland plants may be affected because changes in objective flows could potentially change the duration of time and frequency that current vegetation is submerged.

Changes in reservoir operational criteria also may affect water quality parameters such as pH, dissolved oxygen, turbidity, salinity, and temperature. These changes may alter treatment requirements for water supplies, crop yields for sensitive crops, amounts of sedimentation in canals, etc.

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9.0 Acronyms and Abbreviations

1-D ..... one-dimensional

AEP ..... annual exceedence probability

BO ..... Biological Opinion

Board ..... Central Valley Flood Protection Board

cfs ..... cubic feet per second

Comprehensive Study Sacramento and San Joaquin River Basins Comprehensive Study, California

CU ..... Conjunctive Use

CVFPP ..... Central Valley Flood Protection Plan

CVP ..... Central Valley Project

Delta ..... Sacramento-San Joaquin Delta

DWR ..... California Department of Water Resources

ESRD ..... Emergency Spillway Release Diagram

F-BO ..... Forecast-Based Operations

F-CO ..... Forecast-Coordinated Operations

FEMA ..... Federal Emergency Management Agency

FERC ..... Federal Energy Regulatory Commission

FPO ..... Folsom Dam Permanent Operations

FWUA ..... Friant Water Users Authority

HEC ..... Hydrologic Engineering Center

ID ..... Irrigation District

JFP ..... Joint Federal Project

NMFS ..... National Marine Fisheries Service

NOAA ..... National Oceanic and Atmospheric Administration

PEIS/R ..... Program Environmental Impact Statement/Report

Reclamation ..... U.S. Department of the Interior, Bureau of Reclamation

RO ..... Reservoir Operations

ROD ..... Record of Decision

SAC ..... Sacramento

SAFCA ..... Sacramento Area Flood Control Agency
SJQ ..... San Joaquin
SJRRP ..... San Joaquin River Restoration Program
SPFC ..... State Plan of Flood Control
SWP ..... State Water Project
TAF ..... thousand acre-feet
USACE ..... U.S. Army Corps of Engineers
USFWS ..... U.S. Fish and Wildlife Service
V9B ..... Version 9B
YCWA ..... Yuba County Water Agency

STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

Attachment 8C

Riverine Channel Evaluations

CENTRAL VALLEY
FLOOD MANAGEMENT
PLANNING PROGRAM

FloodSAFE
CALIFORNIA

Public Draft

2012 Central Valley Flood Protection Plan

Attachment 8C: Riverine Channel Evaluations

January 2012

STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

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1.0 Introduction

This section states the purpose of this attachment, gives background information (including a description of planning areas, goals, and approaches), overviews the Sacramento and San Joaquin river basins, Stockton area, and the San Joaquin Area Flood Control Agency (SJAFCA) Flood Protection Restoration Project, and provides an overview of the report organization.

1.1 Purpose of this Attachment

As part of development of the 2012 Central Valley Flood Protection Plan (CVFPP), hydraulic modeling was performed for the Sacramento and San Joaquin river basins, Stockton area, and Sacramento-San Joaquin River Delta (Delta) to support flood management system evaluations. The analysis in the Sacramento and San Joaquin river basins was performed using hydrology and hydraulic models initially developed as part of the *Sacramento and San Joaquin River Basins Comprehensive Study* (Comprehensive Study) (USACE, 2002a).

The Comprehensive Study did not develop impact areas or models on the Calaveras River (including Mormon Slough and the Stockton Diverting Canal) and Bear Creek in Stockton, even though the streams include State Plan of Flood Control (SPFC) levees. Therefore, this attachment also documents the development and use of hydrology and hydraulic models for those two streams in the Stockton area. Note that hydraulic modeling for the Delta is documented in Attachment 8D: Estuary Channel Evaluation.

Results from the modeling were used to describe the hydraulic performance of the existing flood management system (No Project condition) and to simulate management actions for various approaches for improving the system. Modeling results were also used as input to flood damage evaluation models to estimate economic values of flood damages. All modeling was done at a reconnaissance level for use in comparing approaches on a systemwide basis, and should not be used for any other purpose.

This attachment documents riverine channel hydraulic modeling methodology and results for the Sacramento and San Joaquin river basins and Stockton area for the No Project condition and each of the following CVFPP approaches:

- Achieve SPFC Design Flow Capacity Preliminary Approach
- Protect High Risk Communities Preliminary Approach
- Enhance Flood System Capacity Preliminary Approach
- State Systemwide Investment Approach

The riverine channel hydraulic modeling of the No Project condition was done to provide a baseline for comparison with the four approaches. While the No Project condition is meant to describe the existing conditions of flood management systems in the Central Valley, it also includes projects that have been authorized and have funding, or that have started construction or implementation. The No Project condition includes the following:

- Levee improvements in south Yuba County implemented by the Three Rivers Levee Improvement Authority (TRLIA) since 2004 (TRLIA, 2011)
- Natomas Levee Improvement Program by the Sacramento Area Flood Control Agency (SAFCA) (SAFCA, 2011)
- Folsom Dam Joint Federal Project to improve the ability of Folsom Dam to manage major floods by allowing more water to be safely released earlier in a storm event, leaving more storage capacity for capturing peak inflow (Reclamation, 2009)
- Levee improvements along the American River to safely pass a flow of 160,000 cubic feet per second (cfs) as part of the American River Common Features Project (SAFCA, 2011)
- Marysville levee improvements (USACE, 2009b)

Riverine channel hydraulic modeling developed flow rates (discharge in cubic feet per second (cfs)) and water surface elevations (stage in feet above the National Geodetic Vertical Datum of 1929 (NGVD29)) for various theoretical floods in the Sacramento and San Joaquin river basins for each CVFPP approach. Elevations are in NGVD29 instead of the more commonly used North American Vertical Datum of 1988 (NAVD88) for consistency with the Comprehensive Study.

This attachment documents the following modeling results:

- The discharge-frequency (Q-F) relationship for in-river locations in the Sacramento and San Joaquin river basins and Stockton area. Discharge is in cfs and storm event frequency, or annual exceedence probability

(AEP), is expressed in percentage (i.e., 1 percent AEP, or a storm with a 100-year return period).

- The stage-frequency (S-F) relationship for in-river locations inside the Sacramento and San Joaquin river basins and Stockton area. Stage is in feet above the NGVD29 and frequency (AEP) is expressed in percentage.

- Out-of-system volume from river reaches in the Sacramento and San Joaquin river basins and Stockton area. This represents the total volume of water that leaves a section of channel and enters the adjacent floodplain, typically through a breach in a levee. Out-of-system volume is expressed in thousand acre-feet (TAF).

These modeling results were used to assess the hydraulic performance at a systemwide scale under the No Project condition and each of the four approaches. After completion of the 2012 CVFPP, new riverine and floodplain models developed by DWR's Central Valley Floodplain Evaluation and Delineation Program (CVFED) will be become available for use in the 2017 CVFPP.

1.2 Background

As authorized by Senate Bill 5, also known as the Central Valley Flood Protection Act of 2008, the California Department of Water Resources (DWR) has prepared a sustainable, integrated flood management plan called the CVFPP, for adoption by the Central Valley Flood Protection Board (Board). The 2012 CVFPP provides a systemwide approach to protecting lands currently protected from flooding by existing facilities of the SPFC, and will be updated every 5 years.

As part of development of the 2012 CVFPP, a series of technical analyses were conducted to evaluate hydrologic, hydraulic, geotechnical, economic, ecosystem, and related conditions within the flood management system and to support formulation of system improvements. These analyses were conducted in the Sacramento River Basin, San Joaquin River Basin, and Delta.

1.3 CVFPP Planning Areas

For planning and analysis purposes, and consistent with legislative direction, two geographical planning areas were important for CVFPP development (Figure 1-1):

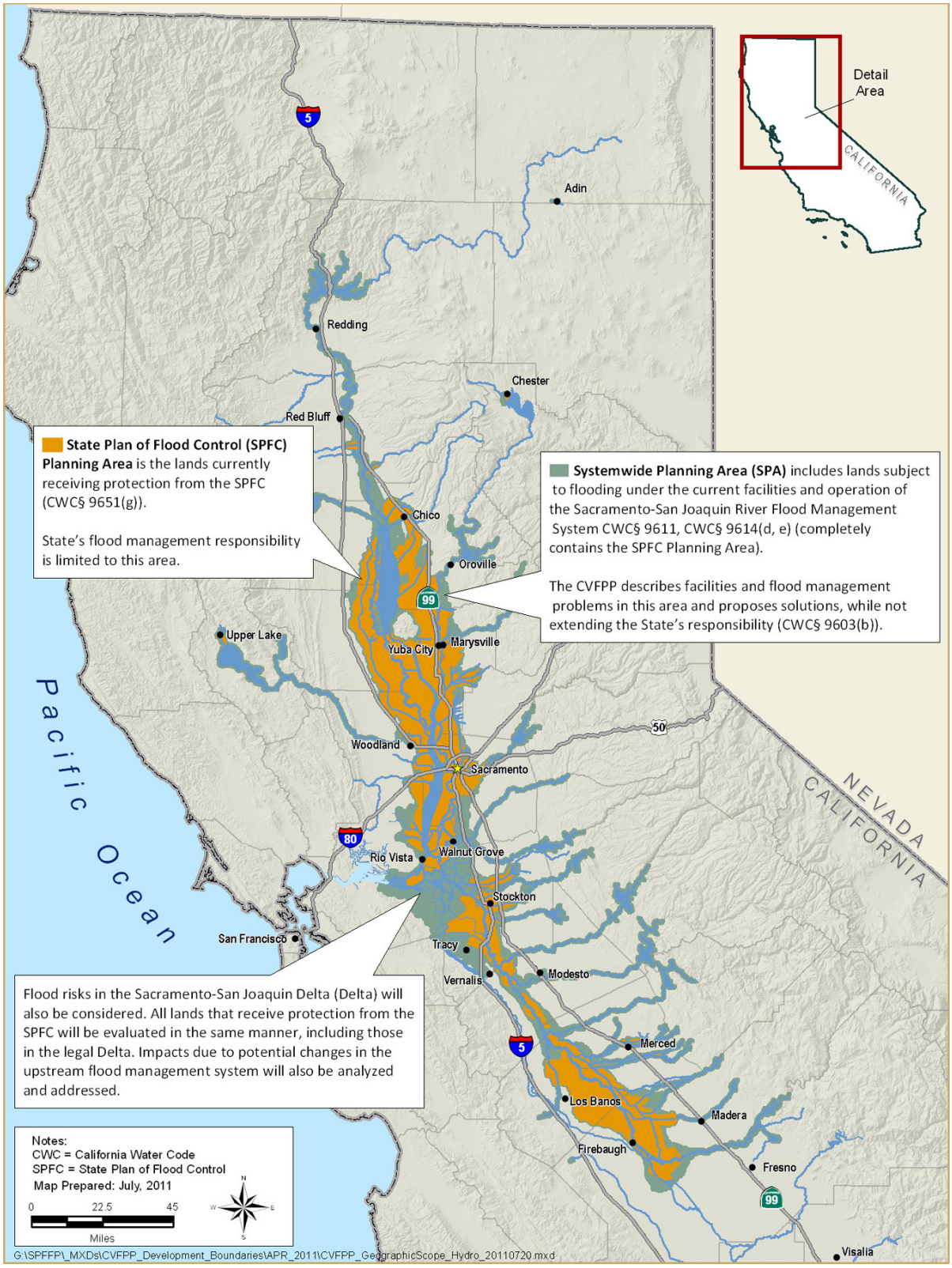

Figure 1-1. Central Valley Flood Protection Plan Planning Areas

- **SPFC Planning Area –** This area is defined by the lands currently receiving flood protection from facilities of the SPFC (see *State Plan of Flood Control Descriptive Document* (DWR, 2010)). The State of California's (State) flood management responsibility is limited to this area.

- **Systemwide Planning Area** – This area includes the lands that are subject to flooding under the current facilities and operation of the Sacramento-San Joaquin River Flood Management System (California Water Code Section 9611). The SPFC Planning Area is completely contained within the Systemwide Planning Area which includes the Sacramento River Basin, San Joaquin River Basin, and Delta regions.

Planning and development for the CVFPP occurs differently in these planning areas. The CVFPP focused on SPFC facilities; therefore, evaluations and analyses were conducted at a greater level of detail within the SPFC Planning Area than in the Systemwide Planning Area.

Hydraulic modeling was performed for major waterways and river channels within the SPFC Planning Area and Delta. This attachment describes the riverine modeling in the Sacramento River Basin, which comprises the entire northern part of the SPFC Planning Area, and the riverine modeling for the San Joaquin River Basin, which includes almost the entire portion of the southern part of the SPFC Planning Area. Hydraulic modeling of the Stockton area in the San Joaquin River Basin was also conducted and covers portions of the City of Stockton and vicinity on reaches of the Calaveras River, Mormon Slough, Stockton Diverting Canal, and Bear Creek that are protected by SPFC levees and facilities. Modeling results from the Sacramento and San Joaquin river basins provided the upstream boundary conditions for Delta hydraulic modeling that is described in Attachment 8D – Estuary Channel Evaluations.

1.4 2012 CVFPP Planning Goals

To help direct CVFPP development to meet legislative requirements and address identified flood-management-related problems and opportunities, a primary and four supporting goals were developed:

- **Primary Goal** – Improve Flood Risk Management

- **Supporting Goals**:

  - Improve Operations and Maintenance

- Promote Ecosystem Functions
- Improve Institutional Support
- Promote Multi-Benefit Projects

Modeling results in this attachment demonstrate how each of the approaches (described below) meets the primary goal.

1.5 2012 CVFPP Planning Approaches

In addition to the **No Project** condition, three fundamentally different approaches to flood management were initially compared to explore potential improvements in the Central Valley. These preliminary approaches are not alternatives; rather, they bracket a range of potential actions and help explore trade-offs in costs, benefits, and other factors important in decision making. The preliminary approaches are as follows:

- **Achieve SPFC Design Flow Capacity** – Address capacity inadequacies and other adverse conditions associated with existing SPFC facilities, without making major changes to the footprint or operation of those facilities.
- **Protect High Risk Communities** – Focus on protecting life safety for populations at highest risk, including urban areas and small communities.
- **Enhance Flood System Capacity** – Seek various opportunities to achieve multiple benefits through enhancing flood system storage and conveyance capacity.

Comparing these preliminary approaches helped identify the advantages and disadvantages of different combinations of management actions, and demonstrated opportunities to address the CVFPP goals to different degrees.

Based on this evaluation, a **State Systemwide Investment Approach** was developed that encompasses aspects of each of the preliminary approaches to balance achievement of the goals from a systemwide perspective, and includes integrated conservation elements. Figure 1-2 illustrates this plan formulation process.

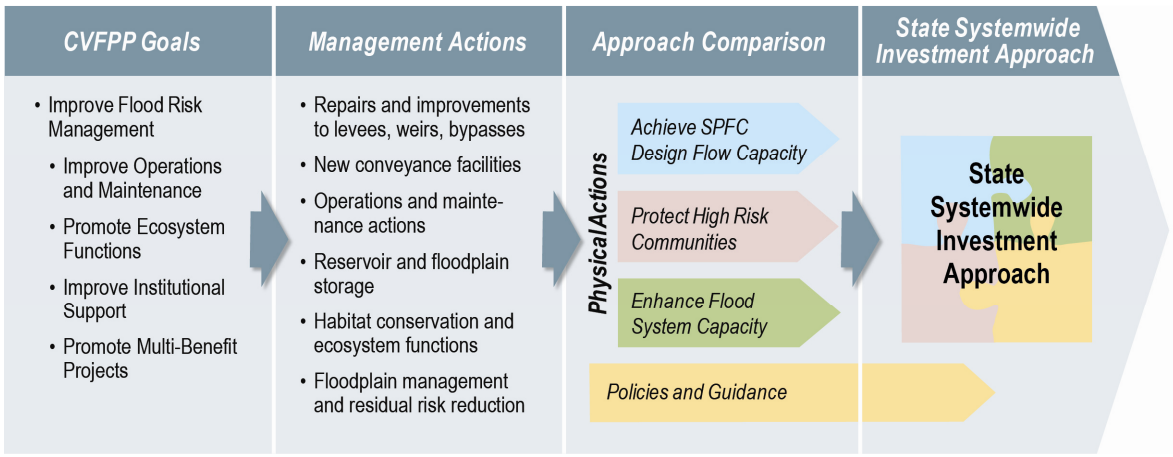

Figure 1-2. Formulation Process for State Systemwide Investment Approach

1.6 Sacramento River Basin

The flood management system in the Sacramento River Basin manages flows from approximately 27,000 square miles in the Sierra Nevada, Central Valley, and Coast Ranges in Northern California. Major tributaries to the Sacramento River include the Feather, Yuba, Bear, and American rivers, which discharge to the Sacramento River from the east. Additional tributaries, such as Cottonwood Creek, enter the mainstem of the Sacramento from the west and can provide significant flood flows. Flood management facilities in the Sacramento Valley include the following:

- Six dams and reservoirs that contribute to flood management (Shasta, Black Butte, Folsom, Oroville, New Bullards Bar, and Indian Valley dams)
- Levees along the Sacramento River and major tributaries
- Four leveed bypasses (Sutter, Tisdale, Sacramento, and Yolo bypasses)
- Five weirs (Moulton, Colusa, Tisdale, Fremont, and Sacramento weirs)
- Two sets of outfall gates (Butte Slough, Knights Landing)
- Six major drainage pumping plants (Sutter Bypass 1, 2, and 3, American River 1 and 2, and Magpie Creek)

1.7 San Joaquin River Basin

The flood management system in the San Joaquin River Basin manages flows from approximately 16,700 square miles in the Sierra Nevada, Central Valley, and Coast Ranges in Central California. Major tributaries to the San Joaquin River include the Mokelumne, Calaveras, Stanislaus, Tuolumne, Merced, and Fresno rivers, and Littlejohns Creek, which discharge to the San Joaquin River from the east. Streams on the west side of the basin, including Los Banos, Orestimba, and Del Puerto creeks, are intermittent, and their flows rarely reach the San Joaquin River except during large floods. In addition, floodflows from Kings River are diverted north into the San Joaquin River during periods of high flow in the Tulare Lake Basin. Flood management facilities in the San Joaquin Valley include the following:

- Levees along the San Joaquin River and major tributaries
- Three leveed bypasses (Eastside, Chowchilla, and Mariposa bypasses)
- Six in-stream control structures (Chowchilla Canal Bypass, San Joaquin River, Mariposa Bypass, Eastside Bypass, Sand Slough, and San Joaquin River Structure)
- Sixteen dams and reservoirs that contribute to flood management (Friant, New Exchequer, New Don Pedro, Hidden, Buchanan, New Melones, Los Banos Detention, Pardee, Camanche, New Hogan, Little Panoche Detention, Mariposa, Owens, Burns, Castle, and Bear dams)
- Five major pumping plants (Lower San Joaquin River, Mormon Slough 1 to 3, and Weatherbee Lake)

1.8 Stockton Area

The Stockton area as defined for this analysis includes portions of the City of Stockton and vicinity, as well as Lower Roberts Island, as shown on Figure 1-3. These hydraulic modeling extents were selected based on available data and the location of existing SPFC facilities.

This region is inside the SPFC planning area but no study was conducted there for the Comprehensive Study. Because of its location in the Delta, hydraulic modeling for Lower Roberts Island (STK01 on Figure 1-3) was conducted using the RMA Delta Model (see Attachment 8D: Estuary Channel Evaluations for details). This technical attachment focuses on areas labeled STK06 through STK10 on Figure 1-3.

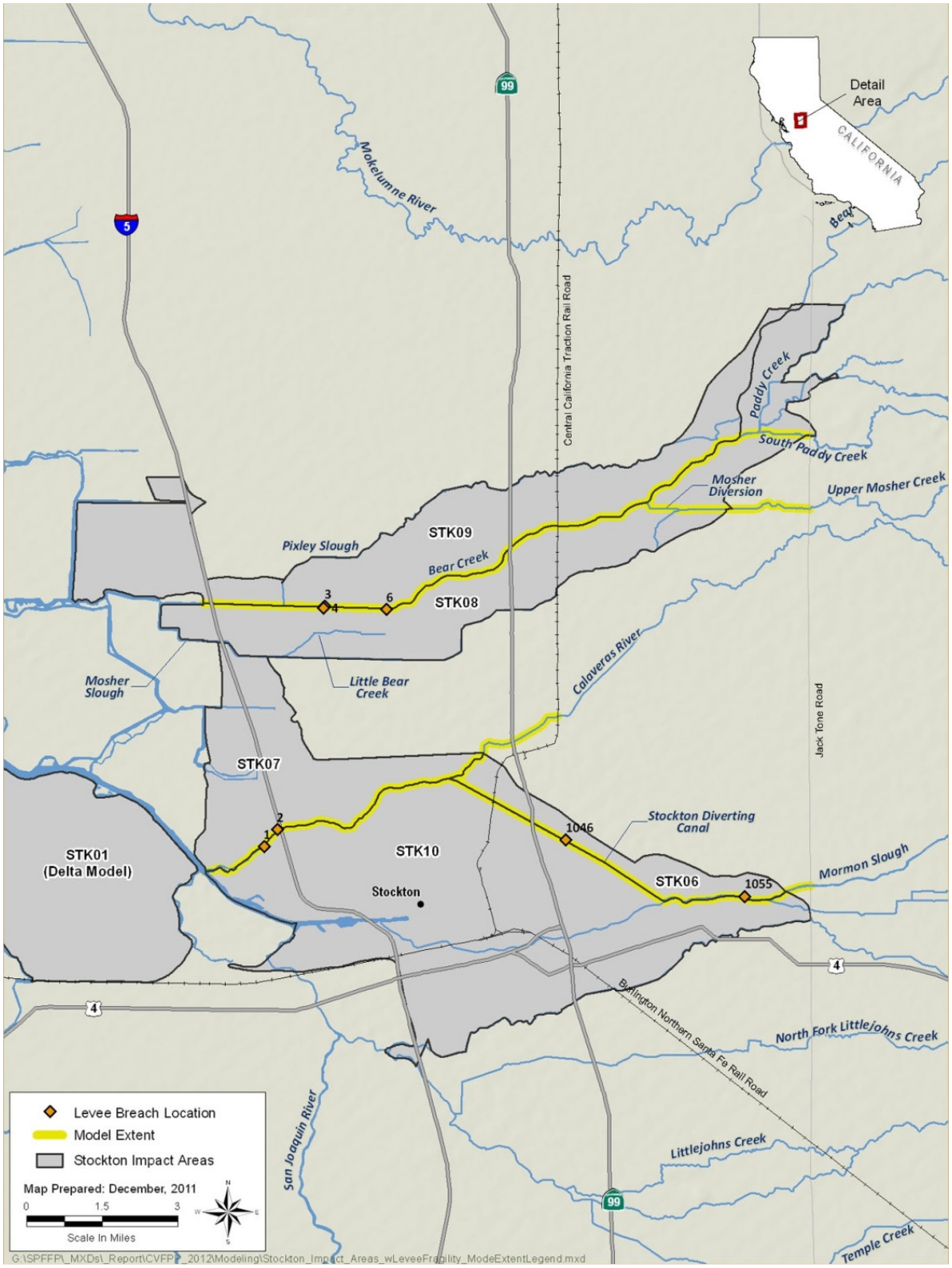

Figure 1-3. Model Extents for Stockton Area Analysis

The Stockton area streams include Bear Creek, Paddy Creek, Calaveras River, Mormon Slough/Stockton Diverting Canal, and Mosher Slough. Along the Bear Creek, SPFC levees extend from South Paddy Creek at Jack Tone Road to Bear Creek's crossing with Interstate 5 on the northwest side of the City of Stockton. The SPFC levees along Mormon Slough extend from Jack Tone Road to where it enters the Stockton Diverting Canal. The levees continue along the Stockton Diverting Canal to where it ends at the Calaveras River and then to the Calaveras River's crossing with Interstate 5 on the west side of the City of Stockton. The SPFC facilities also include three pumping plants on the Stockton Diverting Canal.

In 1998, the San Joaquin Area Flood Control Agency (SJAFCA) completed both hydrologic and hydraulic analyses for streams near the City of Stockton as part of its Flood Protection Restoration Project. The objective of this analysis was to identify solutions to resolve the finding that four streams (Bear Creek, Calaveras River, Mormon Slough/Diverting Canal, and Mosher Slough) were deficient in containing the 100-year flood flows in accordance with Federal Emergency Management Agency (FEMA) requirements. To accomplish this, SJAFCA reviewed the hydrology used by FEMA to make its deficiency finding. Additional information regarding the assumptions made in verifying and developing the hydrology and hydraulics can be found in the Flood Protection Restoration Project's *Final Technical Memorandum No. 1 – Hydrology* (SJAFCA, 1998a) and *Final Technical Memorandum No. 2 – Hydraulics* (SJAFCA, 1998b).

Models developed from the SJAFCA Flood Protection Restoration Project fit the purpose of this analysis and were used to assess the performance of the streams in the Stockton area.

1.9 Report Organization

Organization of this document is as follows:

- Section 1 describes the purpose of this attachment, and provides an overview of the CVFPP and the Sacramento and San Joaquin river basins.
- Section 2 summarizes results and findings for CVFPP riverine hydraulic modeling in the Sacramento and San Joaquin river basins.
- Section 3 describes the overall CVFPP modeling methodology, the CVFPP hydraulic model, and the model selection process for the Sacramento and San Joaquin river basins.
- Section 4 provides complete results for the riverine hydraulic analysis by CVFPP approach for the Sacramento and San Joaquin river basins.
- Section 5 provides methodology and results for the Stockton area analysis.
- Section 6 contains references for the sources cited in this document.
- Section 7 lists acronyms and abbreviations used in this document.

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2.0 Results Summary and Findings

Results from hydraulic modeling of the Sacramento and San Joaquin rivers and their major tributaries are summarized in Figures 2-1 through 2-12, which map the changes in stage between the No Project condition and the four CVFPP approaches throughout the system. Methodology and results for the Stockton area are contained in Section 5.

Maps are only included for AEPs of 2 percent, 1 percent, and 0.5 percent (50-, 100-, and 200-year return period) because the flood management system doesn't exhibit significant differences between the No Project and four approaches for the 10 percent and 4 percent (10- and 25-year return period), and similarly the 0.2 percent AEP flood (500-year return period) overwhelms the flood management system in all cases.

2.1 Achieve SPFC Design Flow Capacity Approach

Figures 2-1 through 2-3 indicate the changes in stage that would result from repairing or improving all SPFC levees to meet their design flows (Section 3.6, Tables 3-1 and 3-2) as specified by the 55/57 design profiles. Overall, for all of the AEPs there would be fewer upstream levee breaks, resulting in increased flows and higher water surface elevations in downstream reaches for both the Sacramento and San Joaquin rivers. In the San Joaquin River, higher stages (more than three feet) would be seen in the bypass system because of the reduction in levee breaks in the bypass. This would carry over into the San Joaquin River downstream from the Merced River as these increased flows leave the bypass system and enter the San Joaquin River.

2.2 Protect High Risk Communities Approach

Figures 2-4 through 2-6 indicate the changes in stage that would result from repairing or improving all urban levees to meet the 0.5 percent AEP (200-year) design criteria (Section 3.7, Tables 3-1 and 3-2), and providing increased protection to selected small communities. Since this approach would improve only urban and small community levees, other levees would be untouched and function as in the No Project condition. Stage increases of a foot or less would be seen on the lower Sacramento River as a result of increased protection for upstream urban areas. Little change would be seen

along the San Joaquin River with maximum changes of much less than a foot near the Tuolumne River confluence.

2.3 Enhance Flood System Capacity Approach

Figures 2-7 through 2-9 indicate the changes in stage that would result from modifying the flood management system as described in Section 3.8 and shown in Tables 3-1 and 3-2. Key components of the approach are added upstream reservoir storage, improving SPFC levees to their design flow capacity, improving urban levees to pass the 0.5 AEP flood, widened and new bypasses, levee setbacks, and floodplain storage. Added upstream storage would result in lower stages in the upper Feather, San Joaquin, Merced, and Tuolumne rivers. Floodplain storage and levee setbacks would result in lower stages in the Sutter Bypass and lower Feather River, as well as the Sacramento River downstream from the Tisdale Weir. These lower stages would continue downstream in the Yolo Bypass and lower Sacramento River. In the San Joaquin River, a reduction in levee breaks in the Chowchilla and Eastside bypasses because of fixes to SPFC levees would result in higher stages (more than three feet higher) because of the increased the volume of water remaining in the bypasses all the way from the San Joaquin River to the Merced River. This would carry over into the San Joaquin River downstream from the Merced River as these increased flows leave the bypass system and enter the San Joaquin River. Stages downstream from the Tuolumne River to Stockton would be lowered as a result of floodplain storage and levee setbacks.

2.4 State Systemwide Investment Approach

Figures 2-10 through 2-12 indicate the changes in stage resulting from repairing or improving all urban levees to meet the 0.5 percent AEP (200-year) design criteria and other improvements in the State Systemwide Investment Approach (Section 3.9, Tables 3-1 and 3-2). Because this approach would improve only urban levees, other levees would be untouched and function as in the No Project condition. Stage decreases would be seen in the upper Feather River as a result of the new bypass from the Feather River to the Butte Basin (Biggs Bypass), which would also result in a slight increase in stage in the upper end of the Sutter Bypass. Stages would be lower in the Sutter Bypass and the Sacramento River downstream from the Tisdale Weir as a result of the levee setbacks in the Sutter Bypass and lengthening of the Fremont Weir. Stages would also be lower in portions of the Yolo Bypass as a result of levee setbacks. Slight stage increases (one foot or less) would be seen on the lower Sacramento River as a result of the increased protection for upstream urban areas.

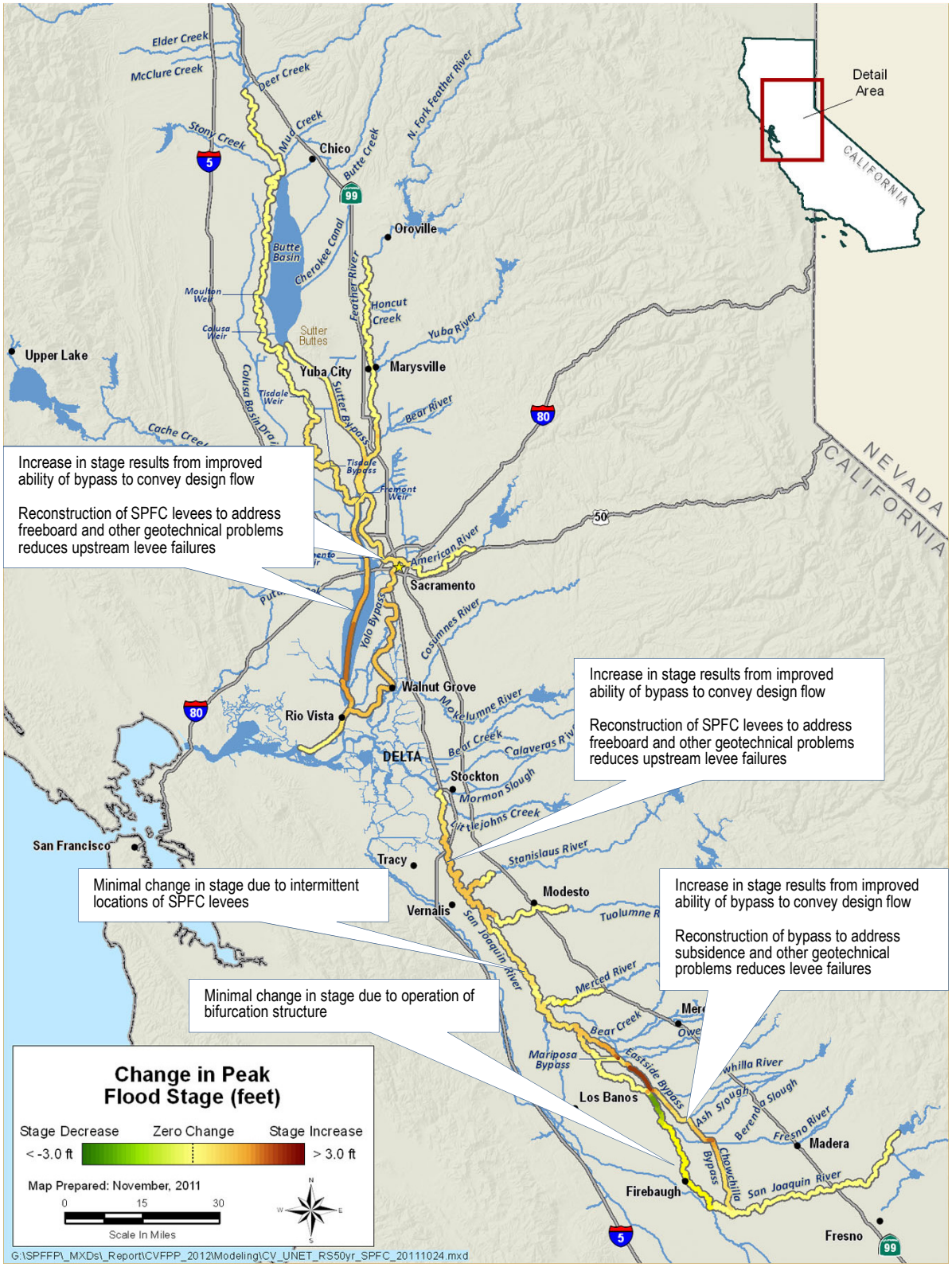


Figure 2-1. Stage Changes from No Project Condition to Achieve SPFC Design Flow Capacity Approach – 2 Percent AEP (50-Year)

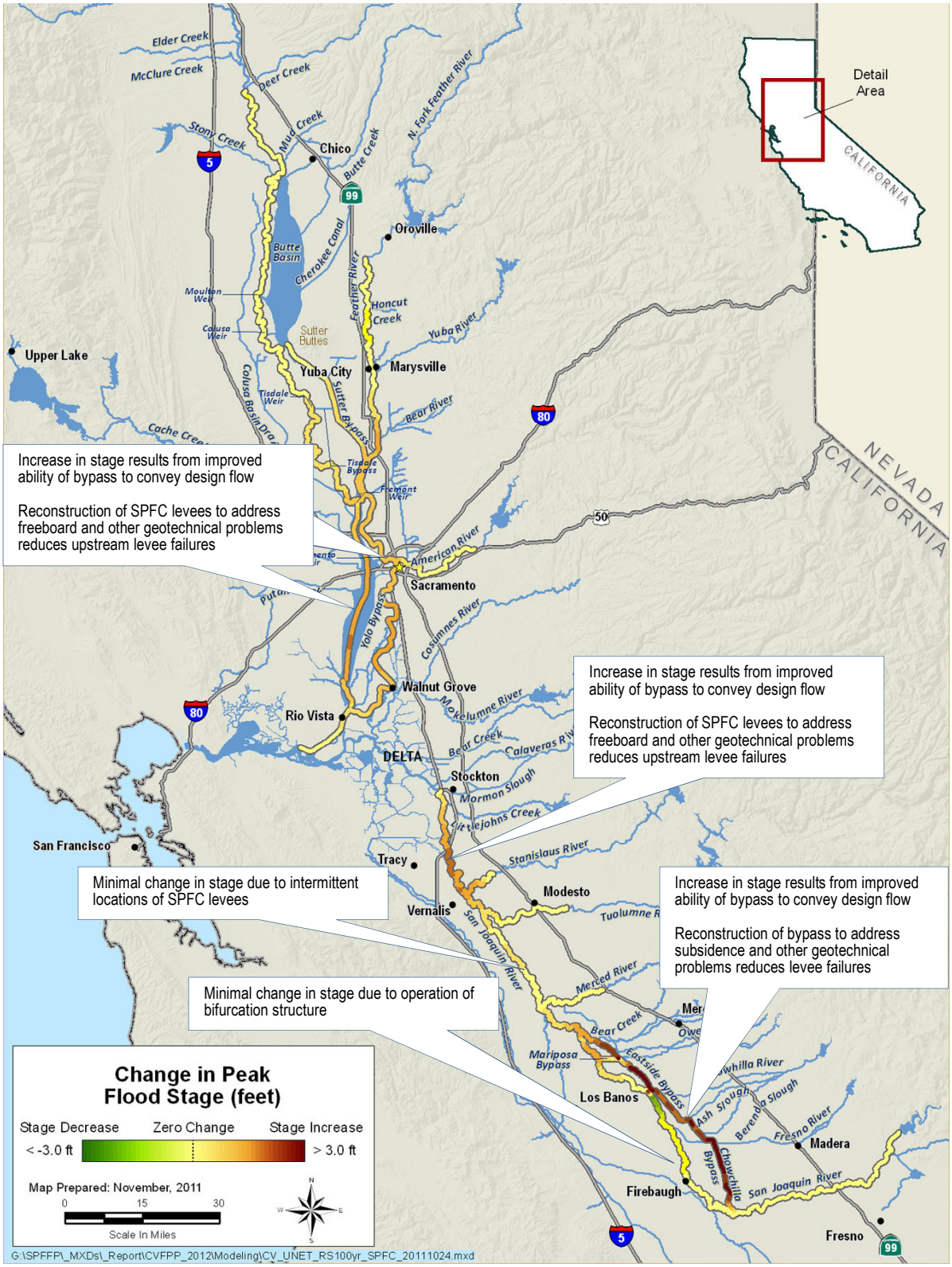


Figure 2-2. Stage Changes from No Project Condition to Achieve SPFC Design Flow Capacity Approach – 1 Percent AEP (100-Year)

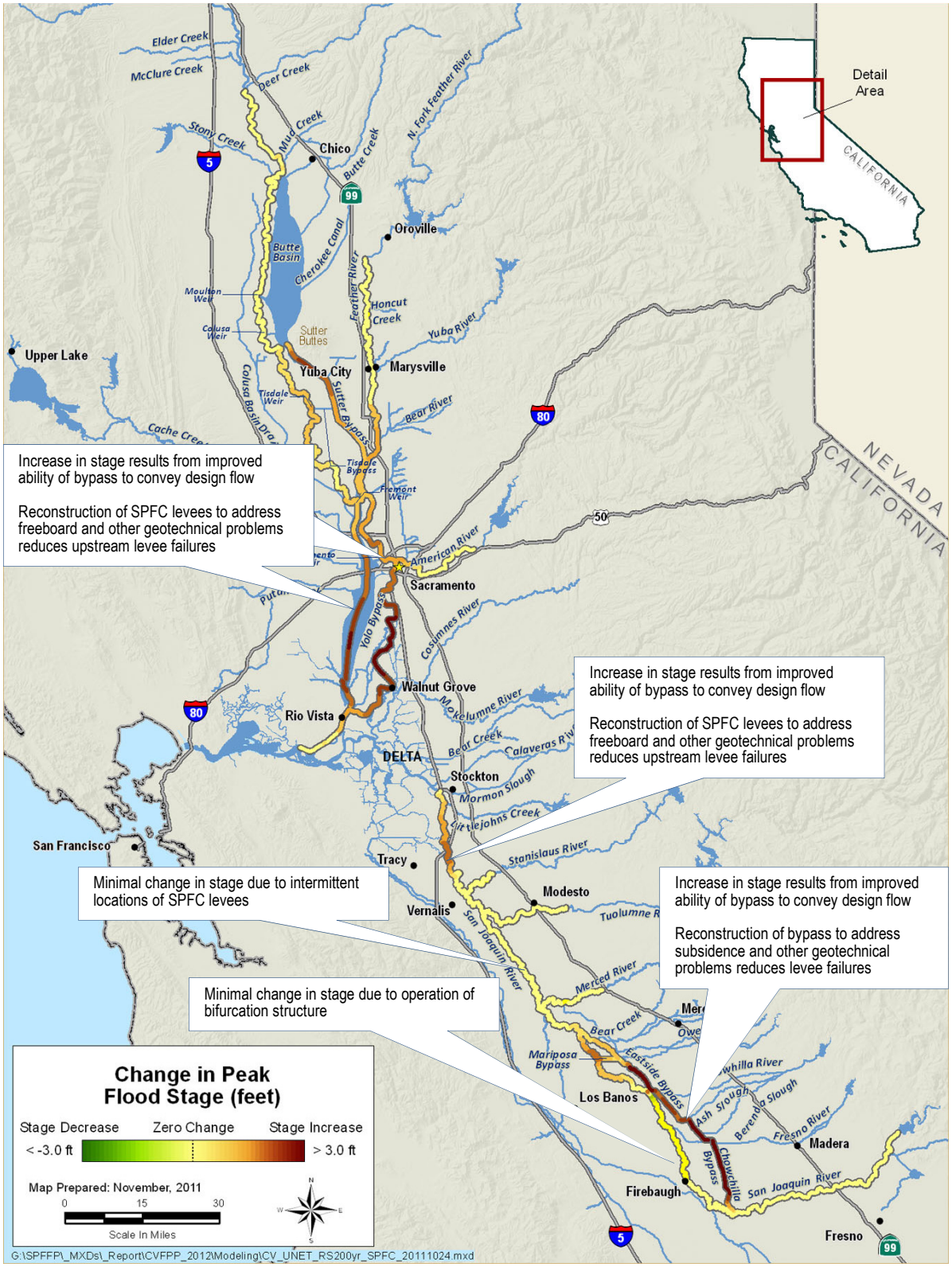


Figure 2-3. Stage Changes from No Project Condition to Achieve SPFC Design Flow Capacity Approach – 0.5 Percent AEP (200-Year)

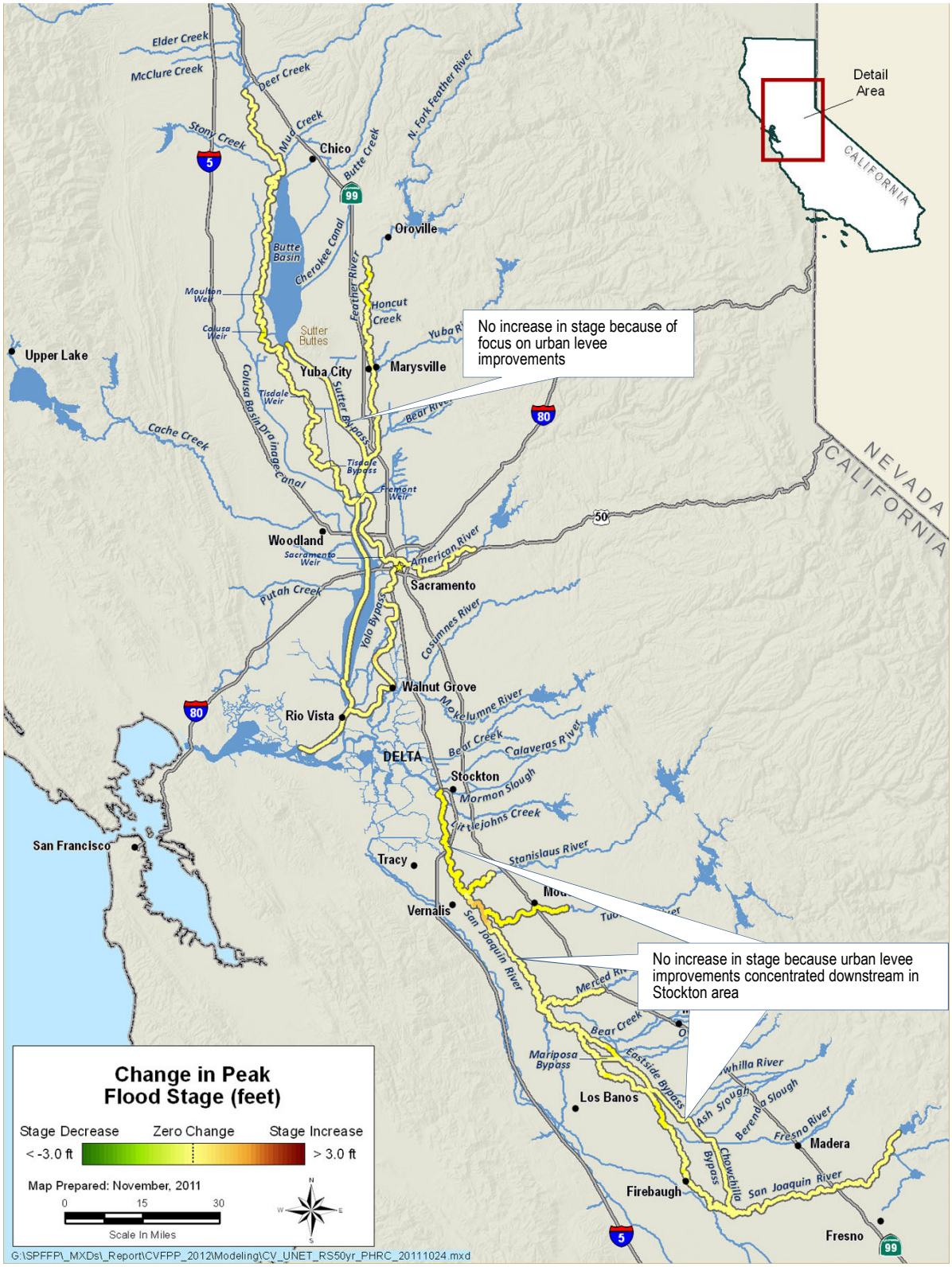


Figure 2-4. Stage Changes from No Project Condition to Protect High Risk Communities Approach – 2 Percent AEP (50-Year)

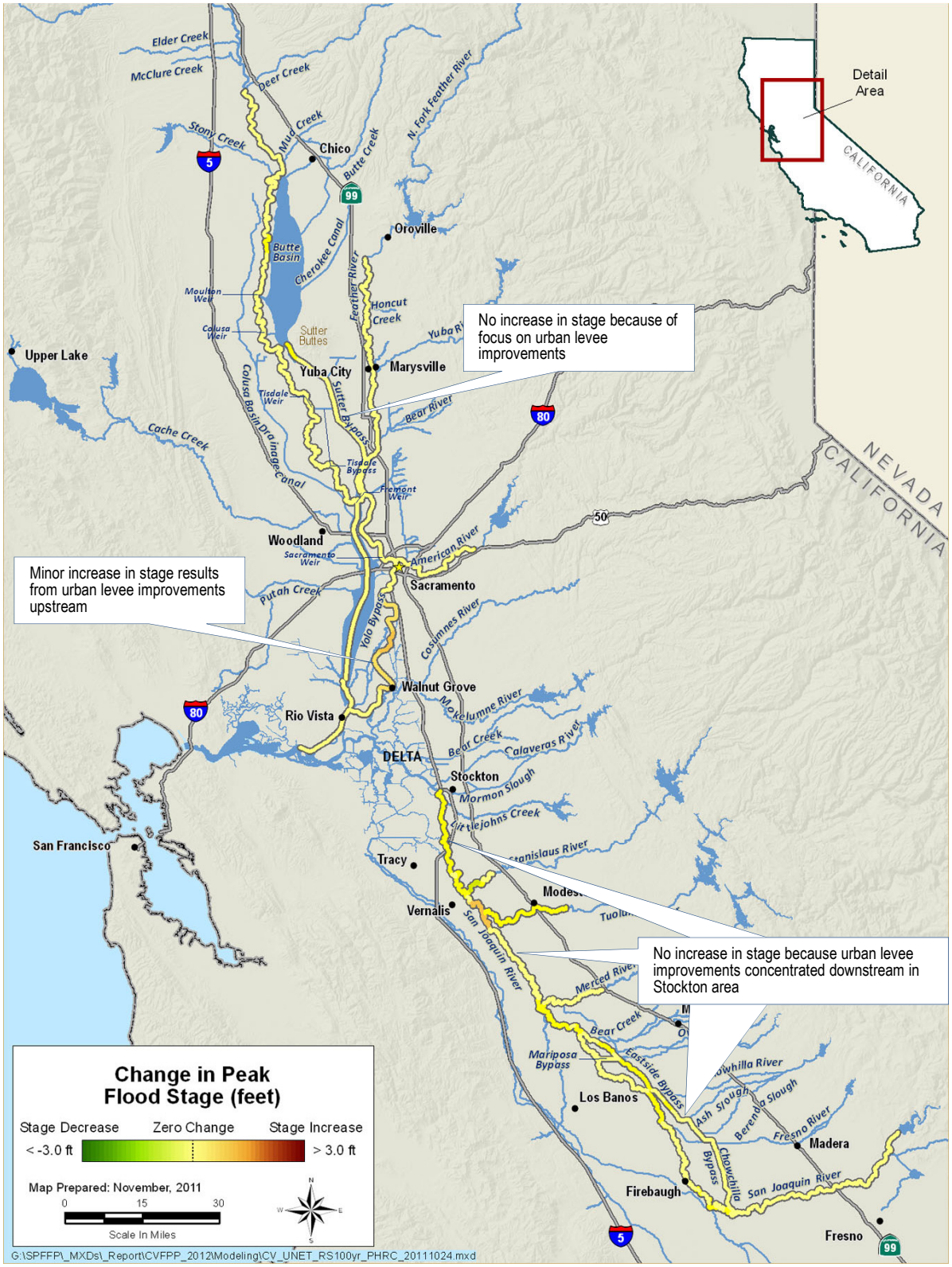


Figure 2-5. Stage Changes from No Project Condition to Protect High Risk Communities Approach – 1 Percent AEP (100-Year)

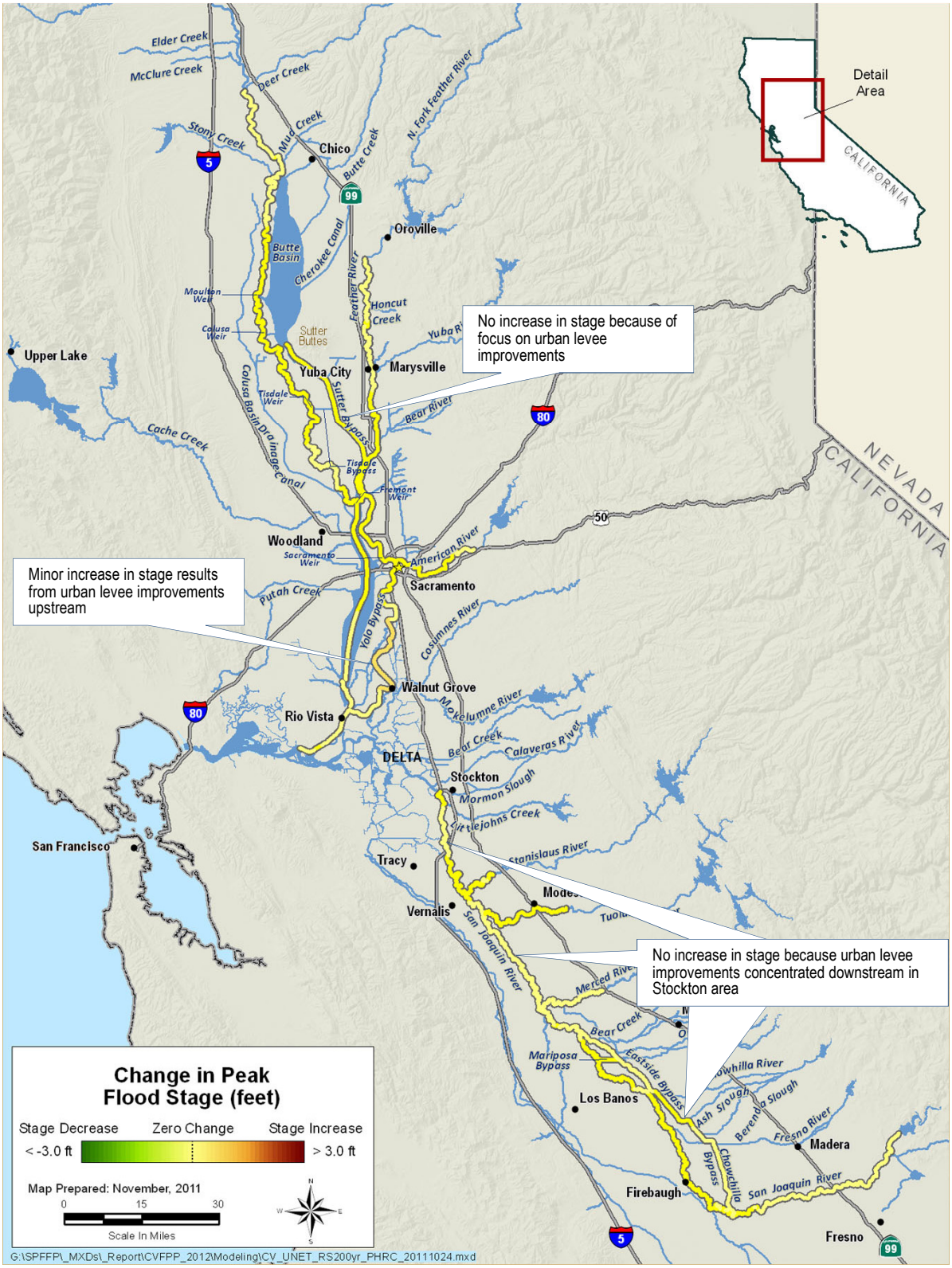


Figure 2-6. Stage Changes from No Project Condition to Protect High Risk Communities Approach – 0.5 Percent AEP (200-Year)

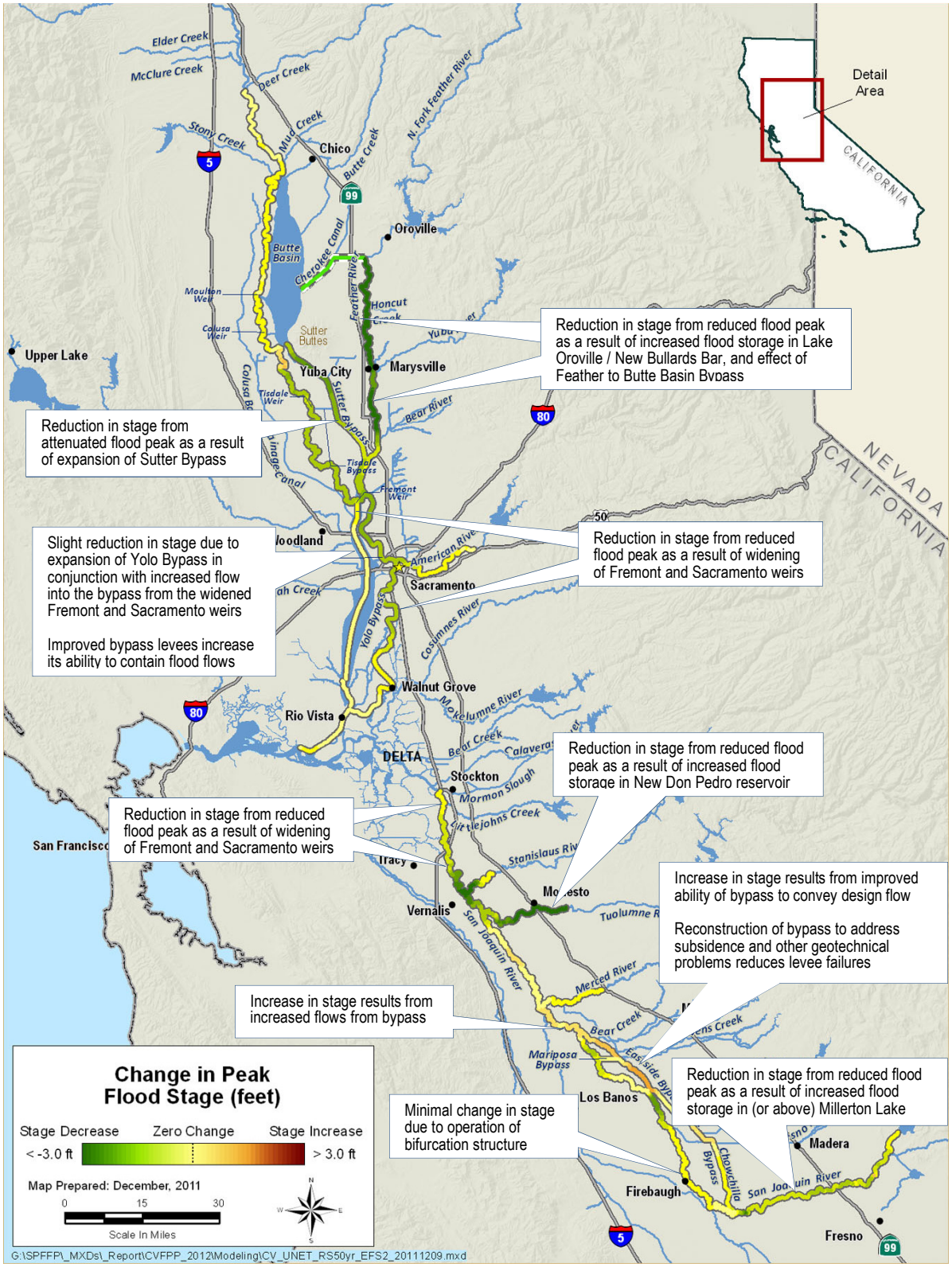


Figure 2-7. Stage Changes from No Project Condition to Enhance Flood System Capacity Approach – 2 percent AEP (50-year)

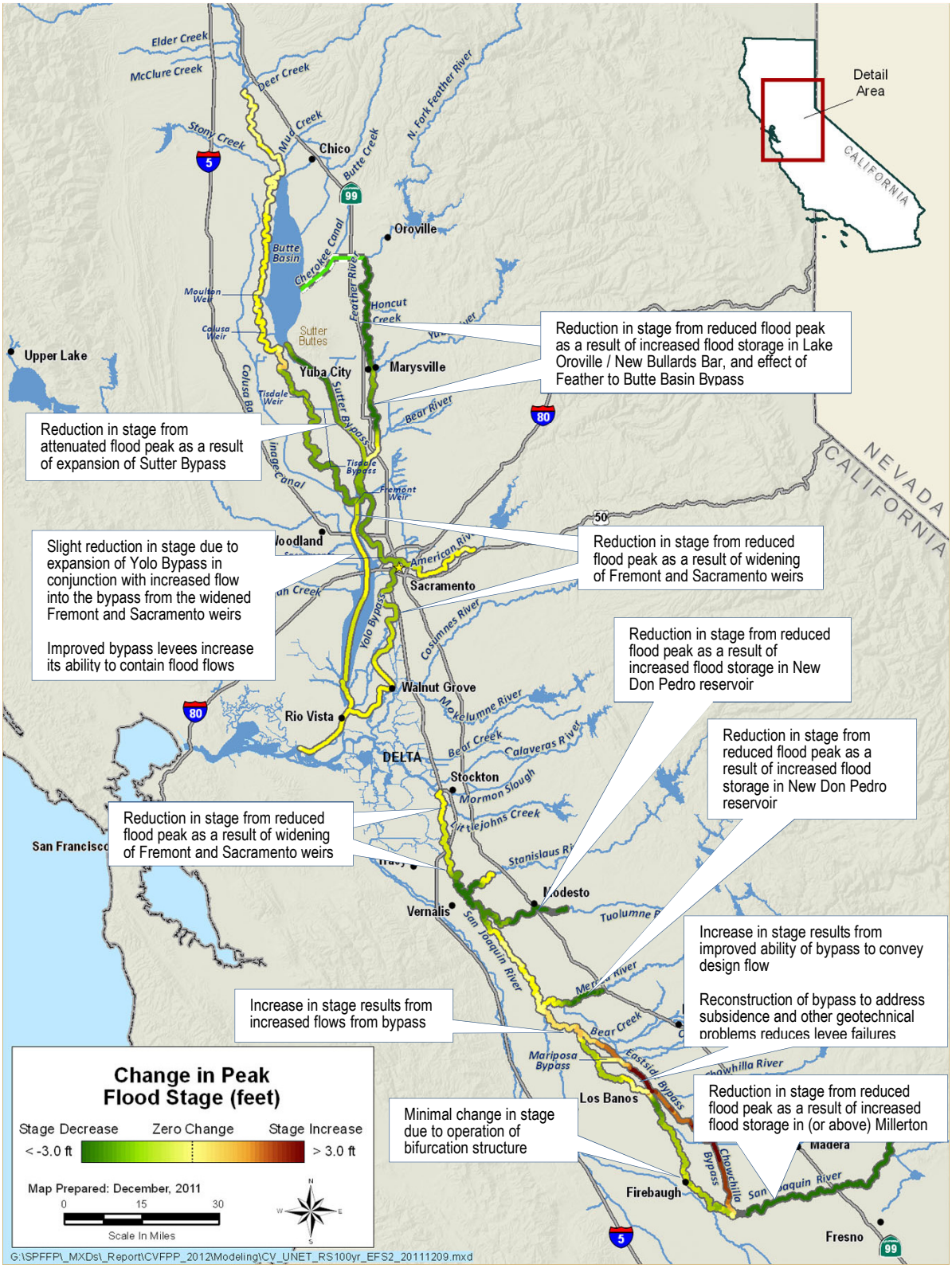


Figure 2-8. Stage Changes from No Project Condition to Enhance Flood System Capacity Approach – 1 Percent AEP (100-Year)

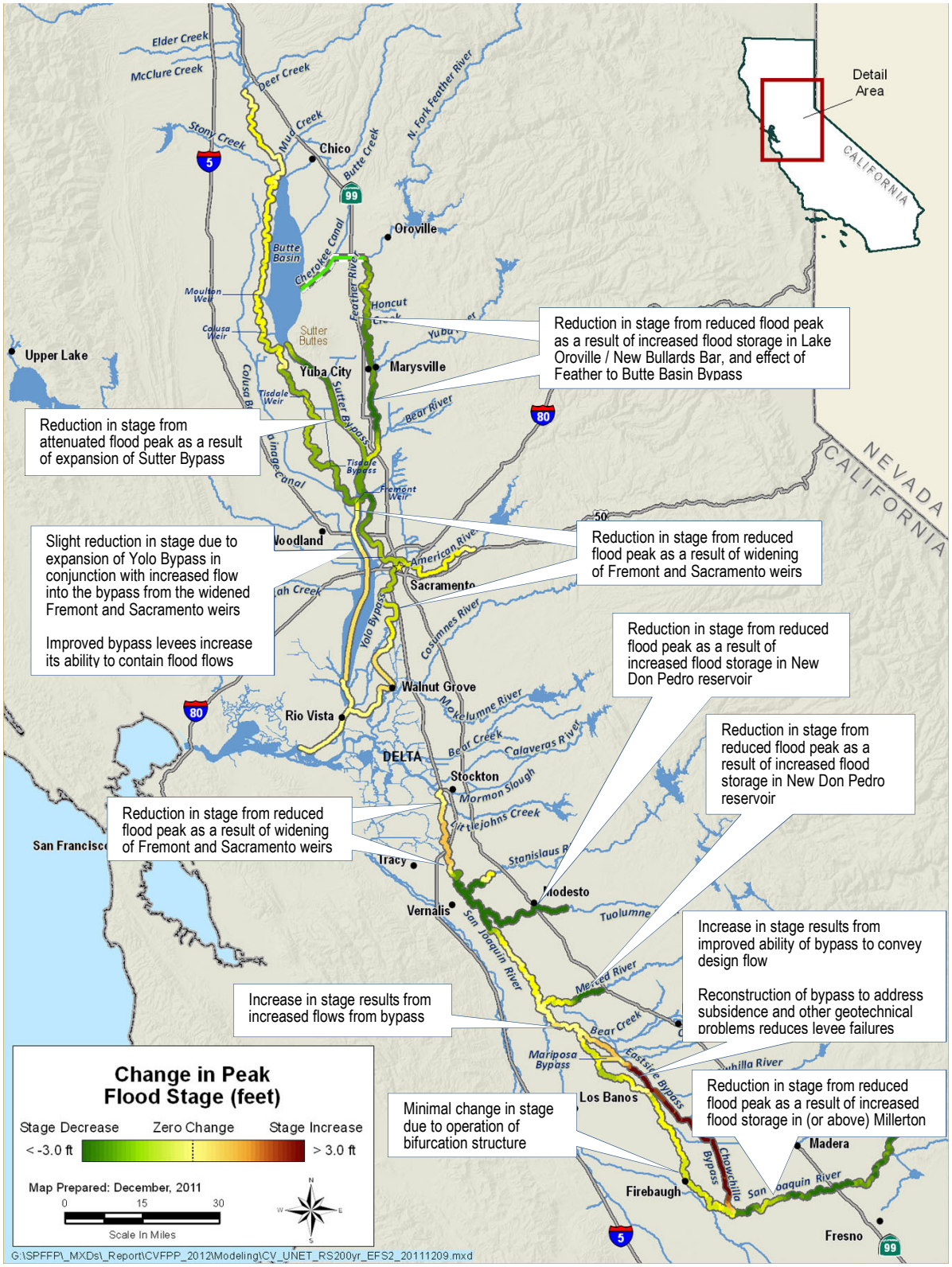


Figure 2-9. Stage Changes from No Project Condition to Enhance Flood System Capacity Approach – 0.5 Percent AEP (200-Year)

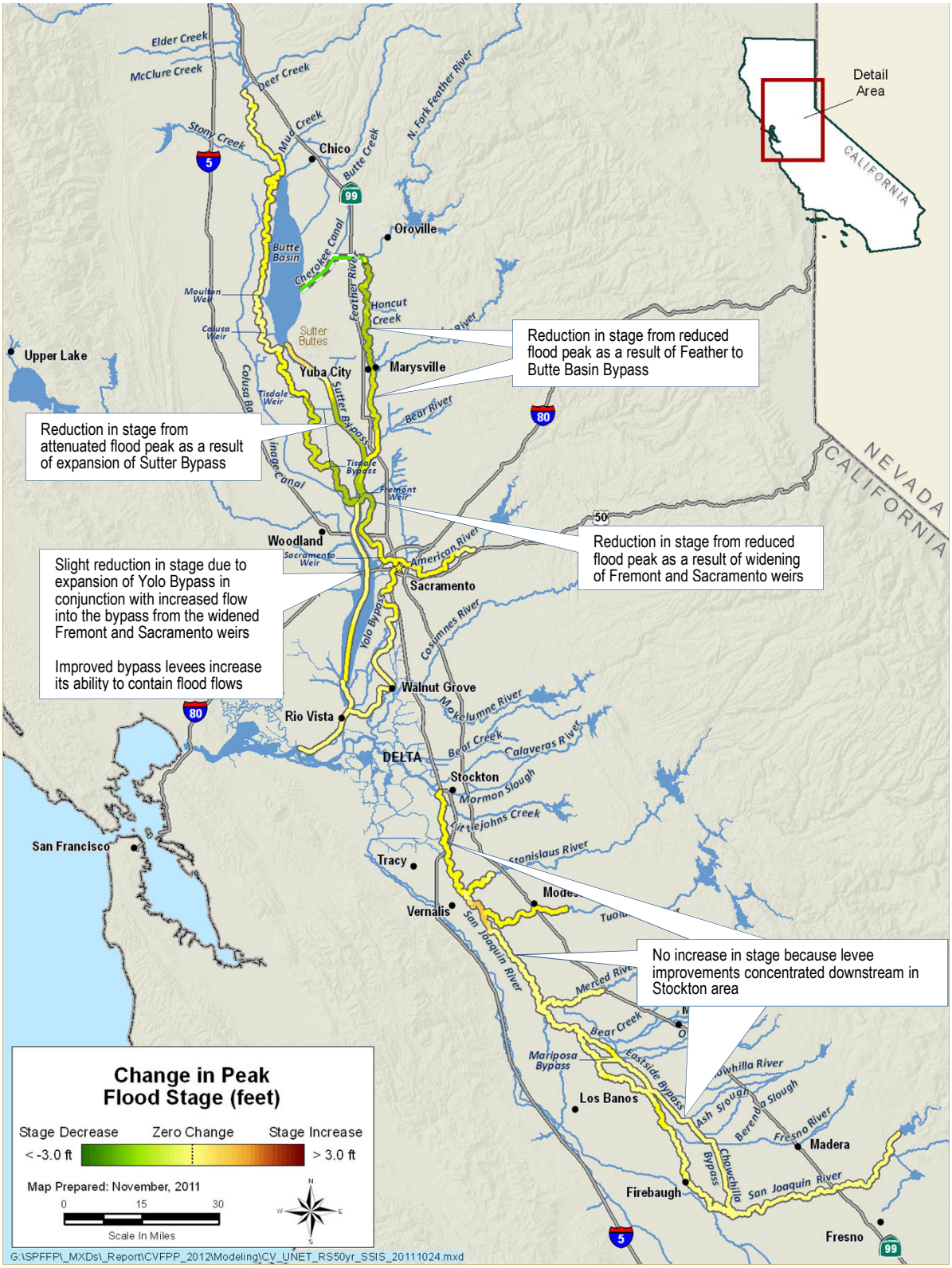


Figure 2-10. Stage Changes from No Project Condition to State Systemwide Investment Approach – 2 Percent AEP (50-Year)

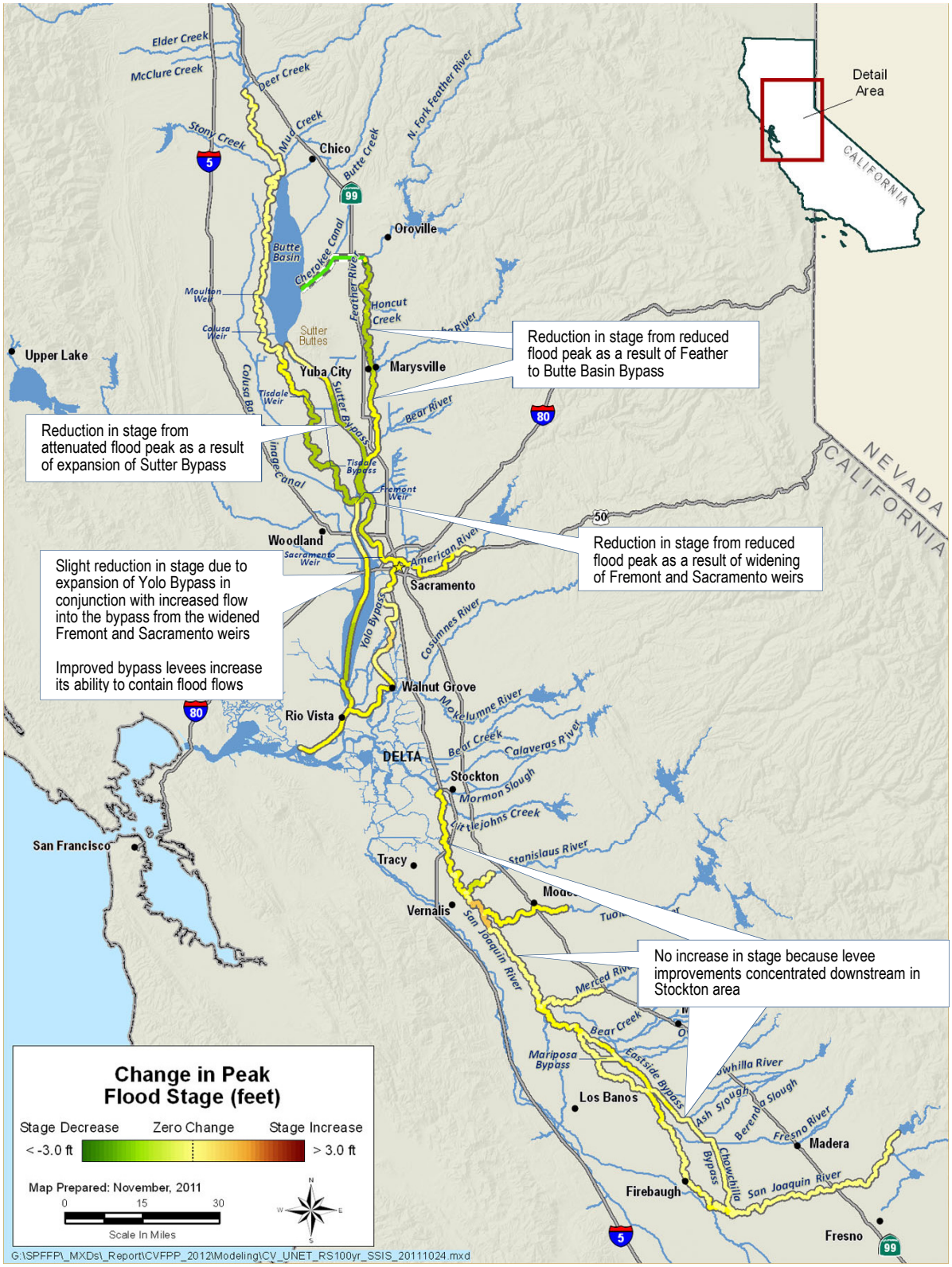


Figure 2-11. Stage Changes from No Project Condition to State Systemwide Investment Approach – 1 Percent AEP (100-Year)

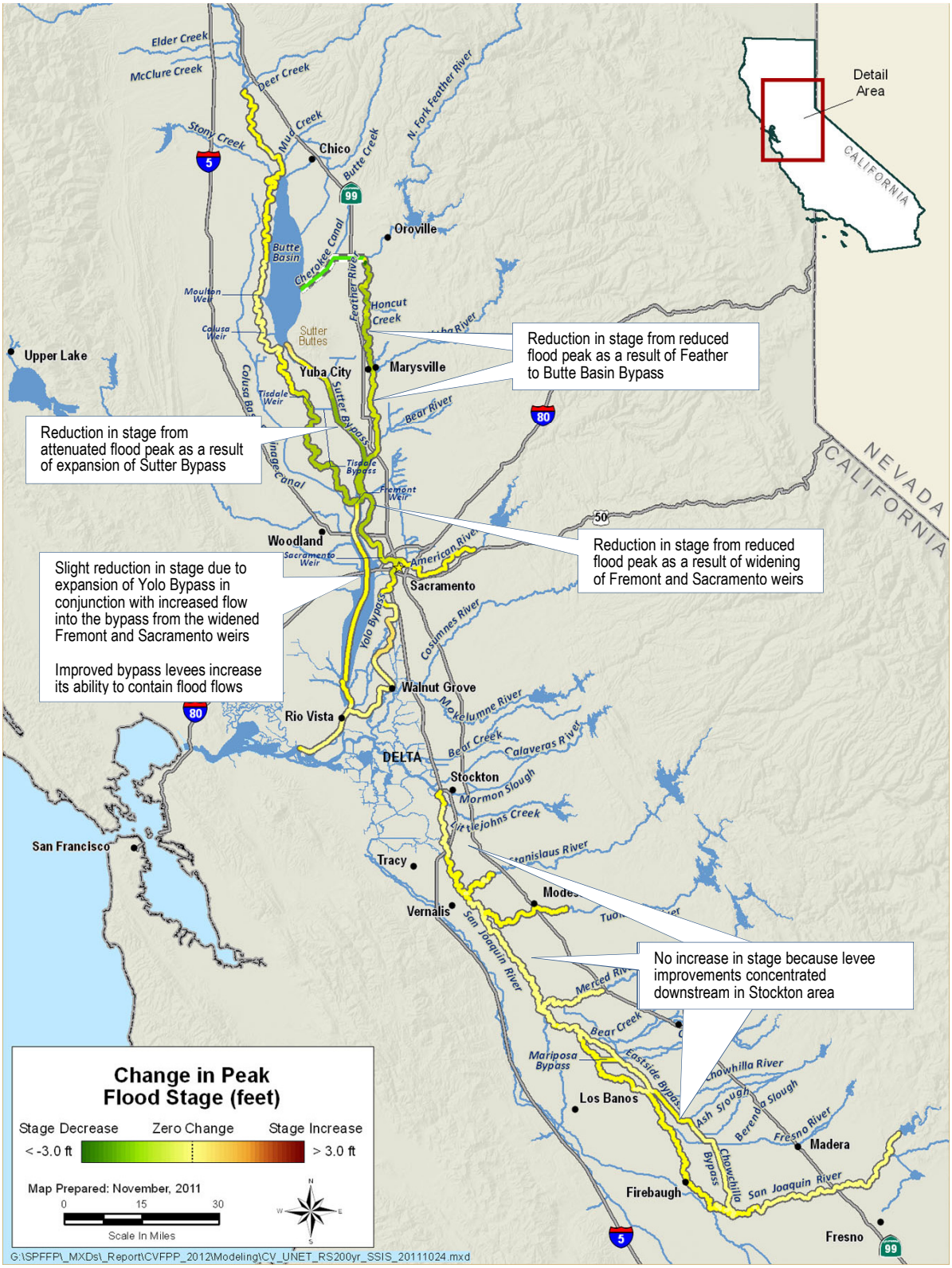


Figure 2-12. Stage Changes from No Project Condition to State Systemwide Investment Approach – 0.5 Percent AEP (200-Year)

3.0 Methodology

This section provides an overview of the 2012 CVFPP riverine modeling framework for the Sacramento and San Joaquin river basins, and discusses model selection, the UNET hydraulic models, levee performance curves, assumptions for the riverine channel evaluation, and modeling assumptions for the No Project condition and each CVFPP approach. Methodology and results for the Stockton area are contained in Section 5.

It is important to note that the hydraulic modeling described below was performed only to support development of the 2012 CVFPP. The modeling is a deterministic process that simulates levee breaches based on data provided regarding levee stability, but it cannot and does not predict the location of actual levee breaches.

3.1 2012 CVFPP Riverine Modeling Overview

Figure 3-1 shows the overall riverine hydraulic modeling schematic for the CVFPP. With defined boundary conditions (including upstream hydrographs to represent storm events, downstream tailwater stage, levee breach scenarios, etc.), riverine hydraulic conditions were simulated to generate hydrographs that would be the upstream boundary conditions for the Delta hydraulic model. The simulated riverine water stages were also used to evaluate flood damage (Attachment 8F: Flood Damage Analysis).

3.2 Model Selection

DWR is developing new riverine hydraulic models through the CVFED Program, but these models were not completed in time to be used for the 2012 CVFPP. Therefore, it was necessary for DWR to use readily available models and data for the CVFPP riverine hydraulic evaluation. Two sets of existing models were considered for the CVFPP riverine hydraulic evaluation: UNET models from the Comprehensive Study[1]

[1] In response to extensive flooding and damage experienced in California in1997, the United States Congress authorized the Sacramento District of the U.S. Army Corps of Engineers (USACE) to provide a comprehensive analysis of the Sacramento and San Joaquin river basin flood management systems and to partner with the State of California to develop master plans for flood management and integrate ecosystem restoration in the Sacramento and San Joaquin river basins (USACE, 2002a).

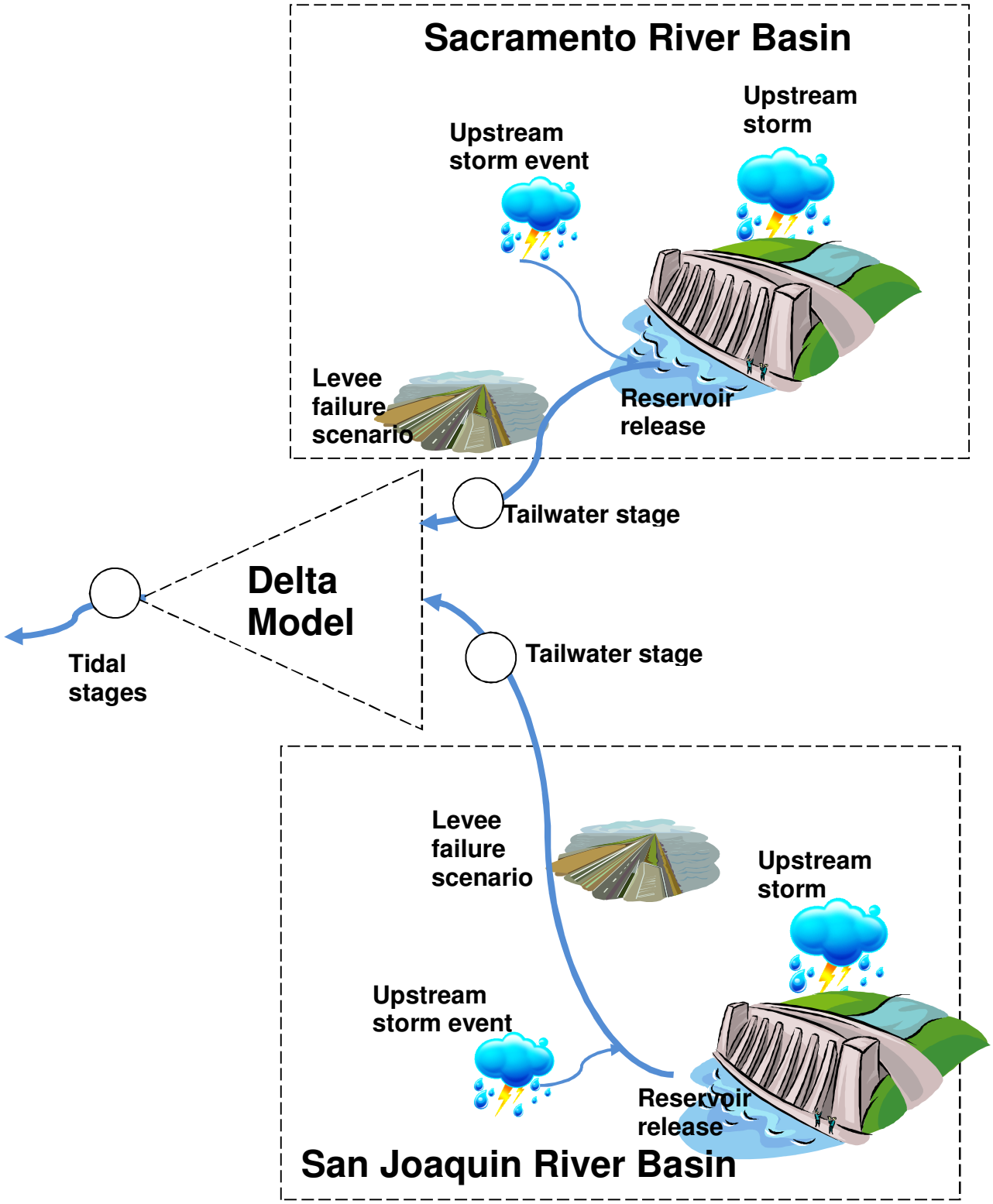


Figure 3-1. Schematic of Central Valley Flood Protection Plan Hydraulic Modeling

(USACE 2002a) and models based on the Hydrologic Engineering Center River Analysis System (HEC-RAS).

3.2.1 Comprehensive Study UNET Models

UNET is a computer model designed to simulate one-dimensional (1-D), fully unsteady flow through a full network of open channels, weirs, bypasses, and storage areas. It is a fixed-bed analysis and does not account for sediment movement, scour, or deposition. UNET assumes no exchange with groundwater and is capable of simulating levee breaks and breaches (USACE, 1997; 2002c).

The authorization for the Comprehensive Study directed the U.S. Army Corps of Engineers (USACE) to develop a UNET application for the Sacramento and San Joaquin river basins to simulate the Sacramento and San Joaquin River Flood Management System and allow basin-wide, systematic evaluation. The August 1998 UNET Version 4.0, with additional modifications made in April 2000, was used for the Comprehensive Study. Separate UNET model data sets were developed for the Sacramento River system and the San Joaquin River system. The Comprehensive Study UNET models incorporated synthetic hydrology floodflows, reservoir operations, and flows in the river systems and major tributaries to evaluate the hydraulic performance of the flood management systems of the two rivers. For a given inflow hydrology, the riverine hydraulic models were used to determine river flow, stage, velocity, and depth, as well as levee breaches and breakout and return flows from overbank areas, allowing the modeler to assess the systemwide performance of a range of flood management modifications under various hydrologic conditions.

3.2.2 HEC-RAS Model

The HEC-RAS software can perform hydraulic calculations for a full network of natural and constructed channels in steady or unsteady mode. The 1-D river analysis components include steady flow, unsteady flow, sediment transport/mobile bed computations, and water quality. (UNET is the predecessor of the unsteady module used in HEC-RAS (USACE, 2010)). Unlike UNET, HEC-RAS has a graphical user interface and advanced capabilities for data input and output.

HEC-RAS has been applied in the Sacramento River Basin through multiple individual evaluations focusing on localized projects, instead of basin-wide effects. The USACE Sacramento District has converted the Comprehensive Study UNET model for the Sacramento River Basin into the HEC-RAS platform (USACE, 2009). The two models (UNET and HEC-RAS) have almost the same study area, except that the HEC-RAS

model has no coverage in the Butte Basin, the Sacramento River north of Colusa, Colusa Basin Drain, Natomas Cross and Natomas East Main Drainage canals and tributaries (USACE, 2008).

For the San Joaquin River Basin, the conversion from the Comprehensive Study UNET model into the HEC-RAS platform was completed in February 2010 (DWR, 2009). Results from the San Joaquin HEC-RAS model using Comprehensive Study hydrology, however, were different from the results of the accepted Comprehensive Study UNET model.

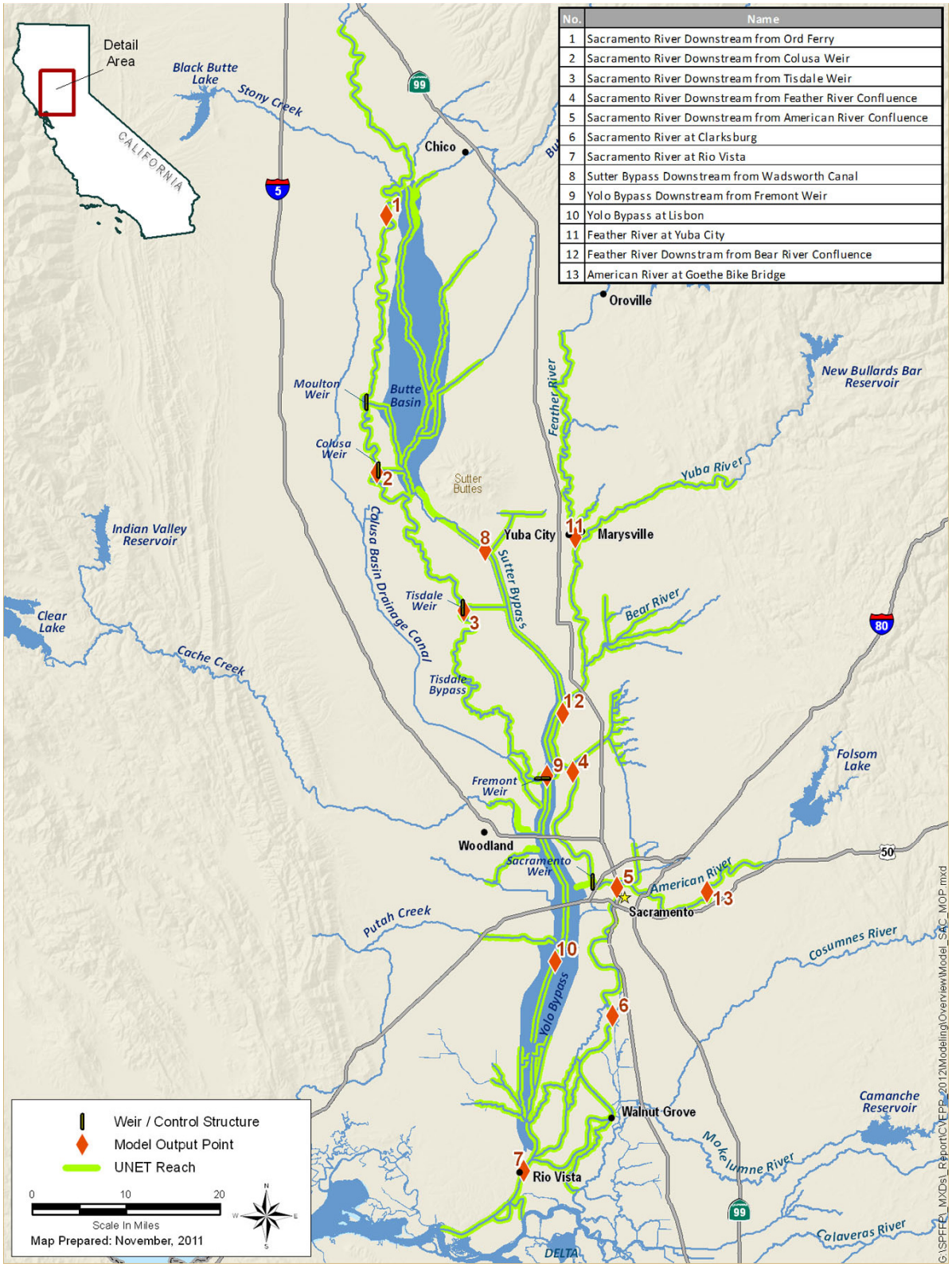
3.2.3 Model Selection for 2012 CVFPP

The HEC-RAS and UNET models for the Sacramento and San Joaquin river basins use the same Central Valley hydrology. As previously described, HEC-RAS has more user-friendly functions, such as a graphical user interface, and multiple input and output options that are not available in UNET. However, coverage for the Sacramento River Basin in the existing UNET model is more extensive than in the available HEC-RAS model. Because this more extensive modeling coverage is important to the systemwide planning effort, the Comprehensive Study UNET model for the Sacramento River Basin was selected as the base riverine hydraulic model for 2012 CVFPP hydraulic model development. To be consistent with the Sacramento River Basin, the Comprehensive Study UNET model for the San Joaquin River Basin was also selected to be the base riverine model for 2012 CVFPP development.

3.3 CVFPP UNET Model Overview

The two Comprehensive Study UNET models, one for the Sacramento River Basin and one for the San Joaquin River Basin, provided a means for understanding and representing channel hydraulics in the two river systems for development of the 2012 CVFPP. Modifications were made for the CVFPP application, and these two modified models for the CVFPP are referred to in this attachment as the Sacramento UNET Model and San Joaquin UNET Model.

As described previously, the Sacramento and San Joaquin UNET models were used to determine river stage, velocity, depth, and levee breaches, as well as breakout and return flows from overbank areas for each CVFPP approach. Extensive topographic data were collected and assembled to develop digital river alignments and cross sections by USACE as part of the Comprehensive Study effort. UNET modeling coverage and output data locations for the Sacramento River Basin and the San Joaquin River Basin are shown in Figures 3-2 and 3-3, respectively. Assumptions for all CVFPP approaches are described in detail in the following sections.



No.	Name
1	Sacramento River Downstream from Ord Ferry
2	Sacramento River Downstream from Colusa Weir
3	Sacramento River Downstream from Tisdale Weir
4	Sacramento River Downstream from Feather River Confluence
5	Sacramento River Downstream from American River Confluence
6	Sacramento River at Clarksburg
7	Sacramento River at Rio Vista
8	Sutter Bypass Downstream from Wadsworth Canal
9	Yolo Bypass Downstream from Fremont Weir
10	Yolo Bypass at Lisbon
11	Feather River at Yuba City
12	Feather River Downstram from Bear River Confluence
13	American River at Goethe Bike Bridge

Figure 3-2. UNET Coverage in Sacramento River Basin

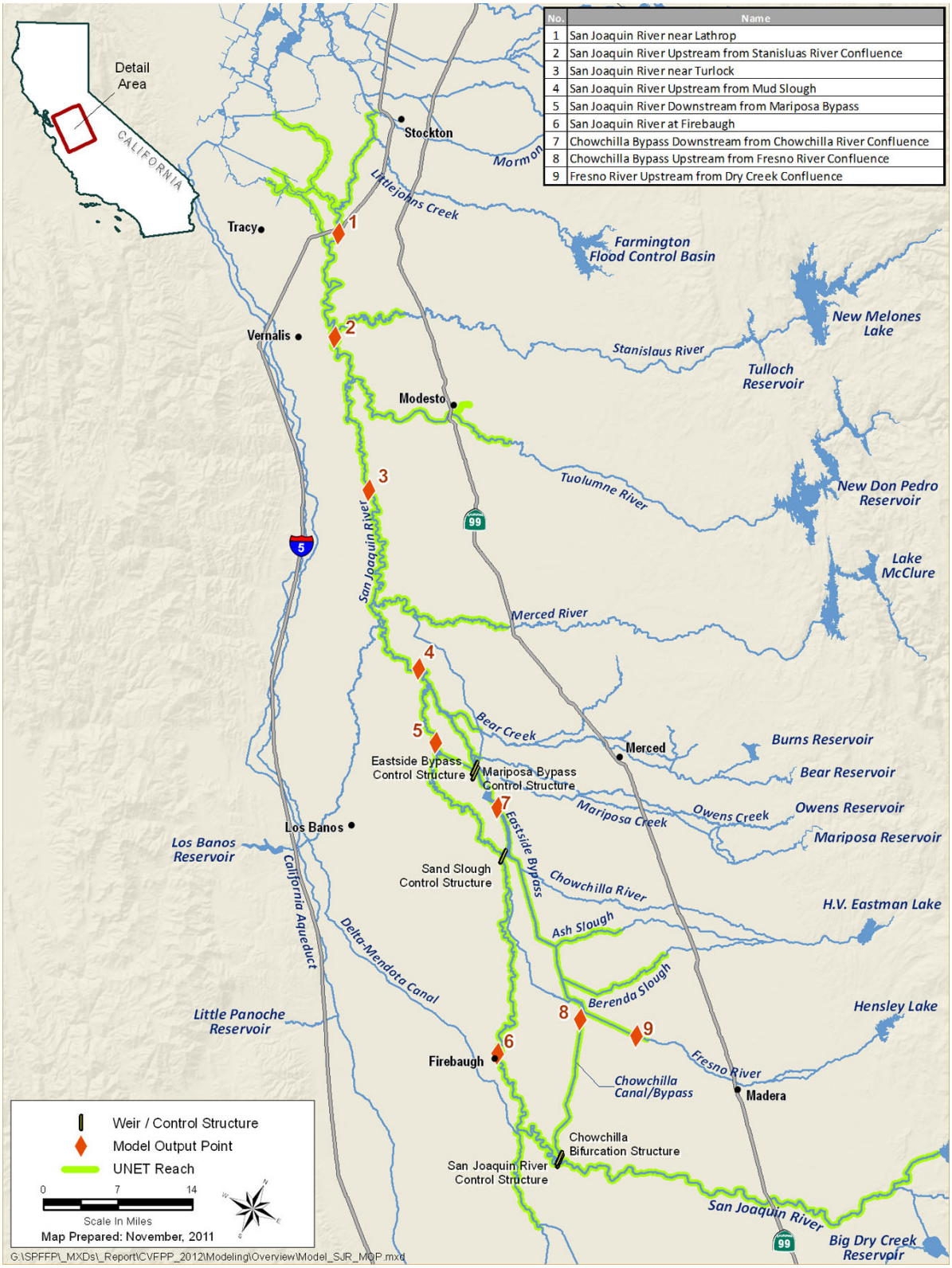


Figure 3-3. UNET Coverage in San Joaquin River Basin

3.4 Levee Performance Curves for CVFPP

The Urban Levee Evaluations (ULE) Project and the Non-Urban Levee Evaluations (NULE) Project under the DWR Levee Evaluations Program developed performance curves for levees in the Sacramento River and San Joaquin River basins. Levee performance curves provide geotechnical relationships between river stage and the probability that a levee segment will breach (water from the water side of the levee flows in an uncontrolled manner to the landside of the levee) at that stage. Details on levee performance curve development are contained in Attachment 8E: System/Levee Performance.

Levee performance curves from ULE and NULE were used to identify two water surface elevations of interest for the hydraulic analyses. These water surface elevations, and the corresponding probability of levee failure at a particular levee location are as follows:

- **Probable failure point (PFP)** – 85 percent probability of failure
- **Top of levee (TOL)** – 100 percent probability of failure

These two water surface elevations were incorporated into UNET models to simulate conditional levee failure, meaning that once the simulated river stage at a specific levee location reaches the specified breach elevation (PFP or TOL depending on the CVFPP approach being modeled), a levee breach would begin to develop in UNET. Water from the river would then enter into the adjacent floodplain through the levee breach, and the downstream river stage and flow would be reduced. Because the PFP is always lower than the top of the levee, the breach would begin to form at below the TOL. On the other hand, if a TOL breach elevation is used in the simulation, the water surface elevation and flow would be higher than with the PFP before the levee breach, because the TOL is always higher than the PFP.

The water surface elevations of interest described above are not intended to represent or predict how levees would fail under an actual flood event. For example, under the PFP scenario, all levees assigned a PFP would fail in a simulated flood event once the water surface was equal to or higher than the PFP elevation. In reality, many of these levees would not fail even when the stage exceeds the PFP elevations, while others might fail before the stage reaches the PFP elevations. Further, floodfighting and other emergency actions (conditions that are not simulated in the hydraulic models) could result in very different levee failure probabilities.

In addition to simulating the PFP and TOL scenarios, model simulations were also conducted that considered very tall levees along the river channels. These "infinite levee" simulations helped determine the maximum possible floodflows at various locations in the Sacramento and San Joaquin river flood management system.

3.5 Model Assumptions: No Project

Tables 3-1 and 3-2 show modeling assumptions for the Sacramento and San Joaquin river basins, respectively, for the No Project condition and each of the CVFPP approaches. The following sections describe assumptions for the Sacramento and San Joaquin UNET models for the No Project condition.

3.5.1 Sacramento UNET Model

Simulation Period

The simulation period for the Sacramento UNET model is from 9:00 a.m., January 6, to 9:00 a.m., January 29. Peak flows for all flood events occur in the simulation between January 18 and 20.

Upstream Boundary Conditions

Upstream boundary conditions for the Sacramento River UNET model are flow hydrographs (i.e., discharge in cfs over time) for each particular flood at the upstream boundary of all reaches that are not connected to another reach at their upstream end.

Each set of hydrographs represents either unregulated flows (no reservoir upstream) or regulated flows (reservoir releases simulated by reservoir models) under different storm centerings. A centering is a set of synthetic floods for a range of AEP that would result in peak flows at a given location (see Attachment 8A: Hydrology for details). The CVFPP followed the composite floodplain methodology used in the Comprehensive Study (USACE, 2002b) to define the maximum extent of inundation at all locations for a flood of any given AEP. As described in Attachment 8A: Hydrology, five storm centerings were used for the Sacramento River Basin: three mainstem centerings (Ord Ferry, Sacramento, and Shasta) and two tributary centerings (Yuba River, and American River). Each storm center had six flood events, with AEPs of 10, 4, 2, 1, 0.5, and 0.2 percent, corresponding to 10-, 25-, 50-, 100-, 200-, and 500-year return periods.

Table 3-1. Summary of Sacramento River Basin Modeling Assumptions

Element	Description	No Project (NPRJ)	Achieve SPFC Design Flow Capacity (SPFC)	Protect High Risk Communities (PHRC)	Enhance Flood System Capacity (EFSC)	State Systemwide Investment (SSIA)
Levee Setback	Sacramento River RM 199.5 to 197				√	
	Sacramento River RM 169.5 to 111.25				√	
	Feather River RM 24.5 to 0				√	
Levee Improvement	Restore 1955/1957 design levee: Assume levee breach at top of levee in hydraulic model		√		√	
	Fix urban area levee: Assume levee breach at top of levee in hydraulic model			√	√	√
	TRLIA levee improvement	√	√	√	√	√
	Marysville levee improvement	√	√	√	√	√
	Natomas levee improvement	√	√	√	√	√
Bypass	Widen Yolo Bypass[1] & lengthen Fremont Weir				√	√
	Widen Sacramento Bypass and Gates				√	
	Widen Sutter Bypass				√	√
	Feather to Butte Basin (Biggs) Bypass				√	√
Reservoir Storage and Operations	Folsom Dam Joint Federal Project	√	√	√	√	√
	Lake Oroville: Modify Lake Oroville release schedule				√	
	New Bullards Bar and Lake Oroville: Implement coordinated operation of the Feather-Yuba River Basin	√	√	√	√	√
Floodplain Storage	Sutter Butte Basin				√	
	Feather River Basin				√	
	Elkhorn				√	
	Merritt Island				√	

Notes:
55/57 levee design profile was the design standard for the State Plan of Flood Control.
[1] Use off-stream storage to model levee setback.

Key:
EFSC = Enhance Flood System Capacity Approach
NPRJ = No project
PHRC = Protect High Risk Communities Approach
RM = River Mile
SPFC = State Plan of Flood Control
SSIA = State Systemwide Investment Approach
TRLIA = Three Rivers Levee Improvement Authority

Table 3-2. Summary of San Joaquin River Basin Modeling Assumptions

Element	Description	No Project (NPRJ)	Achieve SPFC Design Flow Capacity (SPFC)	Protect High Risk Communities (PHRC)	Enhance Flood System Capacity (EFSC)	State Systemwide Investment (SSIA)
Levee Setback	SJR RM115 to 99				√	
	SJR RM 81.5 to 72.5				√	
Levee Improvement	Restore 55/57 levee design profile: Assume levees breach at top of levee in hydraulic model		√		√	
	Fix urban area levees: Assume levees breach at top of levee in hydraulic model			√	√	√
	Restore bypass levees: Assume levees breach at top of levee in hydraulic model		√		√	
Bypass	Widen Paradise Cut				√	√
Reservoir Storage and Operations	New Don Pedro Reservoir: Increase flood storage allocation by 230,000 acre-feet				√	
	Friant Dam and Millerton Lake: Increase flood storage allocation by 60,000 acre-feet				√	
	New Exchequer Dam and Lake: Increase flood storage allocation by 100,000 acre-feet				√	
Floodplain Storage	Roberts Island				√	
	San Joaquin River: between Merced and Tuolumne rivers				√	
	San Joaquin River: between Tuolumne River and Stanislaus River				√	

Note:
55/57 levee design profile was the design standard for the State Plan of Flood Control

Key:
EFSC = Enhance Flood System Capacity Approach
NPRJ = No project
PHRC = Protect High Risk Communities Approach
RM = river mile
SJR = San Joaquin River
SPFC = State Plan of Flood Control
SSIA = State Systemwide Investment Approach
TRLIA = Three Rivers Levee Improvement Authority

Frequent flows, with an AEP of greater than 10 percent (e.g., return period less than 10-year), were not modeled because the Sacramento River flood management systems can handle at a minimum floods that have AEPs of 4 percent or greater (25-year or less return period). Therefore, it is anticipated that storms with greater than a 10 percent AEP would not cause serious economic impacts.

Interior Boundary Conditions

Interior boundary conditions define the connections between stream reaches, as well as between stream reaches and other parts of the model. The UNET model uses flow and stage continuity to control normal reach connections. Extensive topographic data were collected and incorporated during Comprehensive Study model development to represent river channel alignment, cross sections, and bridge geometries in the UNET model (USACE, 2002c). During the model development process for the 2012 CVFPP, updates were made to cross sections in the Tisdale and Yolo bypasses to reflect excavation work completed on those two areas after the Comprehensive Study (DWR, 2006a and 2006b).

Downstream Boundary Conditions

To function properly, a hydraulic model of a river system must define the water surface elevation at the downstream end of all model reaches not connected to another reach or river. Downstream boundary conditions are usually in the form of tailwater stage hydrographs that describe the variation of the downstream water surface elevation over time.

The downstream boundaries for the Sacramento River hydraulic model are in the Delta, and, as a result, represent tailwater conditions under tidal and estuary influences. Tailwater hydrographs for the Sacramento River hydraulic model include the Sacramento River at Collinsville and the downstream ends of Three-Mile and Georgiana sloughs. The tailwater hydrographs were developed from information gathered at tide gages during the 1997 flood, which represents conservative or high tailwater conditions.

Internal Boundary Conditions

As mentioned, interior boundary conditions define the connections between stream reaches, and between stream reaches and other parts of the model. Internal boundary conditions, however, are placed in the model to represent levee failure scenarios or storage interactions, spillways or weir overflow/diversion structures, bridge or culvert hydraulics, or pumped diversions.

Operation rules for weirs are embedded in the model. For example, the Colusa Weir was modeled as an uncontrolled lateral spillway 1,736 feet long that begins spilling at a river elevation of 58.89 feet. As another example, the Sacramento Weir was modeled as a controlled lateral spillway. All 48 gates on the weir were modeled in groups of 8. Each group of eight gates is 300 feet wide and was explicitly named so that it can be referenced in the boundary conditions for a time series of gate openings.

Levee Breach Modeling

The Sacramento UNET model, for the No Project condition, simulates levee breaches using the simple levee failure option; once the water surface elevation at a levee breach location reaches the PFP elevation, the levee breaches and allows water to flow from the channel to the attached storage (floodplain) area, consequently reducing the stage at the breach and the flow downstream in the channel. Levee breach locations and elevations were from levee performance curves developed from data from the ULE and NULE projects.

The simple levee failure option used in the Sacramento UNET model applies a simple storage connection concept in which the flow through a breach is computed by multiplying the volume of available storage by a coefficient. Because information on the size and evolution of breaches in levee systems is limited, and detailed levee breach information is often not available, modeling of embankment failures is not practical. The UNET simple linear storage algorithm acknowledges this lack of data and applies a simple concept for filling a storage area behind a levee. Flow into the storage area behind the levee is assumed to be proportional to the available storage (i.e., flow through a breach is greatest at the start of the levee breach and decreases as the leveed area fills). This procedure also has a computational advantage in that it is stable and would function with larger time steps.

3.5.2 San Joaquin UNET Model

Simulation Period

The simulation period of the San Joaquin River Basin UNET model is from 10:00 a.m., January 15, through 12 a.m., February 3. Peak flows for all flood events occur in the simulation between January 18 and 20.

Upstream Boundary Conditions

Upstream boundary conditions for the San Joaquin River UNET model are flow hydrographs (i.e., discharge in cfs over time) for each particular flood at the upstream boundary of all reaches that are not connected to another reach at their upstream end.

The use of regulated and unregulated hydrographs in the San Joaquin River UNET model are the same as described for the Sacramento River UNET model in Section 3.5.1. As described in Attachment 8A: Hydrology, upstream boundary conditions for the San Joaquin River Basin are hydrographs from five storm centerings: three mainstem centerings (El Nido, Newman, and Vernalis) and two tributary centerings (Friant Dam and Merced River). Each storm centering had six flood events, with AEPs of 10, 4, 2, 1, 0.5, and 0.2 percent, corresponding to 10-, 25-, 50-, 100-, 200-, and 500-year return periods.

Frequent flows with an AEP of greater than 10 percent (e.g., return period less than 10-year) were not modeled because the San Joaquin River flood management system can handle at a minimum flood events that have AEPs of 10 percent or greater (10-year or less return period). Therefore, it is anticipated that storms with greater than a 10 percent AEP (e.g., return period of less than 10-years) would not cause serious economic impacts.

Interior Boundary Conditions

Interior boundary conditions define the connections between stream reaches, as well as between stream reaches and other parts of the model. The UNET model uses flow and stage continuity to control normal reach connections. Extensive topographic data were collected and incorporated during Comprehensive Study model development to represent river channel alignment, cross sections, and bridge geometries in the UNET Model (USACE, 2002c).

Downstream Boundary Conditions

The downstream boundaries for the San Joaquin River hydraulic model are in the Delta, and, as a result, represent tailwater conditions under tidal and estuary influences. The four tailwater hydrographs for the San Joaquin River are (1) Grant Line Canal at Tracy Boulevard, (2) Middle River at Highway 4, (3) Old River at Tracy Boulevard, and (4) the San Joaquin River at the Stockton Deep Water Ship channel. The tailwater hydrographs were developed from information gathered at tide gages during the 1997 flood, which represents conservative or high tailwater conditions.

Internal Boundary Conditions

Operation rules for weirs are embedded in the model. For example, the bifurcation/diversion structure from the San Joaquin River to the Eastside/Chowchilla Bypass was modeled to control the upstream water surface in the San Joaquin River to an elevation of 172.5 feet NGVD29 using a rating table that divides the flows between the San Joaquin River and the Eastside/Chowchilla Bypass. The model also assumes that 12,500 cfs is the largest flow that would reach the bifurcation structure because higher flows would cause upstream levee breaches. The bifurcation/

diversion structure from the Eastside/Chowchilla Bypass to the Mariposa Bypass and Deep Slough was modeled in the same manner, with the upstream pool elevation held to an elevation of 97 feet NGVD29 and flows divided between the Mariposa Bypass and Deep Slough. Flows in excess of 30,000 cfs were assumed to overtop the control structure and surrounding levees.

Levee Breach Modeling

Similar to the Sacramento River Basin, the San Joaquin River Basin UNET model, for the No Project condition, uses the simple levee failure option to simulate levee breaches when water surface elevation at a specific levee breach location reaches the PFP elevation. Levee breach locations and elevations were from levee performance curves developed from data from the ULE and NULE projects.

3.6 Model Assumptions: Achieve SPFC Design Flow Capacity Approach

This approach focuses on improving existing SPFC facilities so that they can convey their design flows with a high degree of reliability based on current engineering criteria. Levee improvements would be made to SPFC levees regardless of the areas they protect.

Tables 3-1 and 3-2 show modeling assumptions for the Sacramento and San Joaquin river basins, respectively, for the No Project condition and each of the CVFPP approaches. The following sections describe specific assumptions for the Sacramento and San Joaquin UNET models for the Achieve SPFC Design Flow Capacity Approach.

3.6.1 Sacramento UNET Model

Upstream, Downstream, Interior, and Internal Boundary Conditions

Upstream, downstream, interior, and internal boundary conditions were unchanged from the No Project condition.

Levee Breach Modeling

For the Achieve SPFC Design Flow Capacity Approach, the breach elevation at each levee breach point on an SPFC levee was set to be the 55/57 design profile (the design standard for construction of the State Plan of Flood Control) plus freeboard (3 feet), or the existing TOL elevation as determined by the ULE and NULE projects, whichever was greater. This means repairing or reconstructing all SPFC levees to their design TOL, or the existing TOL, whichever is greater. For the purposes of hydraulic

modeling on a systemwide scale, a reconstructed levee is assumed to have zero probability of failure until it is overtopped.

3.6.2 San Joaquin UNET Model

Upstream, Downstream, Interior, and Internal Boundary Conditions
Upstream, downstream, interior, and internal boundary conditions were unchanged from the No Project condition.

Levee Breach Modeling
For the Achieve SPFC Design Flow Capacity Approach, the breach elevation at each levee breach point on an SPFC levee was set to be the 55/57 design profile plus freeboard (3 feet), or the existing TOL elevation as determined by the ULE and NULE projects, whichever is greater. This means repairing or reconstructing all SPFC levees to their design TOL, or the existing TOL, whichever is greater. For the purposes of hydraulic modeling on a systemwide scale, a reconstructed levee is assumed to have zero probability of failure until it is overtopped.

3.7 Model Assumptions: Protect High Risk Communities Approach

This approach evaluates improvements to levees to protect life, safety, and property for high risk population centers, including urban and small communities. Levees in rural-agricultural areas would remain in their existing configurations.

Tables 3-1 and 3-2 show modeling assumptions for the Sacramento and San Joaquin river basins, respectively, for the No Project condition and each of the CVFPP approaches. The following sections describe specific assumptions for the Sacramento UNET Model and San Joaquin UNET Model for the Protect High Risk Communities Approach.

3.7.1 Sacramento UNET Model

Upstream, Downstream, Interior, and Internal Boundary Conditions
Upstream, downstream, interior, and internal boundary conditions were unchanged from No Project.

Levee Breach Modeling
For the Protect High Risk Communities Approach, breach elevations for all levees were the same as for the No Project condition, except levees in urban areas, where the elevations of the levees were raised to a design water surface elevation that would pass a flood with a 0.5 percent AEP

(200-year), with 3 feet of freeboard. The breach elevations for all levees that were modified as part of this approach were set to the TOL, meaning that for the purposes of hydraulic modeling on a systemwide scale, the probability of levee failure is zero until the levee is overtopped. If an existing urban levee had a TOL that was already higher than the 0.5 percent AEP design water surface plus freeboard, the TOL was left as existing, and the breach elevation was set to the TOL.

3.7.2 San Joaquin UNET Model

Upstream, Downstream, Interior, and Internal Boundary Conditions

Upstream, downstream, interior, and internal boundary conditions were unchanged from No Project.

Levee Breach Modeling

For the Protect High Risk Communities Approach, breach elevations for all levees were the same as for the No Project condition, except levees in urban areas, where the elevations of the levees were raised to a design water surface elevation that would pass a flood with a 0.5 percent AEP (200-year), with 3 feet of freeboard. The breach elevations for all levees that were modified as part of this approach were set to the TOL, meaning that for the purposes of hydraulic modeling on a systemwide scale, the probability of levee failure is zero until the levee is overtopped. If an existing urban levee had a TOL that was already higher than the 0.5 percent AEP design water surface plus freeboard, the TOL was left as existing, and the breach elevation was set to the TOL.

3.8 Model Assumptions: Enchance Flood System Capacity Approach

This approach evaluates opportunities to achieve multiple benefits through enhanced flood system storage and conveyance capacity, to protect high risk communities, and to fix levees in place in rural-agricultural areas. This approach combines the features of the above two approaches and provides more room within flood conveyance channels to lower flood stages throughout most of the system, with additional features and functions for ecosystem restoration and enhancements.

Tables 3-1 and 3-2 show modeling assumptions for the Sacramento and San Joaquin river basins, respectively, for the No Project condition and each of the CVFPP approaches. The following sections describe specific assumptions for the Sacramento and San Joaquin UNET models for the Enhance Flood System Capacity Approach.

3.8.1 Sacramento UNET Model

Upstream, Downstream, and Interior Boundary Conditions

Upstream boundary conditions were modified to include reservoir operation criteria modifications at Oroville Dam and New Bullards Bar Dam as described in Attachment 8B: Reservoir Analysis. Downstream and interior boundary conditions were unchanged from the No Project condition.

Levee Breach Modeling

For the Enhance Flood System Capacity Approach, the breach elevation at each levee breach point on a nonurban SPFC levee was set to be the 55/57 design profile (the design standard for construction of the State Plan of Flood Control) plus freeboard (3 feet), or the existing TOL elevation as determined by the ULE and NULE projects, whichever was greater. This means repairing or reconstructing all nonurban SPFC levees to their design TOL, or the existing TOL, whichever is greater.

Levee breach elevations of urban levees were raised to a design water surface elevation that would pass a flood with a 0.5 percent AEP (200-year), with 3 feet of freeboard or the 55/57 design profile with 3 feet of freeboard, whichever was greater. The breach elevations for urban levees were set to the TOL, meaning that for the purposes of hydraulic modeling on a systemwide scale, the probability of levee failure is zero until the levee is overtopped. If an existing urban levee had a TOL that was already higher than the 0.5 percent AEP design water surface plus freeboard, the TOL was left as existing, and the breach elevation was set to the TOL.

The breach elevations for levees on both sides of the channel in reaches where levees were setback were set to the TOL, representing the new setback levees and modifications to the existing levees. A reconstructed levee is assumed to have zero probability of failure until it is overtopped.

Internal Boundary Conditions

Internal boundary conditions were modified to include floodplain storage on easements, as described in Table 3-1. Storage areas were also used in the Yolo Bypass to represent widening of the bypass. Two sets of eight gates were added to the Sacramento Bypass structure. The length of the Fremont Weir was increased by 1 mile. A 25,000 cfs bypass was added between the Feather River near Biggs and the Butte Basin.

Cross Section Modifications

Cross sections were modified in specified reaches (Table 3-1) of the Sacramento and Feather rivers to represent levee setbacks. Cross sections were also modified in the Sutter, Yolo, and Sacramento bypasses to

represent widening of the bypasses. Cross sections were added to represent the bypass between the Feather River and the Butte Basin.

3.8.2 San Joaquin UNET Model

Upstream, Downstream, and Interior Boundary Conditions

Upstream boundary conditions were modified to include reservoir operation criteria modifications to Friant, New Exchequer, and New Don Pedro dams as described in Attachment 8B: Reservoir Analysis. Downstream and interior boundary conditions were unchanged from the No Project condition.

Levee Breach Modeling

For the Enhance Flood System Capacity Approach, the breach elevation at each levee breach point on a nonurban SPFC levee was set to be the 55/57 design profile plus freeboard (3 feet), or the existing TOL elevation as determined by the ULE and NULE projects, whichever is greater. This means repairing or reconstructing all nonurban SPFC levees to their design TOL, or the existing TOL, whichever is greater.

Levee breach elevations of urban levees were raised to a design water surface elevation that would pass a flood with a 0.5 percent AEP (200-year), with 3 feet of freeboard or the 55/57 design profile with 3 feet of freeboard, whichever was greater. The breach elevations for all levees that were modified as part of this approach were set to the TOL, meaning that for the purposes of hydraulic modeling on a systemwide scale, the probability of levee failure is zero until the levee is overtopped. If an existing urban levee had a TOL that was already higher than the 0.5 percent AEP design water surface plus freeboard, the TOL was left as existing, and the breach elevation was set to the TOL.

The breach elevations for levees on both sides of the channel in reaches where levees were setback were set to the TOL, representing the new setback levees and modifications to the existing levees. A reconstructed levee is assumed to have zero probability of failure until it is overtopped.

Internal Boundary Conditions

Internal boundary conditions were modified to include storage on floodplain easements, as outlined in Table 3-2.

Cross Section Modifications

Cross sections were modified to represent levee setbacks along the mainstem San Joaquin River at locations between the Merced and Stanislaus rivers, as described in Table 3-2.

3.9 Model Assumptions: State Systemwide Investment Approach

The State Systemwide Investment Approach (SSIA) reflects the State's strategy to address current challenges and affordably meet the 2012 CVFPP Goals. The preliminary approaches, described previously, suggested a broad range of physical and institutional flood damage reduction actions to improve public safety and achieve economic, environmental, and social sustainability. The SSIA is an assembly of the most promising, affordable, and timely elements of the three preliminary approaches.

Tables 3-1 and 3-2 show modeling assumptions for the Sacramento and San Joaquin river basins, respectively, for the No Project condition and each of the CVFPP approaches. The following sections describe specific assumptions for the Sacramento and San Joaquin UNET models for the State Systemwide Investment Approach.

3.9.1 Sacramento UNET Model

Upstream, Downstream, and Interior Boundary Conditions
Upstream, downstream, and interior boundary conditions were unchanged from the No Project condition.

Internal Boundary Conditions
Storage areas were used in the Yolo Bypass to represent widening of the bypass. The length of the Fremont Weir was increased by 1 mile. A bypass was added between the Feather River near Biggs and the Butte Basin.

Cross Section Modifications
Cross sections were modified in the Sutter and Yolo bypasses to represent widening of the bypasses. Cross sections were added to represent the 25,000 cfs Biggs Bypass from the Feather River to the Butte Basin.

Levee Breach Modeling
Levee breach elevations were the same as in the Protect High Risk Communities Approach, except that new levees resulting from widening the Yolo and Sutter bypasses were assumed to fail only on overtopping.

3.9.2 San Joaquin UNET Model

Upstream, Downstream, Interior, and Internal Boundary Conditions
Upstream, downstream, interior, and internal boundary conditions were unchanged from the No Project condition.

Levee Breach Modeling

Levee breach elevations were the same as the Protect High Risk Communities Approach.

3.10 Model Limitations

It is important to note some of the basic capabilities, assumptions, and limitations inherent with the UNET models. UNET is used to simulate one-dimensional, fully unsteady flow. It is a fixed-bed analysis and does not account for sediment movement, scour, or deposition. The models assume no exchange with groundwater. The model is intended to adequately reproduce levee breaks and breaches and simulate channel hydraulics. The spacing of cross sections in the UNET models (1,000 to 1,500 feet) is appropriate for large systemwide analyses; however, it also limits the application of these models to analysis requiring more detail.

3.11 Model Output Formats

As an unsteady flow model, UNET produces extensive results. For purposes of this attachment, the results are displayed as Stage- and Flow-Frequency curves and as Out-of-System Flows, as described below.

3.11.1 Stage- and Flow-Frequency Curves

Outputs from the hydraulic models would be shown in two formats: stage-frequency curves and flow-frequency curves. For a given location and return period, the highest peak stage, generated by any of the storm centerings, was selected to represent the maximum stage for that location and return period. The maximum stages for all return periods were plotted to generate the stage-frequency curve, as illustrated in Figure 3-4 using stages for only two sets of storm centerings to simplify the example. This same approach was used to obtain the flow-frequency curve for each location.

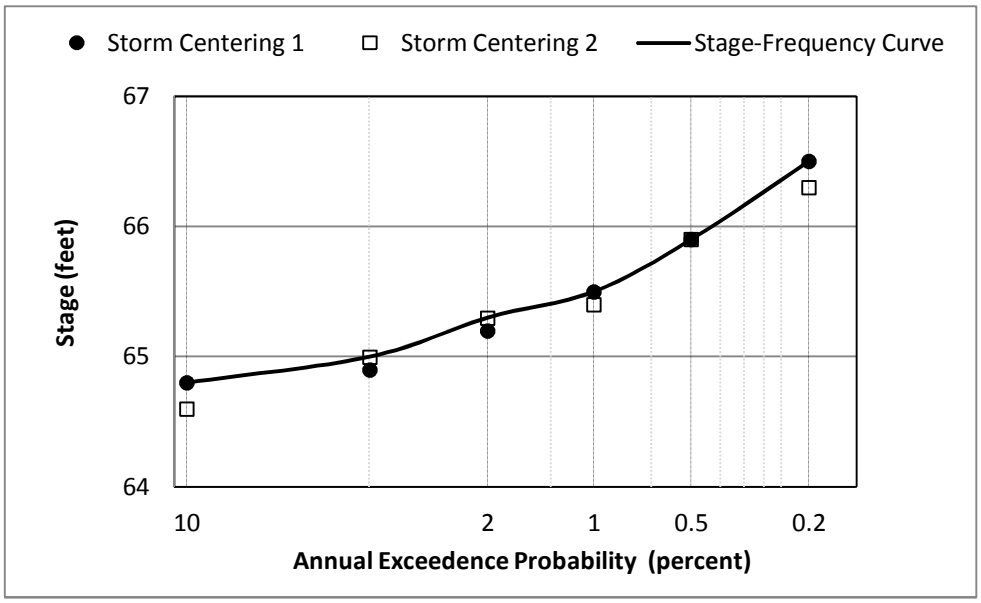


Figure 3-4. Illustration of Stage-Frequency Curve

3.11.2 Out-of-System Flows

To understand the operation of the flood management system, it is also necessary to know how much of a flood has left the river channels and has entered the floodplain. In a leveed reach of a river, this would mean that the levee had breached and water was leaving the river channel and entering the floodplain behind the levee. A levee breach can have a significant effect on stage and flow in the river channel adjacent to or downstream from the breach.

If a flood management system approach improves levees, floodwater that would have previously left the channel through a levee breach would continue downstream, thus increasing stage and flow at downstream locations and potentially causing downstream levee breaches. In addition, stages in the river would increase at the location where the breach previously occurred.

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4.0 Sacramento and San Joaquin River Basins Results

Figures 4-1 and 4-2 indicate the locations in the Sacramento and San Joaquin river basins at which stage- and flow-frequency curves will be plotted to allow comparison of the operations of the flood management systems among the No Project condition and the various approaches. The floodplains of the Sacramento and San Joaquin river basins have been subdivided into flood zones, which are also shown in Figures 4-1 and 4-2. Out-of-system volume in the flood zones was used in conjunction with the flow- and stage-frequency curves to demonstrate how the approaches differ as to in-channel flows, stage, and out-of-channel flow at various locations in each river basin.

It is important to remember that the results shown in this section area based come from a systemwide analysis and while they are indicative of system problems and general results from the various approaches, the results should not be used to design or analyze any specific location. Model results at a given location are often highly dependent on the upstream modeling assumptions.

All graphic and tabular results referenced in this section have been placed at the end of this section for easier access and readability.

4.1 Sacramento River Basin

This section describes UNET model output for the Sacramento River Basin and the simulated peak flow rates and stages for storms of various frequencies for the No Project condition and all approaches.

There are 13 model output locations in the Sacramento River Basin (see Figure 4-1). Seven locations are along the Sacramento River; the remaining six are on the Feather River, American River, Sutter Bypass, and Yolo Bypass.

Abbreviations are used on the flow- and stage-frequency plots to designate the No Project condition and the approaches, as follows:

- No Project = No Project Condition
- SPFC = Achieve SPFC Design Flow Capacity Approach

- PHRC = Protect High Risk Communities Approach
- EFSC = Enhance Flood System Capacity Approach
- SSIA = State Systemwide Investment Approach

4.1.1 Flow- and Stage-Frequency Curves

Figures 4-3 through 4-15 show flow- and stage-frequency curves for all of the approaches for each of the 13 selected output locations in the Sacramento River Basin (Figure 4-1). Because of differences in elevations and flows between the output locations, scales on the flow- and stage-frequency curves are not the same for all the output locations.

A detailed result description is given on the facing page of each of the paired flow- and stage- frequency curves (Figures 4-3 through 4-15).

4.1.2 Out-of-System Volumes

Figure 4-1 shows the geographic extent of designated flood zones in the Sacramento River Basin. The flood zones are groupings of impact areas or floodplains used to tabulate the volume of floodflows leaving the flood management system during a given flood. Table 4-1 contains the out-of-system volume for each of the approaches in each of the flood zones. These out-of-system volumes are instrumental in understanding the function of the system. For example, the stage at a given location may be lower for the 100-year flood than for the 50-year flood. If flood zones upstream from this location are reviewed and a significant increase is observed in out-of-system volume in the upstream flood zones between the 50- and the 100-year floods, it can be concluded that a levee breach upstream from the location likely has reduced the flows to a level less than the 50-year flow.

Another example would be a location where the stage between No Project condition and one of the approaches increases significantly for the same AEP flood. Again, if upstream out-of-system volume is reduced, it can be concluded that additional flow remains in the river because upstream levees may have been reconstructed or raised and no longer breach as they did in the No Project condition.

4.1.3 Flows to Delta

Table 4-2 contains the volume of flow entering the Delta for the No Project condition and each of the approaches for the Sacramento River Basin. Flow volume into the Delta is another important factor to consider when comparing approaches. The model measures flow volume into the Delta as the sum of the volume in the Yolo Bypass that passes Lisbon and the flow

volume downstream from the confluence of the Sacramento and American rivers.

4.2 San Joaquin River Basin

This section describes the UNET model output for the San Joaquin River Basin and the simulated peak flow rates and stages for storms of various frequencies for the No Project condition and all approaches.

There are nine model output locations in the San Joaquin River Basin (see Figure 4-2). Six locations are along the San Joaquin River; the remaining three are on the Fresno River, Chowchilla Bypass, and Eastside Bypass.

4.2.1 Flow- and Stage-Frequency Curves

Results of the San Joaquin River Basin UNET model were processed using the same methodology used for the Sacramento River Basin. Figures 4-16 through 4-25 show flow- and stage-frequency curves for all of the approaches, for each of the nine selected output locations in the San Joaquin River Basin (Figure 4-2). Because of differences in elevations and flows between the output locations, scales on the flow- and stage-frequency curves are not the same for all the output locations.

A detailed result description is given on the facing page of each of the paired flow- and stage- frequency curves (Figures 4-16 through 4-25).

4.2.2 Out-of-System Volumes

Figure 4-2 shows the geographic extent of designated flood zones in the San Joaquin River Basin. Table 4-3 contains the out-of-system volume for each of the approaches in each of the flood zones in the San Joaquin River Basin. These out-of-system volumes are instrumental in understanding the function of the system.

4.2.3 Flows to the Delta

Table 4-4 contains the volume of flow entering the Delta for the No Project condition and each of the approaches for the San Joaquin River Basin. The model measures flow volume into the Delta from the San Joaquin River as the volume that passes the gage at Vernalis.

4.3 Summary Findings

This section describes some of the systemwide findings that can be drawn from the data presented in this section.

4.3.1 Achieve SPFC Design Flow Capacity Approach

Restoring all SPFC levees to their original design flow capacity for the Achieve SPFC Design Flow Capacity Approach would significantly reduce the number of levee breaks and therefore keep more flow in the river channels, causing increased stages and flows in both the Sacramento and San Joaquin river basins. With the restored levees, the floodwaters that would have left the system in the No Project condition would continue downstream. As the increased flows and stages continue downstream they cause levee breaks in the lower reaches of both the Sacramento and San Joaquin rivers (Tables 4-1 and 4-3), sometimes in places where the levees did not break in the No Project condition.

Flow volumes entering the Delta increase significantly over the No Project condition for all flood frequencies in both river basins (Tables 4-2 and 4-4).

Protect High Risk Communities Approach

The Protect High Risk Communities Approach modifies urban levees to pass the 200-year (0.5 percent AEP) flood with 3 feet of freeboard. Since only urban levees and a few small communities are modified, flows and stages in both the Sacramento and San Joaquin river basins would remain essentially the same as for No Project condition. The only exceptions would arise if an urban area sustained a levee breach in the No Project condition. In that case, the flows and stages downstream would increase due to the rebuilding of the urban levee so that the levee breach did not occur.

Flow volumes entering the Delta are essentially the same as No Project condition for all floods, except for the 0.2 percent AEP (500-year) flood in which some urban areas that had levee breaches in the No Project condition remain dry, sending additional flow into the Delta.

Enhance Flood System Capacity Approach

The Enhance Flood System Capacity Approach modifies urban levees to pass the 200-year (0.5 percent AEP) flood with 3 feet of freeboard. In addition, the breach elevations for nonurban SPFC levees were set to be the 55/57 design profile (the design standard for construction of the State Plan of Flood Control) plus freeboard (3 feet), or the existing TOL elevation as determined by the ULE and NULE projects, whichever was greater.

Other key components of the approach are added upstream reservoir storage, widened and new bypasses, levee setbacks, and floodplain storage. The added upstream storage would result in lower stages in the upper Feather, San Joaquin, Merced, and Tuolumne rivers. Floodplain storage and levee setbacks would result in lower stages in the Sutter Bypass and lower Feather River, as well as the Sacramento River downstream from the

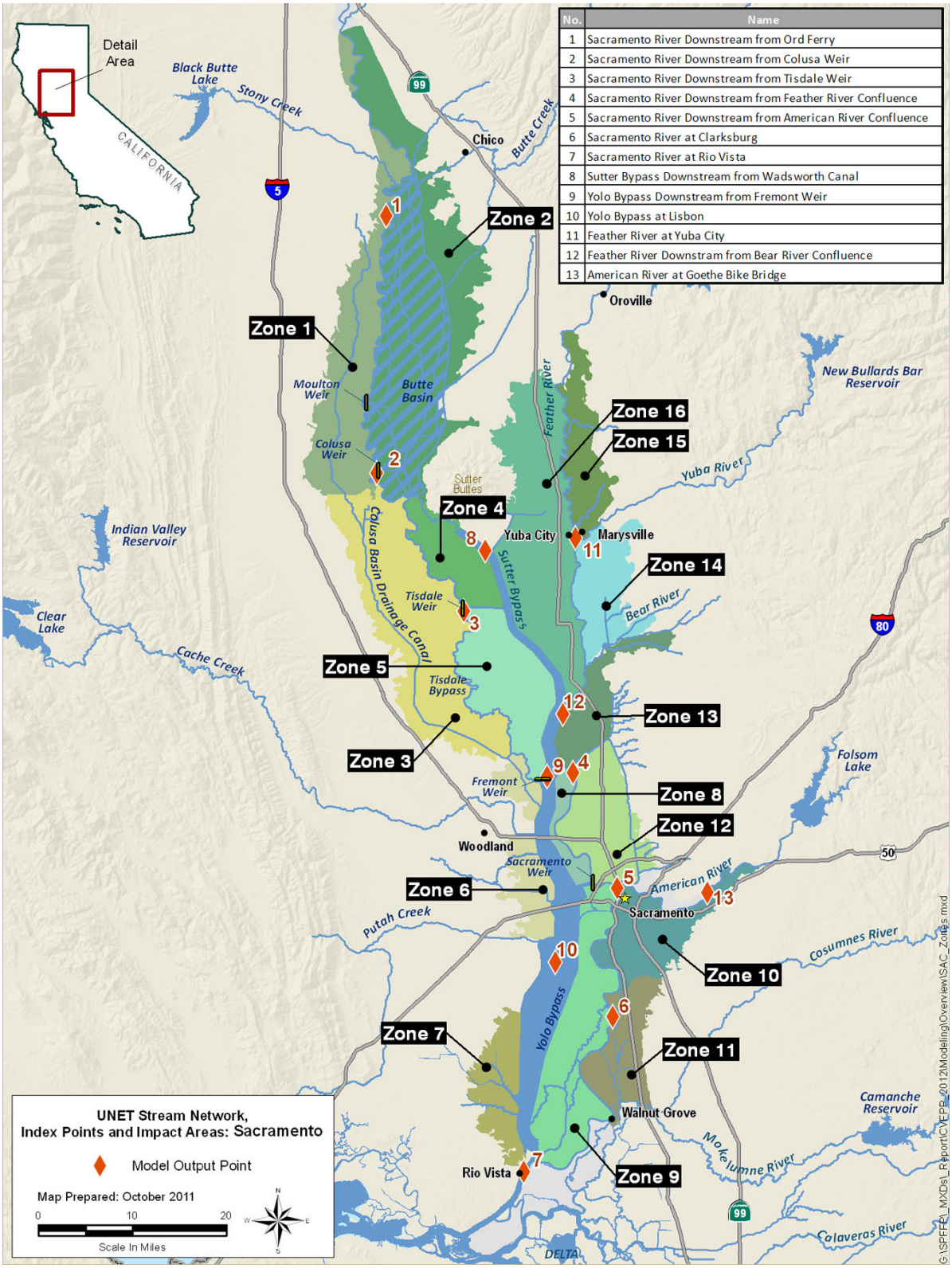
Tisdale Weir. These lower stages would continue downstream in the Yolo Bypass and lower Sacramento River. Higher stages would be seen in the Chowchilla and Eastside bypasses as a result of levee fixes that increase the volume of water remaining in the bypasses all the way from the San Joaquin River to the Merced River. Stages downstream from the Tuolumne River to Stockton would also be lowered as a result of floodplain storage and levee setbacks.

Even though restoring all urban and SPFC levees as described above should result in additional flow volumes entering the Delta, flow volumes entering the Delta are significantly decreased for the 10, 4, 2, and 1 percent (10-, 25, 50-, 100-year) floods as a result of the added upstream reservoir and floodplain storage. For the 0.5 and 0.2 percent AEP (200- and 500-year) floods the reservoir and floodplain storage is not enough to prevent an increase in flow into the Delta.

State Systemwide Investment Approach

The State Systemwide Investment Approach consists of the same improvements to urban levees included in the Protect High Risk Communities Approach. In addition, a new bypass (Biggs) and widening of the Yolo and Sutter bypasses are included in the Sacramento River Basin, and Paradise Cut Bypass is widened in the San Joaquin River Basin. Flows and stages for the State Systemwide Investment Approach are similar to the Protect High Risk Communities Approach, except where changes to the bypasses reduce stages.

Flows entering the Delta from the Sacramento River Basin are marginally increased for less frequent floods because there are fewer levee breaches as a result of the urban levee improvements and the widening of the bypasses. Flows entering the Delta from the San Joaquin River Basin are essentially the same as for the Protect High Risk Communities Approach.



No.	Name
1	Sacramento River Downstream from Ord Ferry
2	Sacramento River Downstream from Colusa Weir
3	Sacramento River Downstream from Tisdale Weir
4	Sacramento River Downstream from Feather River Confluence
5	Sacramento River Downstream from American River Confluence
6	Sacramento River at Clarksburg
7	Sacramento River at Rio Vista
8	Sutter Bypass Downstream from Wadsworth Canal
9	Yolo Bypass Downstream from Fremont Weir
10	Yolo Bypass at Lisbon
11	Feather River at Yuba City
12	Feather River Downstram from Bear River Confluence
13	American River at Goethe Bike Bridge

Figure 4-1. Sacramento River Basin Output Locations and Flood Zones

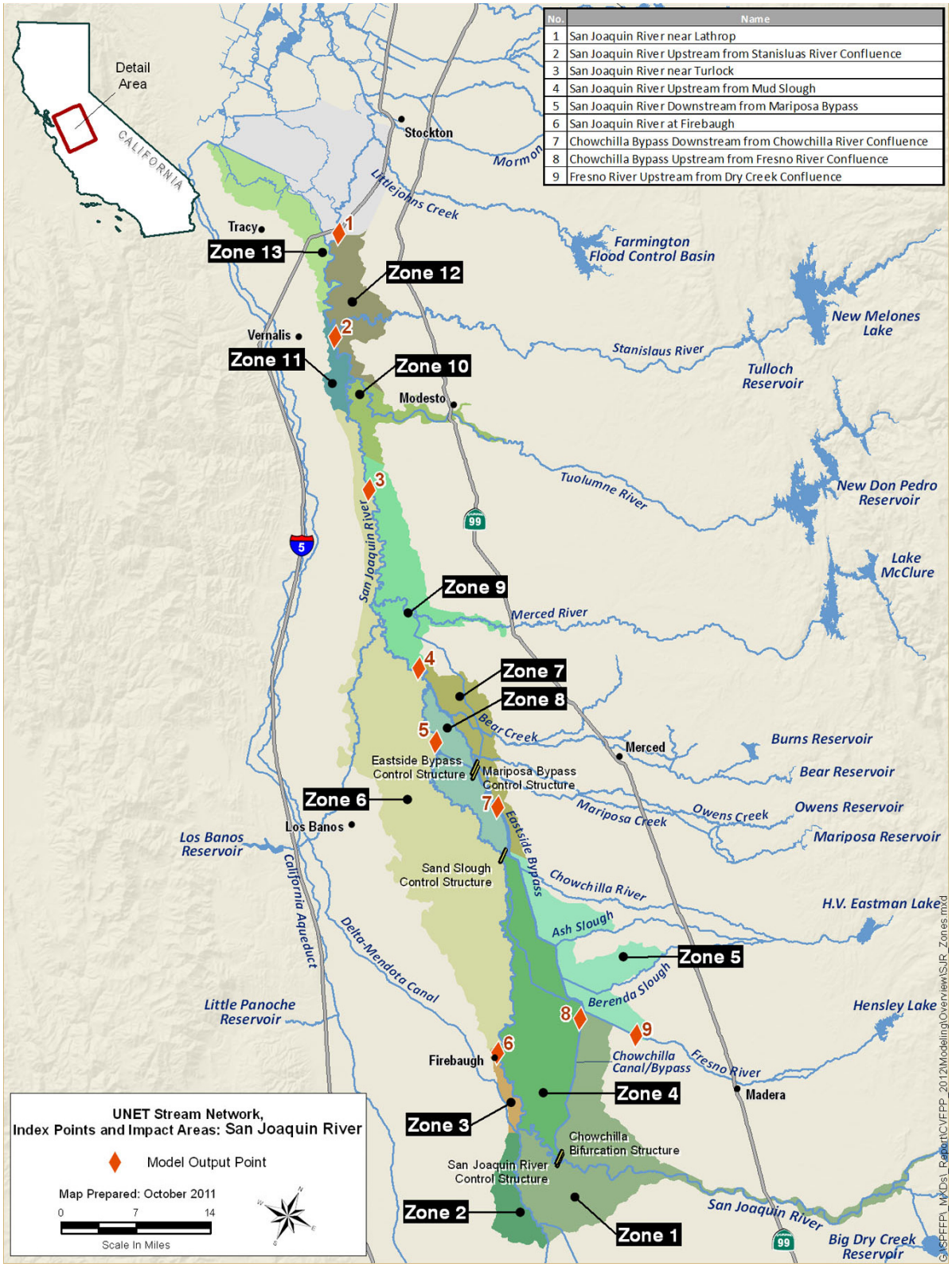
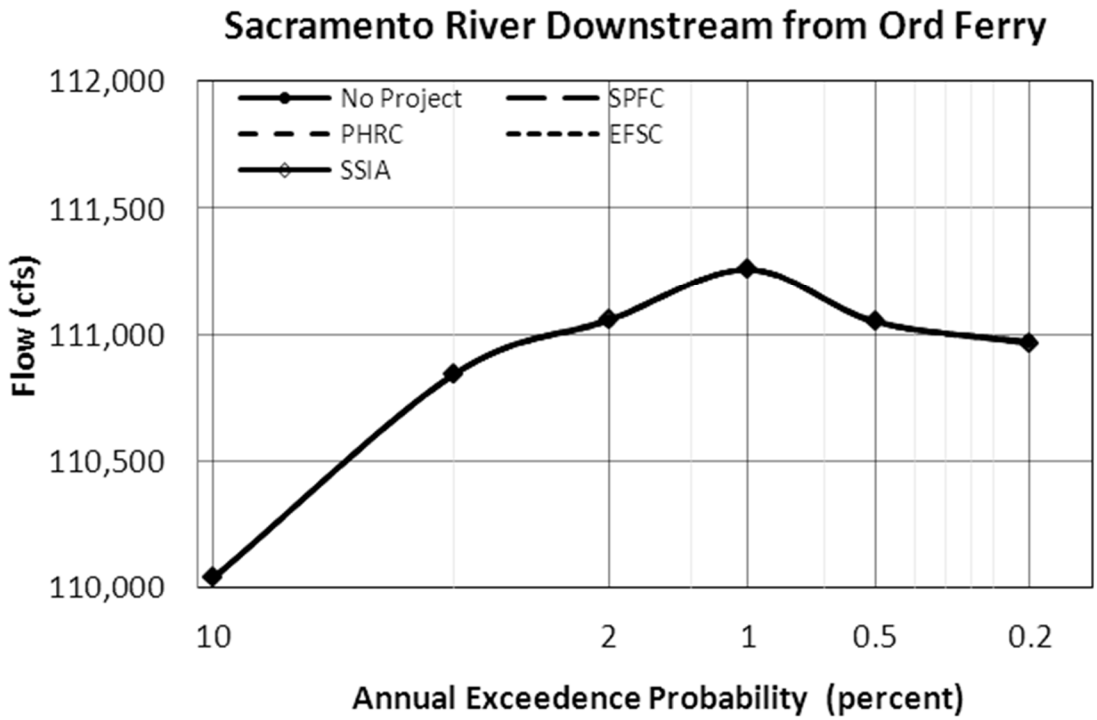


Figure 4-2. San Joaquin River Basin Output Locations and Flood Zones

- **No Project Condition and All Approaches –**No modifications to the existing flood management system upstream from this location or in close proximity downstream, so flows are the same for all cases (flows are largely controlled by boundary inflows). Flows decrease slightly for the 0.5 percent and 0.2 percent floods because higher flows cause more outflow through levee breaks. However, stage continues to rise for larger flood events as a result of increasing backwater effects resulting from increased flows downstream.

- **SPFC Approach –** Restoring SPFC levees (to the 57 design profile) reduces the number of levee breaks downstream from this location, without any improvements to reservoir flood management pools, floodplain storage capacity, bypass conveyance capacity, or channel conveyance capacity, resulting in higher stages downstream from this location than in No Project condition or other approaches. This backwater effect travels upstream to this location, and causes stages to increase slightly.



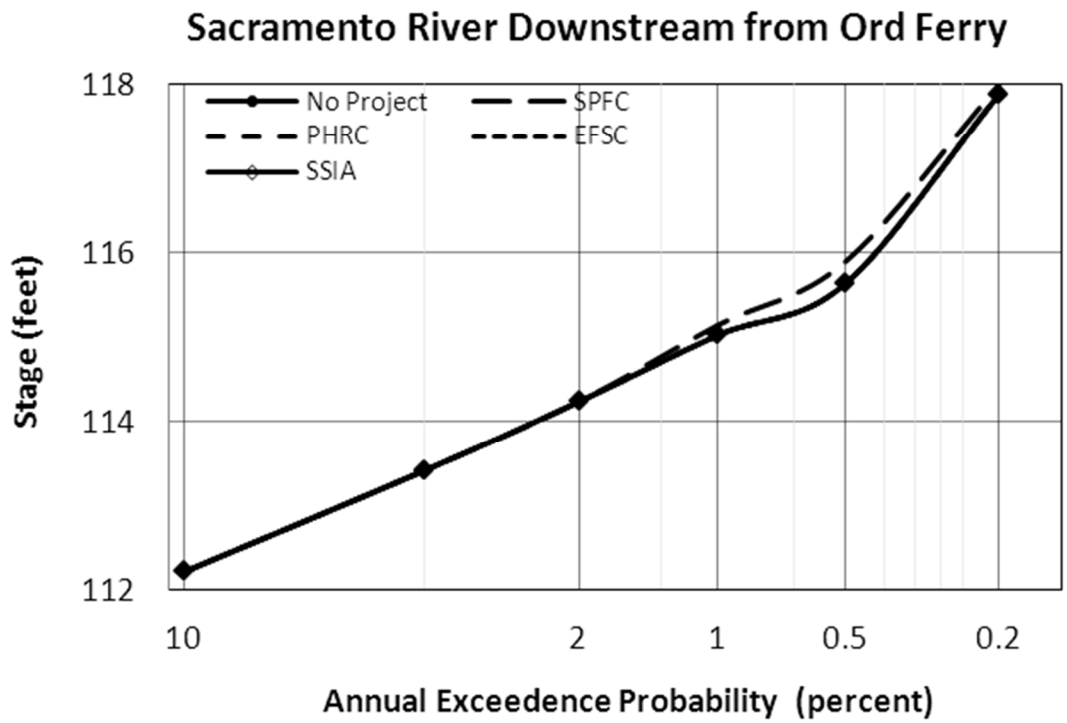
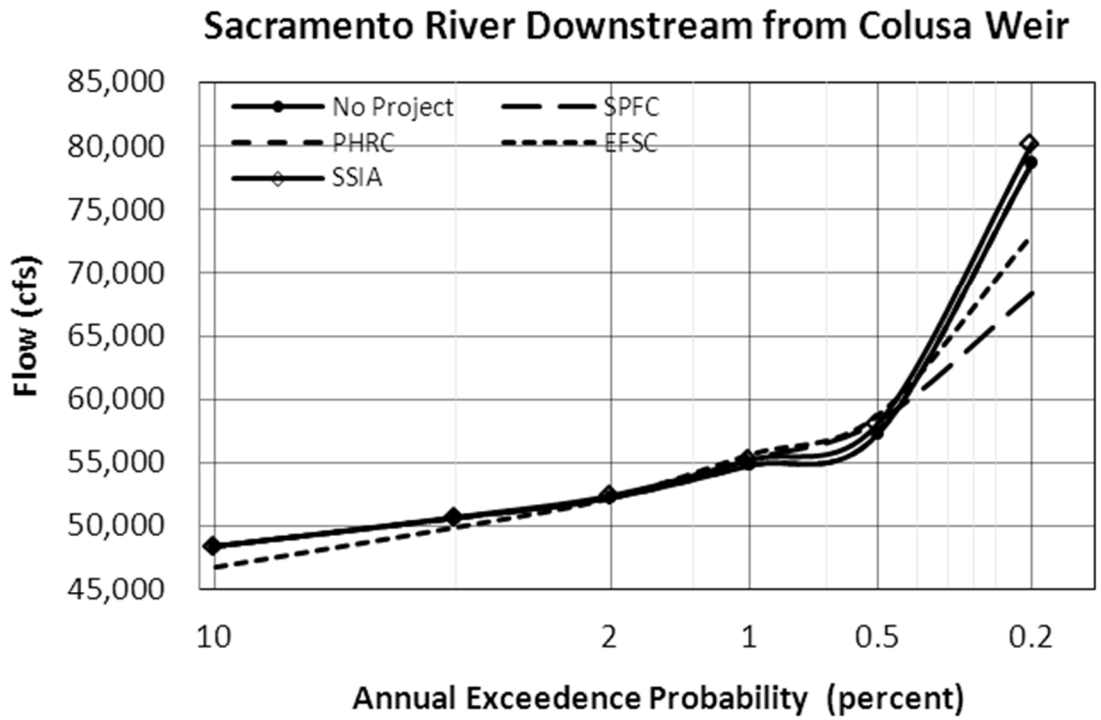


Figure 4-3. Flow- and Stage-Frequency Curves: Sacramento River Downstream from Ord Ferry [1]

- **No Project Condition and All Approaches –**Flow and stage are similar for all events through the 1 percent AEP flood because there are few modifications to the flood management system upstream from this location for any of the approaches.

- **SPFC Approach –** River stage increases slightly at the 0.5 percent and 0.2 percent AEP events compared to the No Project condition due to increased backwater, which results from SPFC levee restoration downstream from this location. However, flow decreases because there is more flow over the Colusa Weir, as a result of the higher stage.

- **EFSC Approach** – Stage decreases at the 0.5 percent and 0.2 percent AEP events as a result of levee setbacks in this reach of the river. Flow also decreases at the 0.2 percent flood as a result of Sutter bypass widening, which results in a lows stage in the Colusa Bypass at Colusa Weir, and allows more flow the enter the bypass.



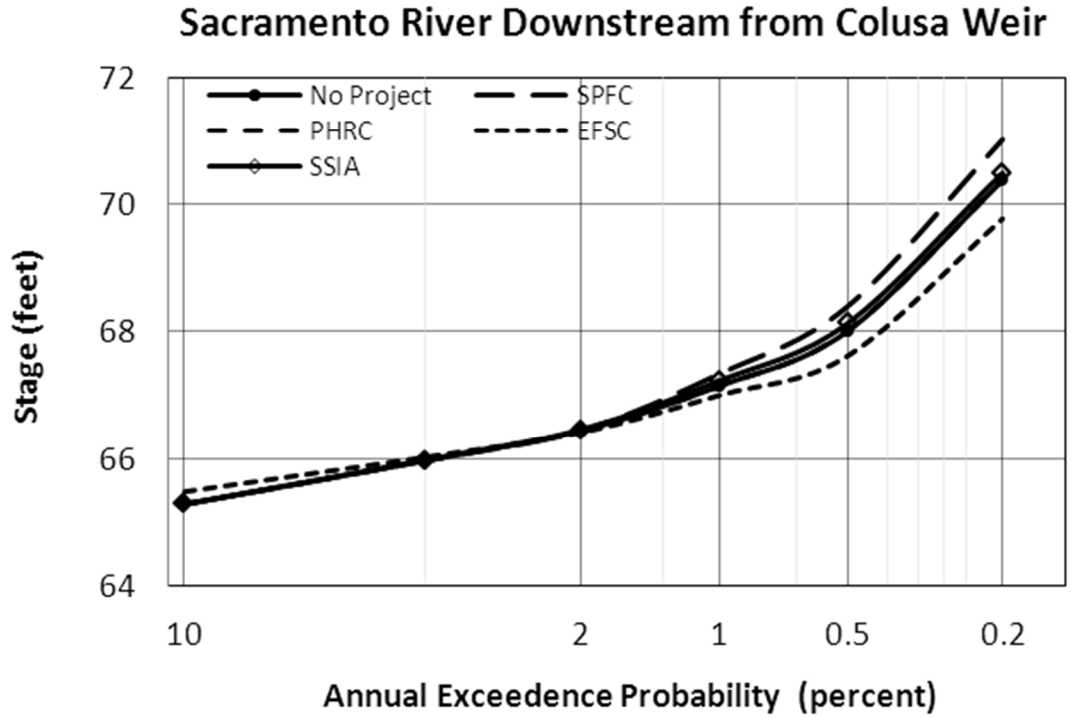


Figure 4-4. Flow- and Stage-Frequency Curves: Sacramento River Downstream from Colusa Weir [2]

- **No Project Condition and PHRC Approach –** Levees break along the Sutter Bypass upstream from the Tisdale Bypass at the 0.5 percent AEP flood event and greater, increasing flow over the Tisdale weir by lowering the backwater from the Sutter Bypass, thus preventing any major increases in flow or stage downstream from the weir.

- **SPFC Approach –** Restoring SPFC levees prevents a number of upstream levee breaks for the 1 percent AEP flood and greater, increasing in-channel flow and river stage upstream from the Tisdale Bypass compared to the No Project condition. However, the flow over the Tisdale Weir into the Tisdale Bypass is generally similar to or less than in the No Project condition, because the stage in the Tisdale and Sutter bypasses is higher (increased stage upstream increases flow over the Moulton and Colusa weirs, so the stage in the Sutter Bypass is higher, resulting in a greater backwater effect on the Tisdale Bypass).

- **EFSC Approach –** The widened Sutter Bypass lowers the stage and allows more flow over the Tisdale weir compared to the No Project condition, as in the SSIA. For the 0.2 percent AEP flood event, the flow in the Sacramento River upstream from the Tisdale Weir increases as a result of levee restoration, but there is also significantly more flow over the Tisdale Bypass as a result of the higher stage in the Sacramento River, so flow and stage are similar to the No Project condition.

- **SSIA –** Stage in the Tisdale Bypass is significantly lower than in the No Project condition through the 0.5 percent AEP flood as a result of widening the Sutter Bypass. The stage in the Sacramento River above the Tisdale Weir is similar for those events, so flow over the Tisdale Weir into the Tisdale bypass is greater. Increasing flow over the Tisdale Weir at the 0.5 percent AEP event prevents any major change in flow or stage downstream on the Sacramento River. For the 0.2 percent AEP event, flow and stage tend to converge with the No Project condition because some of the water in the floodplain enters the Tisdale Bypass, which increases the backwater effect in the bypass to a level similar to the No Project condition (floodplain flows also reenter the bypass in the No Project condition, but at a lower rate, because the stage in the bypass is higher when inflow begins).

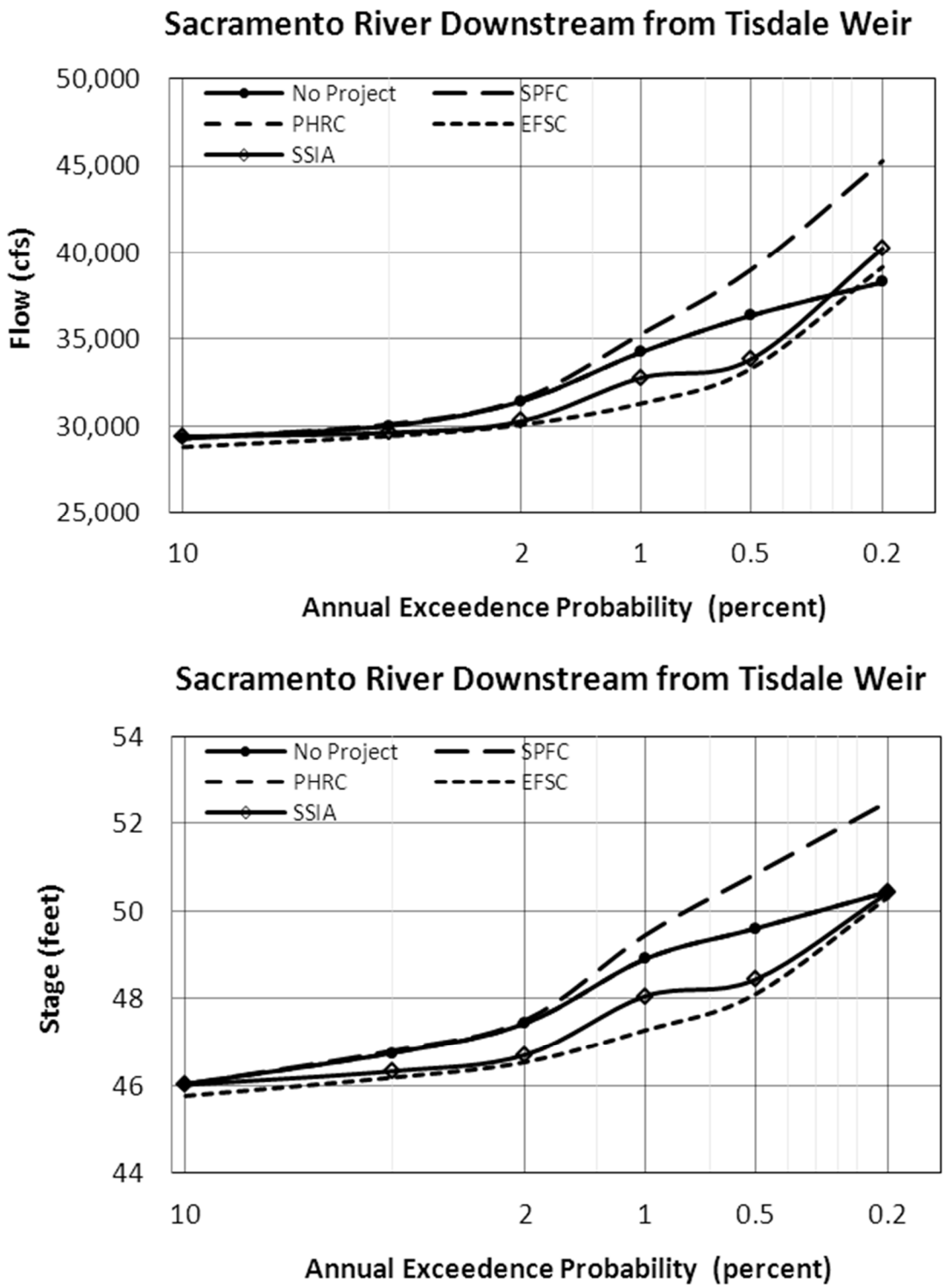


Figure 4-5. Flow- and Stage-Frequency Curves: Sacramento River Downstream from Tisdale Weir [3]

- **SPFC Approach –** Restoring SPFC levees (to the 55/57 design profile) reduces the number of upstream levee breaks, resulting in generally higher in-channel flows. The higher flows are particularly exaggerated for the 0.2 percent AEP. River stages at this point are also higher as a result of the levee reconstruction included in this approach.

- **PHRC Approach –** Produces results similar to the No Project condition at this location because there are few improved upstream urban levees, and effects from downstream changes in river flows resulting from urban levee improvements are negligible.

- **EFSC Approach –** Improvements to the flood management system - including bypass improvements, additional floodplain storage areas, and increased reservoir flood management storage -reduce peak flows for smaller flood events. For larger flood events (0.5 percent AEP and smaller), the relative effect of these improvements on in-channel flows is overwhelmed by the reduced number of upstream levee breaks (resulting from improved urban and restored non-urban levees), which tends to increase in-channel flows. Also, flow into an added floodplain storage area just downstream from this location significantly lowers the river stage adjacent to the storage area. This effectively reduces the backwater effect at the Feather River Confluence, which allows the for a temporarily higher flow rate along with a much lower stage compared to other approaches, especially during very large events.

- **SSIA** – Flows are generally lower than the No Project condition because of bypass improvements, which increase their capacity and reduce river flows at this location. At the 0.2 percent AEP, significant levee breaks occur in the No Project condition, reducing flows and stages to a level closer to the SSIA approach.

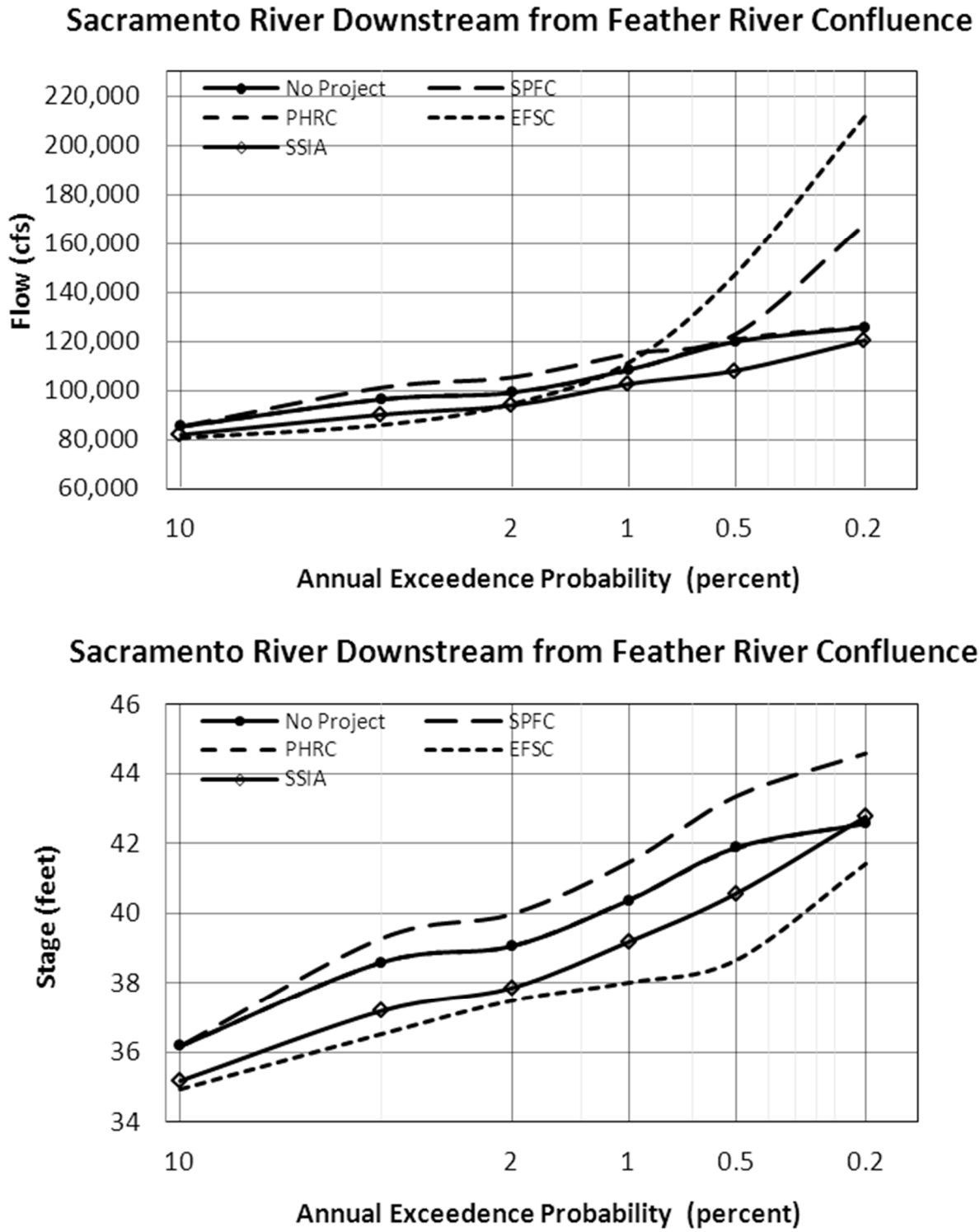
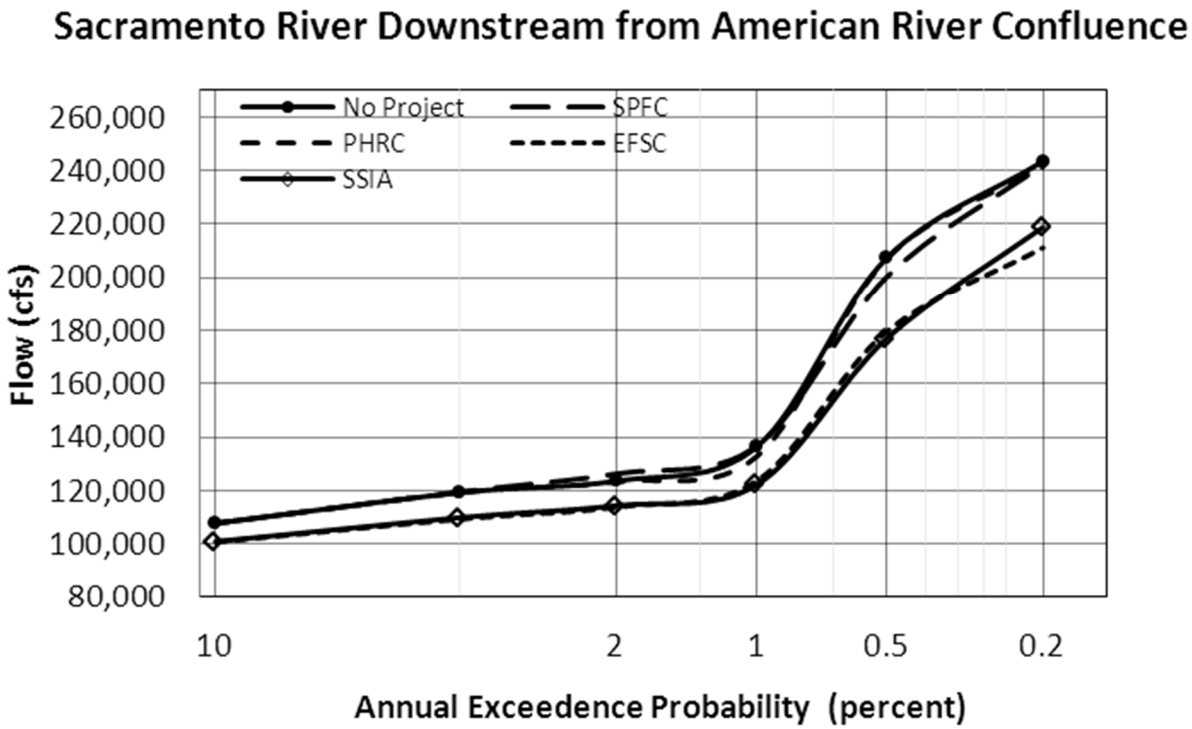


Figure 4-6. Flow- and Stage-Frequency Curves: Sacramento River Downstream from Feather River Confluence [4]

- **No Project Condition, SPFC and PHRC Approaches –** Flows are similar because the Sacramento Bypass diverts a similar portion of flow in each case. For the PHRC and SPFC approaches, stage is generally higher because levee restoration prevents some downstream levee breaks, increasing the backwater effect at this location.

- **EFSC Approach and SSIA** – Flow and stage are lower than the No Project condition because of increased outflows through the widened Sacramento Bypass upstream from this location.



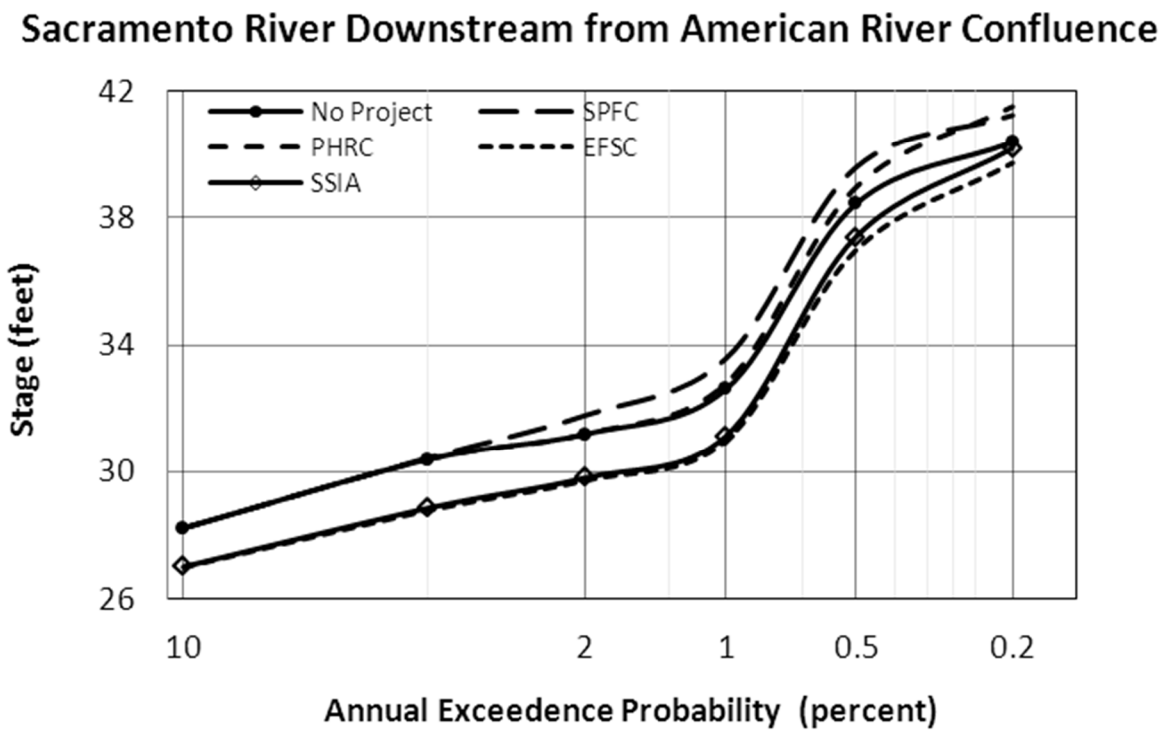


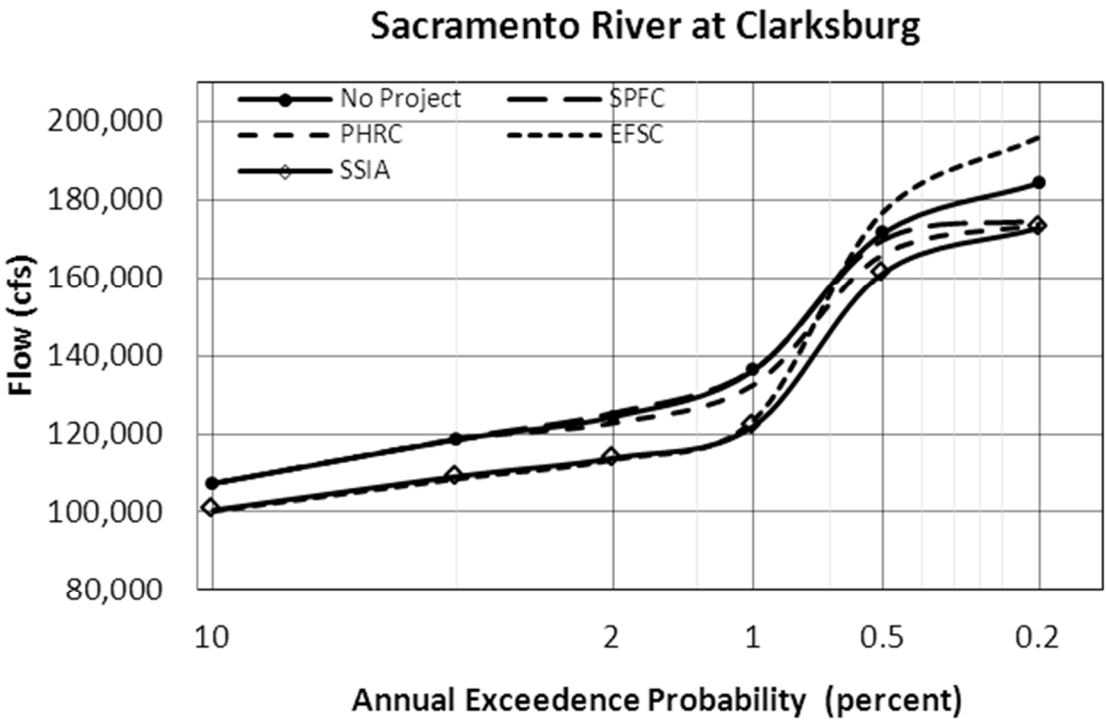
Figure 4-7. Flow- and Stage-Frequency Curves: Sacramento River Downstream from American River Confluence [5]

- **SPFC Approach –** Higher stages at this location than the No Project condition and the other approaches because restoration of all SPFC levees reduces the number of levee breaks both upstream and downstream from this point. However, the increased backwater effect (from increased downstream stages) tends to reduce the velocity of flow, leading to flow rates that are similar to or less than the No Project approach.

- **PHRC Approach** – Leads to higher maximum stages than the No Project condition at this location because improved upstream urban levees would fail at higher flows, resulting in more flow remaining in the system and entering the Yolo Bypass. When that flow re-enters the Sacramento River at Rio Vista, it creates a backwater effect which extends up the Sacramento River to this location. Flows at this location are similar to No Project flows because the levees below Sacramento are unimproved, and tend to break at the same frequency as No Project levees. For large events, the backwater effect is great enough that flow rates are significantly reduced compared to the No Project condition, despite higher water surface elevations.

- **EFSC Approach –** Improvements to the flood management system - including bypass improvements, additional floodplain storage areas, and increase reservoir flood management pools - reduce peak flows for smaller flood events. For larger flood events (200 year and greater), the relative effect of these improvements on in-channel flows is overcome by the reduced number of upstream levee breaches (resulting from improved urban and non-urban levees), which tends to increase in-channel flows. Also, an added floodplain storage area just downstream from this point accepts a large portion of river flow during the peak of each flood event, which significantly lowers the river stage adjacent to the storage area. This effectively reduces the backwater effect at Clarksburg, which allows for a temporarily higher flow rate along with a much lower stage compared to other approaches, especially during very large events.

- **SSIA** – Bypass improvements reduce river flows at this location compared to the No Project condition for all flood events, despite the effect of improved urban levees (which act to increase in-channel flows). Increased flows exiting the Yolo Bypass create a backwater effect on the Sacramento River, which results in higher peak water surface elevations at this location than the No Project condition for the 0.5 and 0.2 percent AEP events.



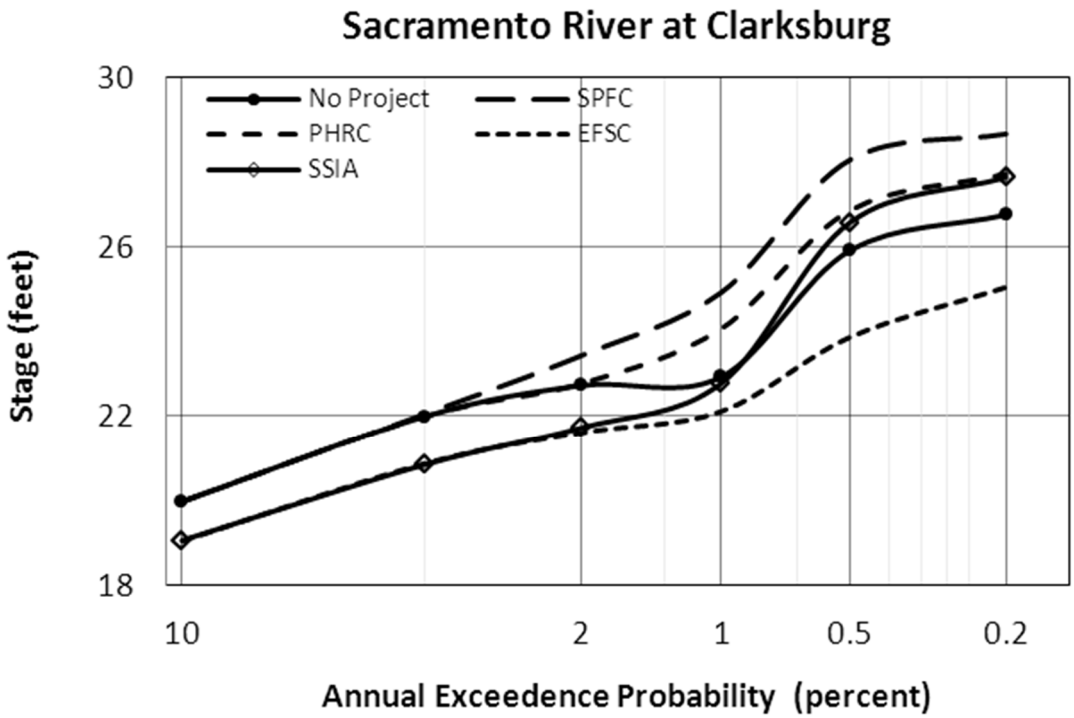


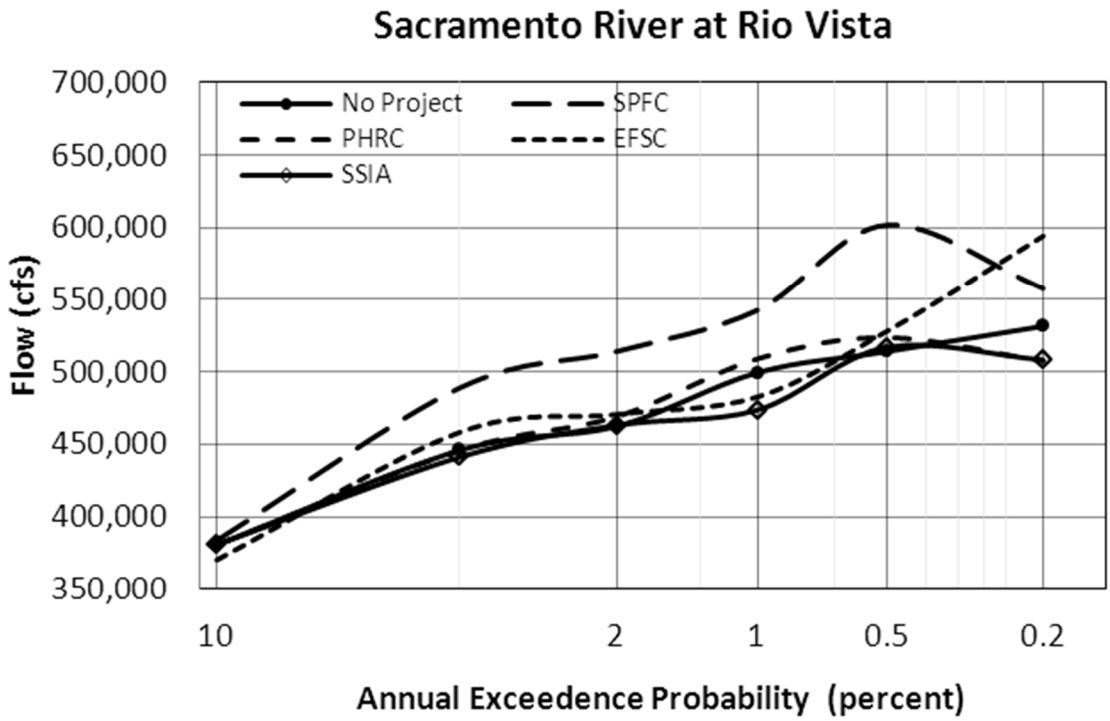
Figure 4-8. Flow- and Stage-Frequency Curves: Sacramento River at Clarksburg [6]

- **No Project Condition and All Approaches –** Flows at this location are largely controlled by the amount of flow reentering the river from the Yolo Bypass through Cache Slough and Steamboat Slough, just upstream from this location.

- **SPFC Approach** – Higher stages than the No Project condition and the other approaches through the 0.5 percent AEP event because restoration of all SPFC levees reduces the number of levee breaks upstream from this location, which increases the amount of inflow to the Yolo Bypass. However, for the 0.2 percent AEP event, levees break along the Yolo bypass as a result of the increased stage.

- **EFSC Approach –** Improvements in the Yolo Bypass as well as rehabilitation of upstream levees result in higher flows from the bypass into the river for the 0.2 percent AEP flood event.

- **SSIA –** inflows to the Yolo Bypass are high, but levee breaks occur in the bypass in the 0.2 percent AEP flood event, resulting in decreased flow and stage at this location.



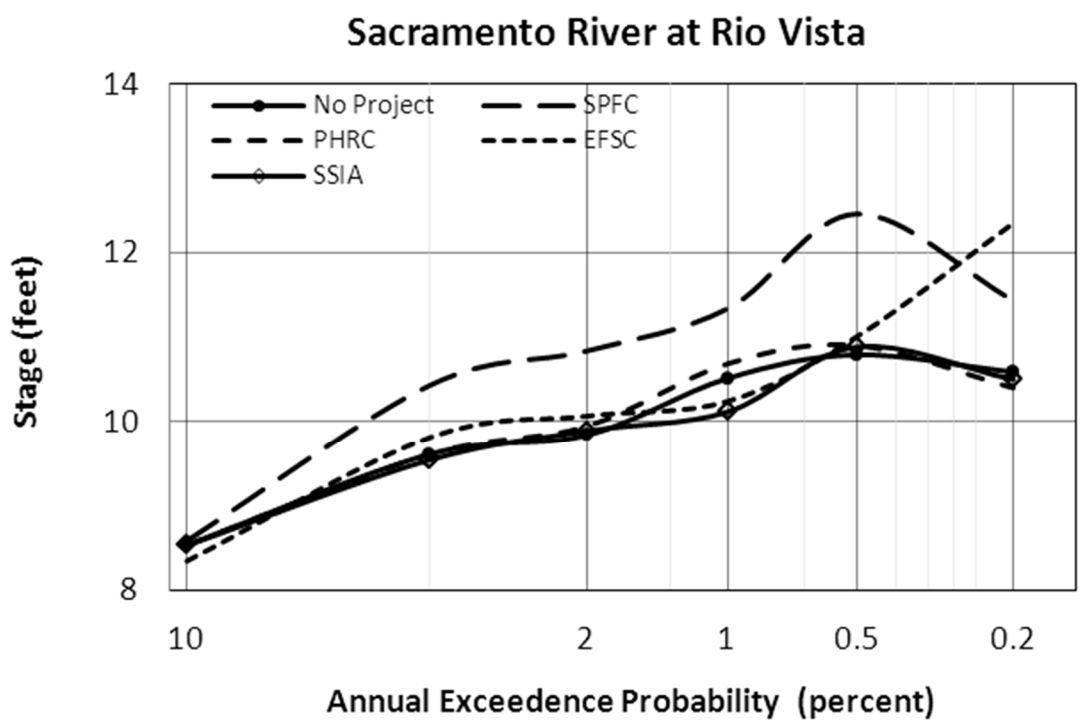


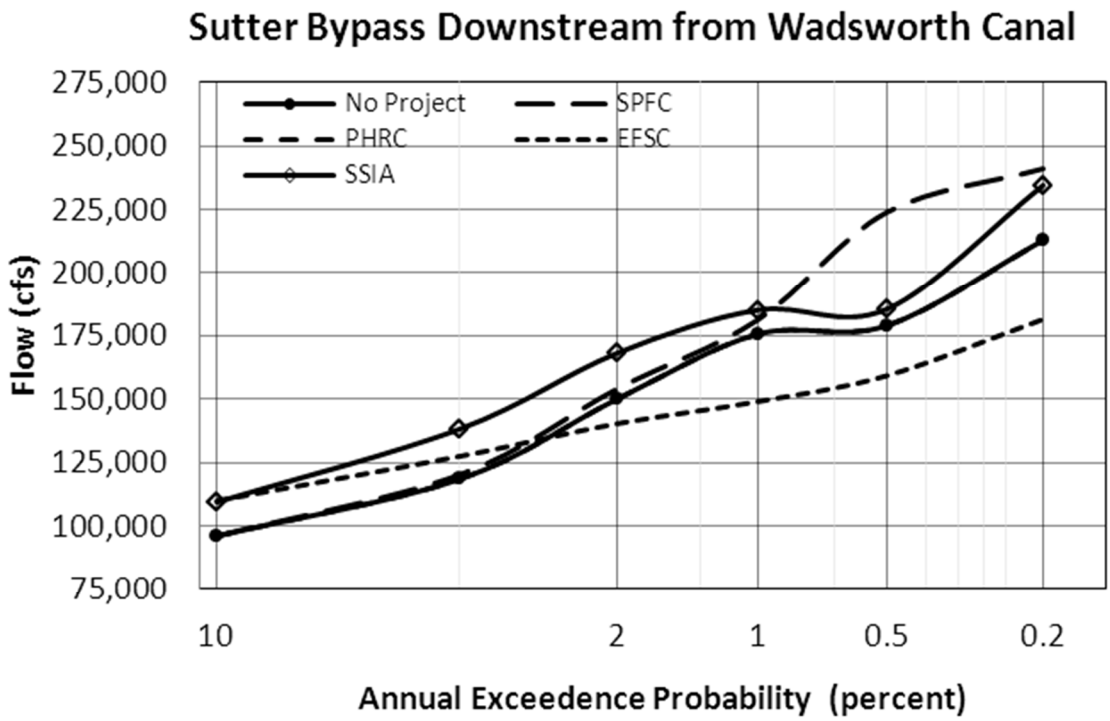
Figure 4-9. Flow- and Stage-Frequency Curves: Sacramento River at Rio Vista [7]

- **No Project Condition and PHRC Approach** – Stage remains relatively constant for the 1 percent AEP event and greater as a result of upstream levee breaks along the Sutter Bypass.

- **SPFC Approach** – Restoring SPFC levees reduces the number of upstream levee breaks in the bypass and along the Sacramento River, so more flow is retained in the channels and stage is increased compared to the No Project condition. For the 0.2 percent AEP event, levees break upstream from this location, so there is relatively little increase in stage and flow.

- **EFSC Approach** – Flow and stage are significantly reduced compared to the No Project condition as a result of the Sutter Butte Basin floodplain storage area, which is immediately upstream from this location and diverts a large portion of the bypass flow, especially for large flood events.

- **SSIA** – Flows are generally higher than in the No Project condition as a result of the addition of the Feather to Butte Basin (Biggs) bypass, which conveys flow to the Sutter Bypass through Cherokee Canal and Butte Creek. However, stages are generally similar to the No Project condition as a result of bypass widening, which increases conveyance capacity for any given stage.



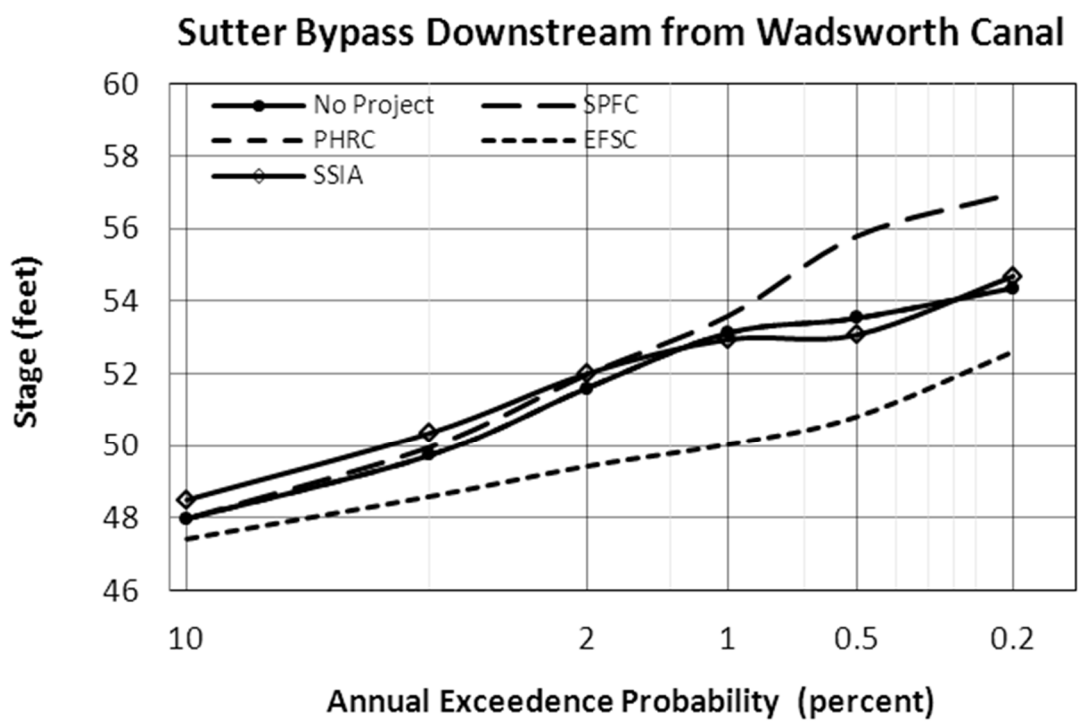


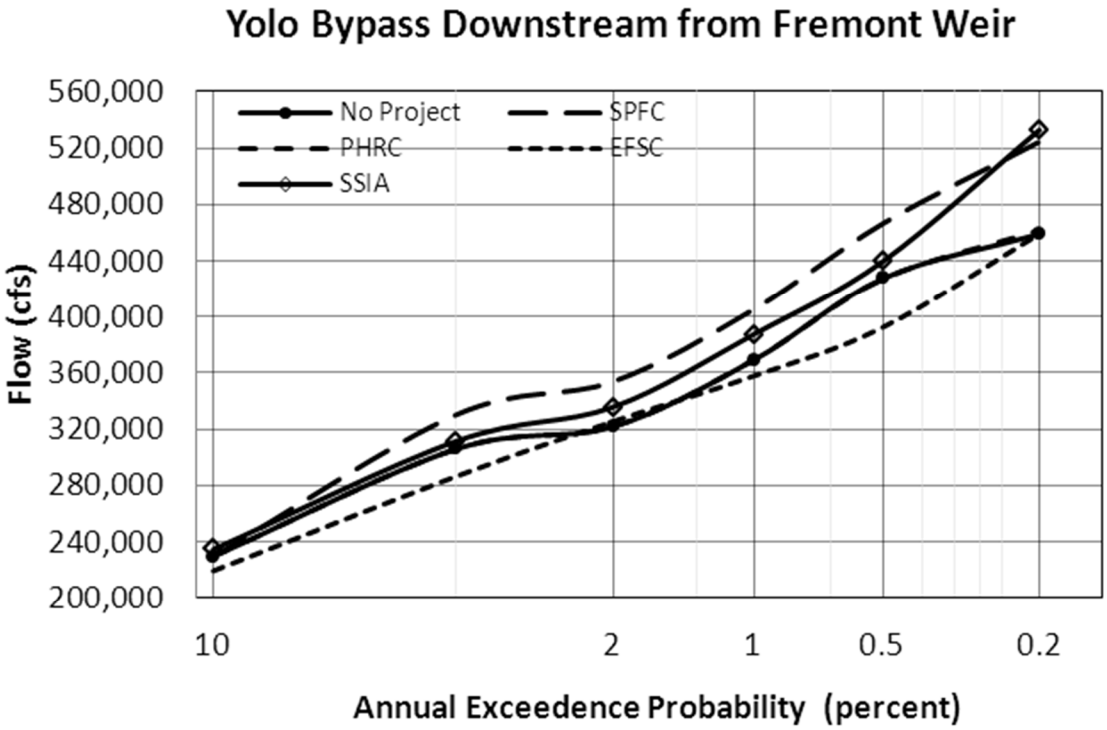
Figure 4-10. Flow- and Stage-Frequency Curves: Sutter Bypass Downstream from Wadsworth Canal [8]

- **No Project Condition and PHRC Approach** – Levee breaks in the Sutter Bypass upstream from the Fremont weir in the 0.2 percent AEP event cause relatively little increase in flow and stage compared to the 0.5 percent AEP event.

- **SPFC Approach** – Restoring SPFC levees reduces the number of upstream levee breaks, resulting in a higher stage at Fremont Weir and a higher flow rate into the Yolo Bypass over the weir compared to the No Project condition for all events.

- **EFSC Approach** – Stage in the Yolo Bypass below the Fremont Weir is generally lower than in the No Project condition as a result of bypass widening. Through the 0.5 percent AEP event, flows in the bypass are decreased by a number of upstream flood management actions, including floodplain storage and modified reservoir operations. However, for the 0.2 percent AEP event, stage is higher than the No Project Condition, while flow is approximately equal, because water stored in the floodplain storage area along the Sacramento River below the Feather River overflows into the Yolo Bypass. These inflows increase the backwater effect at the Fremont Weir, resulting in increased stage and decreased flow over the weir.

- **SSIA** – Widening of the Yolo Bypass results in a lower stage below the Fremont Weir for all events. However, flow is greater than in the No Project condition because the lower stage results in a decreased backwater effect, which allows more flow over the weir, and because there is more inflow to the Sutter Bypass from upstream weirs (also resulting from lower stage in the bypass) and from the addition of the Feather to Butte Basin (Biggs) Bypass.



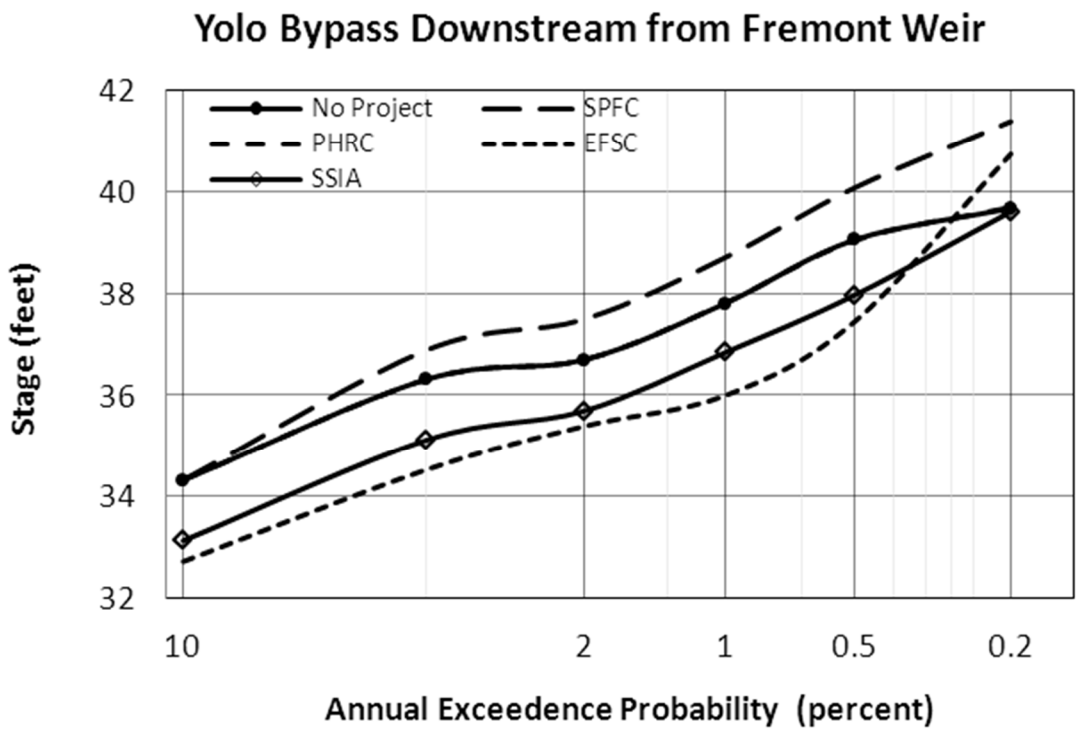


Figure 4-11. Flow- and Stage-Frequency Curves: Yolo Bypass Downstream from Fremont Weir [9]

- **SPFC Approach –** Results in higher stages at this location than the No Project condition and the other approaches through the 0.5 percent AEP event because restoration of all SPFC levees reduces the number of levee breaks upstream on the Sacramento River. This increases river stage, which causes more flow over the both weirs that control inflow to the bypass. However, at the 0.2 percent AEP event, the higher stages in the bypass result in levee breaks in the bypass upstream from this location, lowering the flow compared to the EFSC Approach and SSIA.

- **EFSC Approach** – Maximum flow and stage in the bypass is increased at the 0.2 percent AEP event as a result of upstream levee improvements, which increases the inflow to the bypass, as well as widening of the bypass, which increases its maximum capacity.

- **SSIA –** Flow in the bypass for the 0.2 percent AEP event is greater than in the No Project while stage is similar because widening the bypass increases its conveyance capacity at any given stage. However, levee breaks upstream from this location limit the inflow to the bypass compared to the SPFC and EFSC approaches.

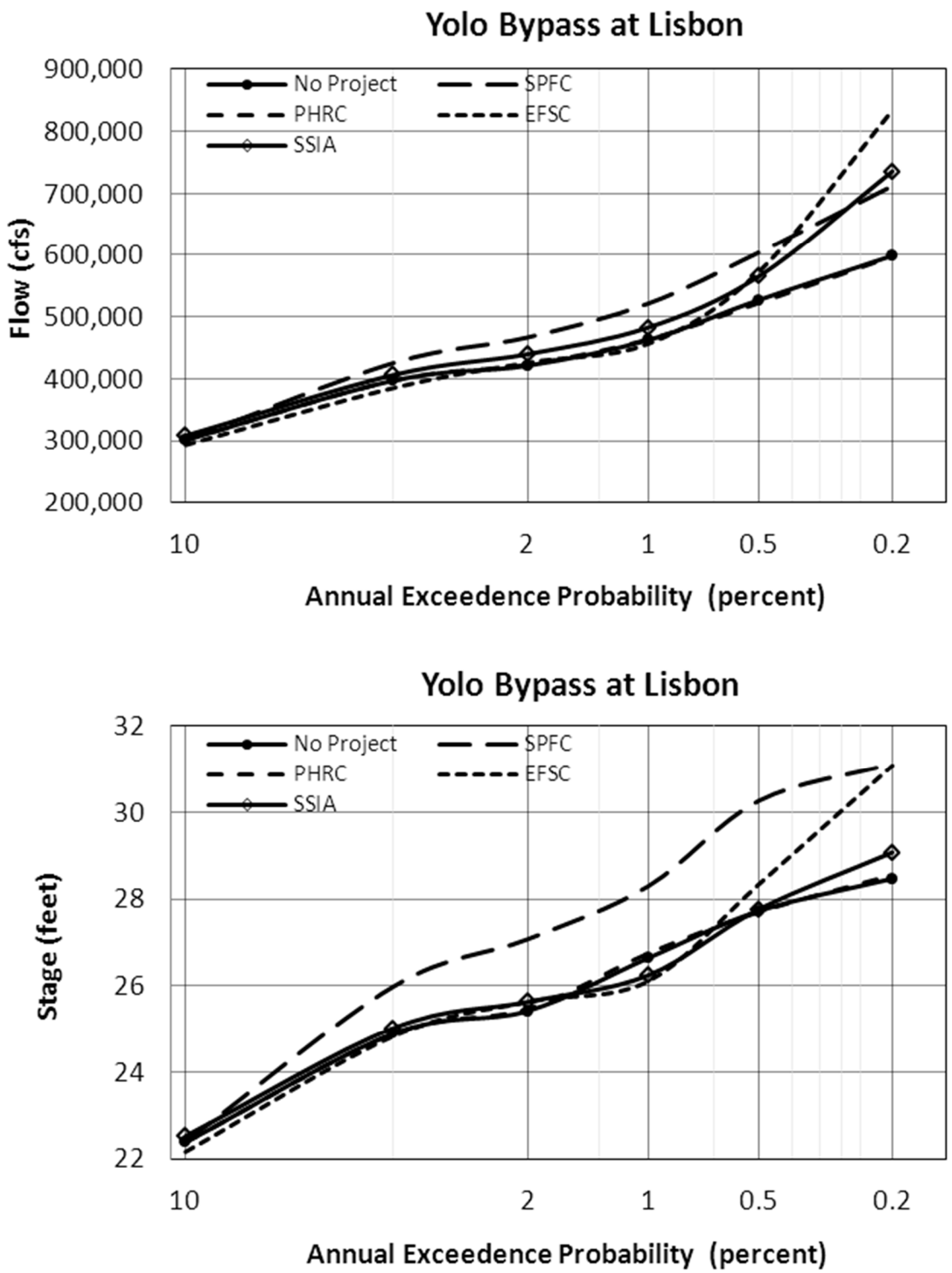
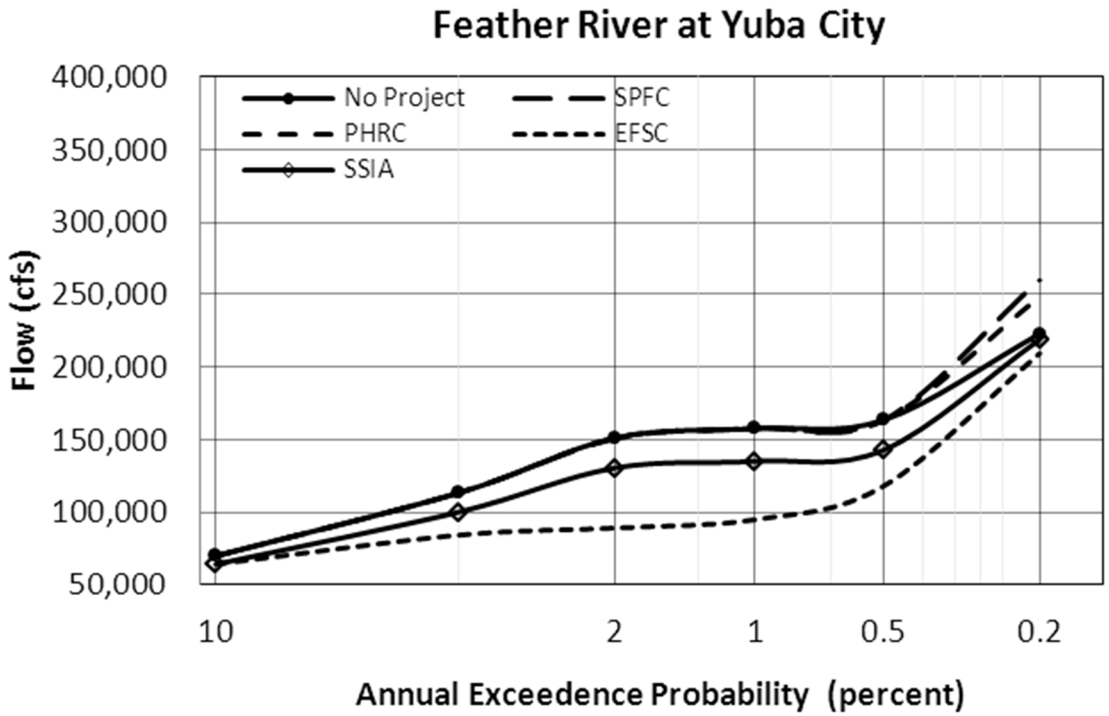


Figure 4-12. Flow- and Stage-Frequency Curves: Yolo Bypass at Lisbon [10]

- **SPFC and PHRC Approaches** – Flow and stage are higher than the No Project condition for the 0.2 percent AEP event because levee rehabilitation decreases the number of upstream levee breaks along the Feather River and more flows remain in the river channel.

- **EFSC Approach –** Increased flood management storage in Lake Oroville and Feather-Sutter Bypass reduce peak stage and flow for all events.

- **SSIA** – The Feather to Butte Basin (Biggs) Bypass diverts flow from the Feather River immediately downstream from Lake Oroville, which reduces flow and stage at this location compared to the No Project condition.



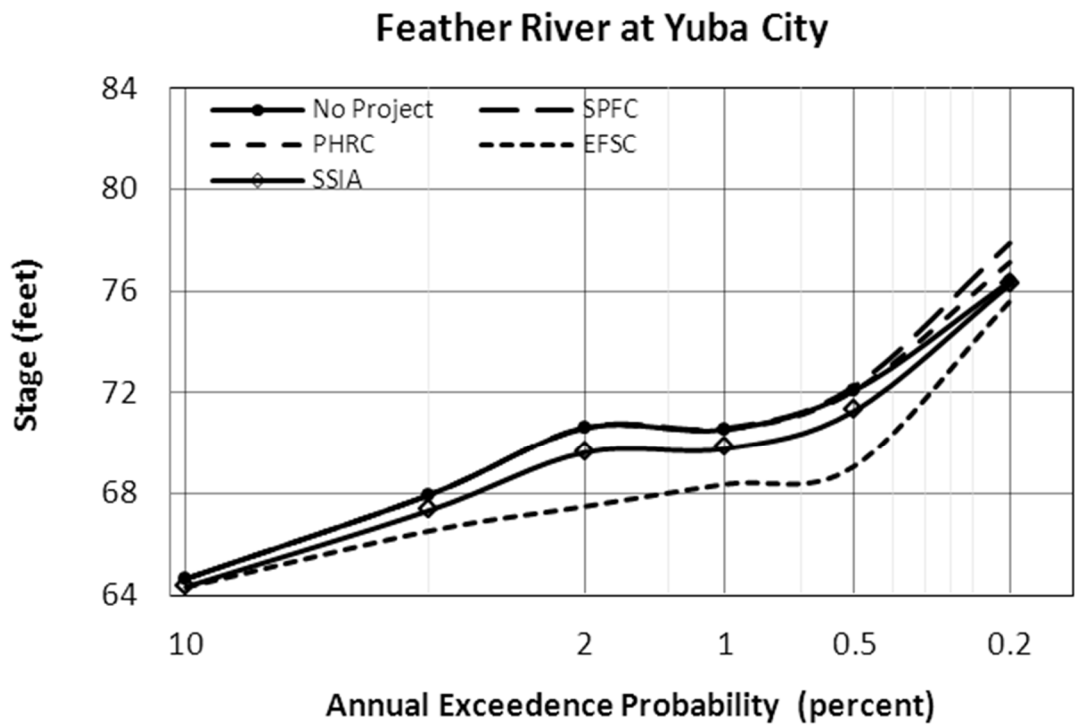


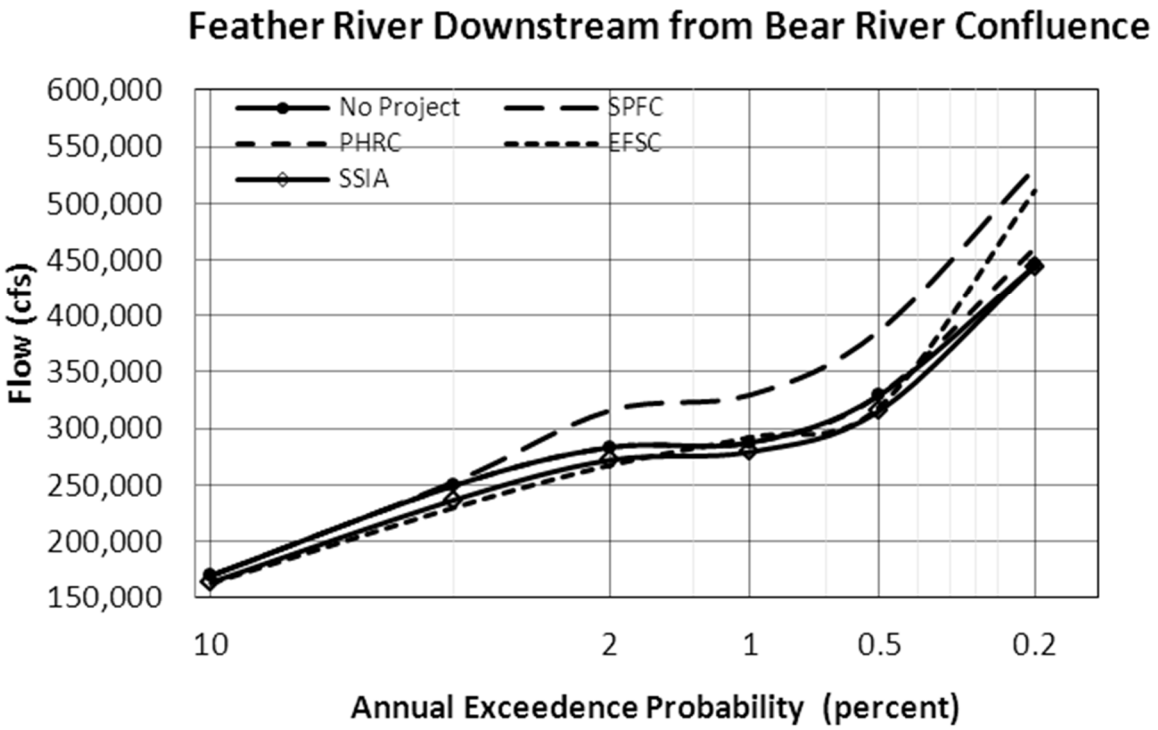
Figure 4-13. Flow- and Stage-Frequency Curves: Feather River at Yuba City [11]

- **No Project Condition and PHRC Approach** – Levee breaks upstream from this location cause the flow and stage to be approximately equal for both cases.

- **SPFC Approach** – Restoring SPFC levees reduces the number of upstream levee breaks along the Feather and Bear Rivers, retaining more in-channel flow compared to the No Project condition and increasing flow and stage.

- **EFSC Approach** – Restoring SPFC levees reduces the number of levee breaks along the Bear River and increases in-channel flow. The increased flow from the Bear is more than offset by the increased flood management storage in Lake Oroville and diversion of flows from the upper Feather River through Feather to Butte Basin (Biggs) Bypass. Peak flows are generally similar to the No Project condition up through the 1 percent AEP event. For larger flood events, the rehabilitated levees prevent significant outflows from levee breaks and greatly increase peak flows compared to the No Project condition.

- **SSIA** – Outflow from the Feather River through the Feather to Butte Basin (Biggs) Bypass causes river flow and stage to be lower than the No Project condition for all flood events



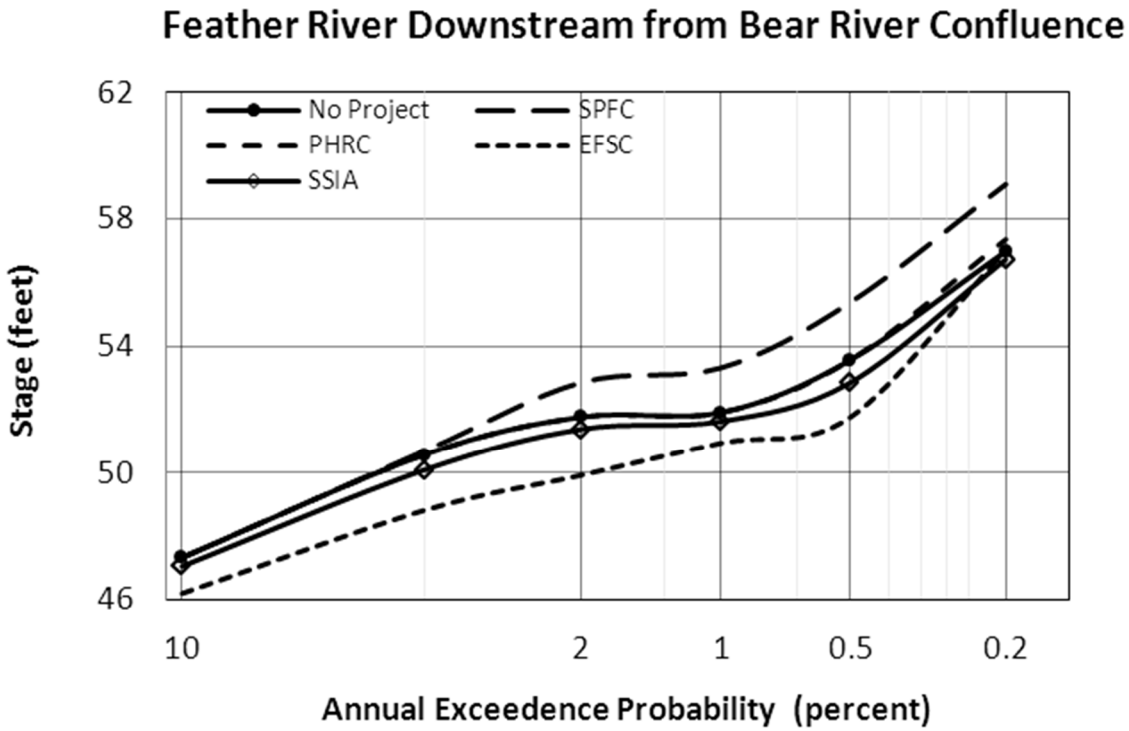
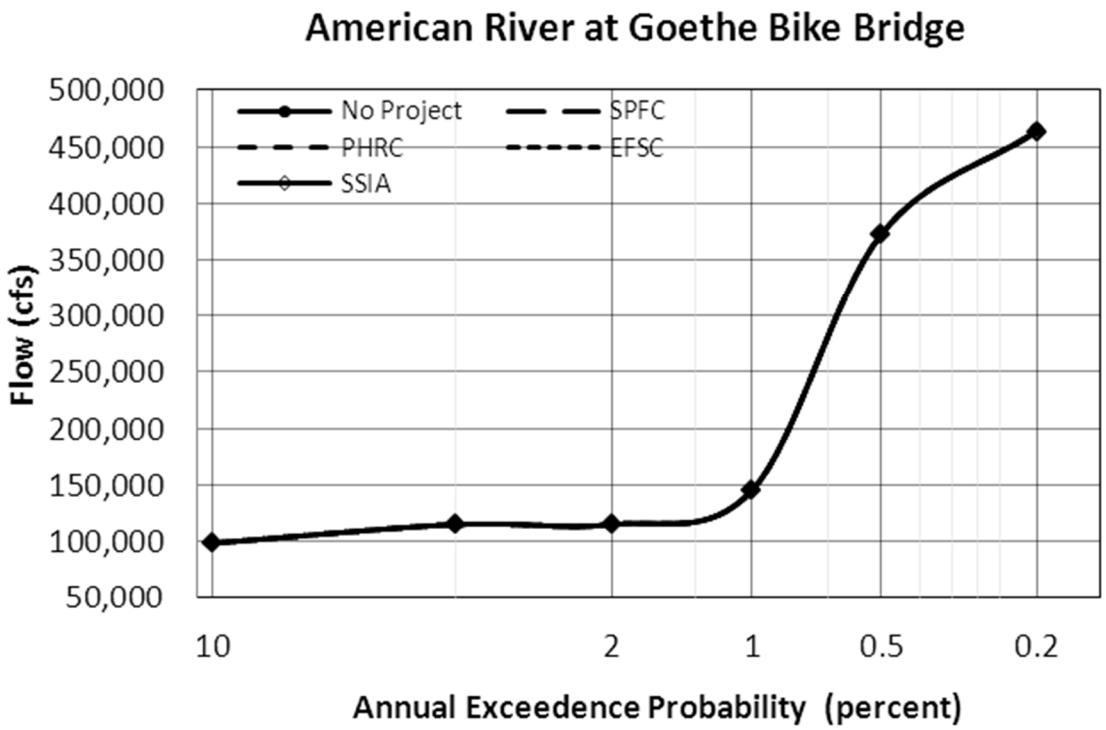


Figure 4-14. Flow- and Stage-Frequency Curves: Feather River Downstream from Bear River Confluence [12]

- **No Project Condition and All Approaches –** No modifications to the flood control system upstream from this location, so flows and stages are similar for the No Project condition and all approaches. Inflows remain relatively constant through the 1 percent AEP event, as a result of upstream reservoir flood management. At the 0.2 percent ARP event and greater, flows cause upstream levee breaks, but some of the flow in the floodplain returns to the channel upstream from this location.



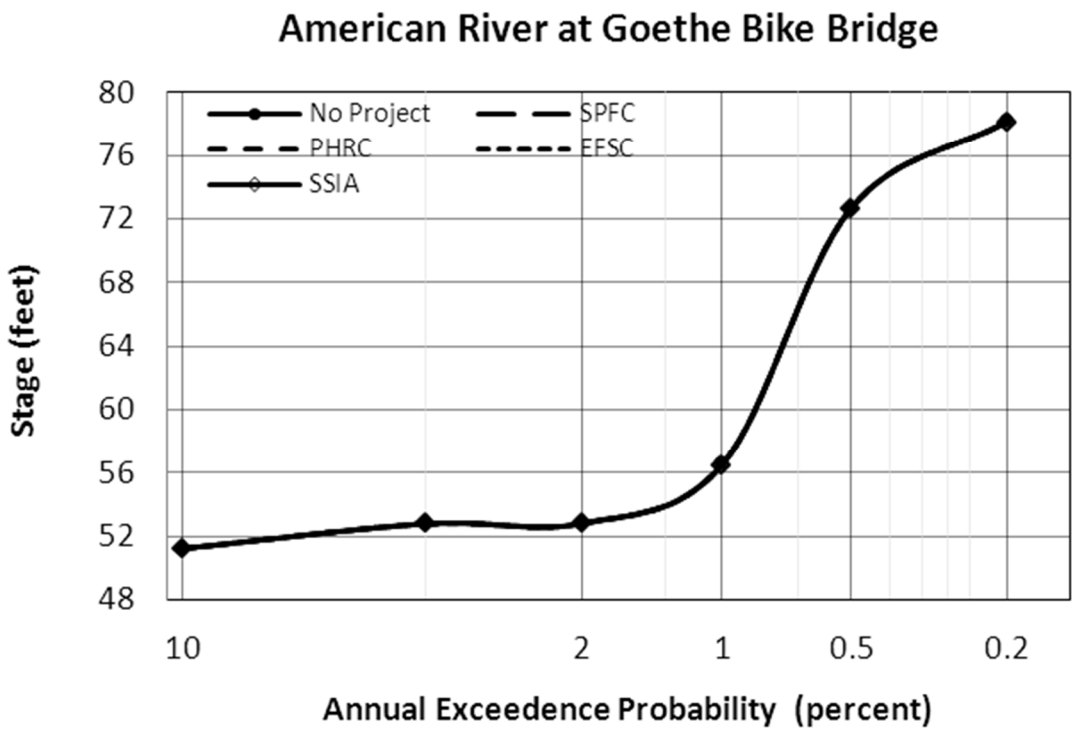


Figure 4-15. Flow- and Stage-Frequency Curves: American River at Goethe Bike Bridge [13]

Table 4-1. Sacramento River Basin Simulated Out-of-System Volumes by Return Period for No Project Condition, and Change in Out-of-System Volumes for CVFPP Approaches

Approach	Return Period (years)	Flood Zones															
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
No Project Volume (acre-feet)	10	7	0	0	0	0	0	0	0	14	0	0	0	0	0	0	0
	25	0	0	0	0	0	37	198	0	18	0	0	0	0	13	0	166
	50	24	0	0	0	0	54	264	0	21	0	91	0	0	19	0	287
	100	36	0	0	0	0	65	355	56	24	0	174	0	0	22	0	354
	200	61	0	0	478	0	349	402	100	450	0	244	0	0	25	0	429
	500	89	19	120	893	929	405	407	94	659	181	177	538	182	121	0	787
Achieve SPFC Design Flow Capacity Volume (acre-feet)	10	0	0	0	0	0	0	0	0	-14	0	0	0	0	0	0	0
	25	0	0	0	0	0	-37	-104	0	-18	0	0	0	0	-13	0	-166
	50	-24	0	0	0	0	-54	-78	0	-21	0	-91	0	0	-19	0	-287
	100	-36	0	0	0	0	-65	59	-56	-24	0	-174	0	0	-22	0	-287
	200	-61	0	0	-478	0	-18	92	-31	-425	0	-244	0	0	-25	0	-325
	500	-89	-19	-120	-127	-929	157	86	89	140	34	-90	33	-182	255	0	-423
Protect High Risk Communities Volume (acre-feet)	10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	50	0	0	0	0	0	0	4	0	0	0	-91	0	0	0	0	0
	100	0	0	0	0	0	1	9	1	0	0	-174	0	0	0	0	0
	200	0	0	0	0	0	0	3	1	234	0	-244	0	0	0	0	0
	500	0	0	-1	-1	39	10	35	28	208	-167	71	-538	11	141	35	-134
Enhance Flood System Capacity Volume (acre-feet)	10	0	0	0	0	0	0	0	0	-14	0	0	0	0	0	0	0
	25	0	0	0	0	0	-37	-143	0	-18	0	0	0	3	-13	0	-166
	50	0	0	0	0	0	-54	-190	0	-21	0	-91	0	4	-9	0	-287
	100	0	0	0	0	0	-65	-240	-56	-24	0	-174	0	4	-8	0	-354
	200	0	0	0	-478	0	-349	2	-100	-450	0	-244	0	3	-13	0	-371
	500	0	-19	-120	-893	-929	76	84	85	-271	-167	-177	-538	-182	-121	0	-509
State Systemwide Investment Approach Volume (acre-feet)	10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	25	0	0	0	0	0	1	2	0	0	0	0	0	2	0	0	-17
	50	0	0	0	0	0	2	4	0	0	0	-91	0	0	0	0	-42
	100	0	0	0	444	0	-6	-52	-4	-1	0	-174	0	0	0	0	-74
	200	0	0	0	26	0	-277	-11	-5	43	0	-244	0	0	0	0	-65
	500	0	1	300	85	-56	-73	31	205	-23	-167	66	-538	-12	44	0	-281

Notes: 10 percent AEP = 10-year return period
2 percent AEP = 50-year return period
0.5 percent AEP = 200-year return period
4 percent AEP = 25-year return period
1 percent AEP = 100-year return period
0.2 percent AEP = 500-year return period

Table 4-2. Sacramento River Basin Simulated Flow Volume into Delta

Approach	Flow Volume Entering Delta (TAF)*											
	10% AEP (10-year)		4% AEP (25-year)		2% AEP (50-year)		1% AEP (100-year)		0.5% AEP (200-year)		0.2% AEP (500-year)	
	Volume	Volume Change**	Volume	Volume Change**	Volume	Volume Change**	Volume	Volume Change**	Volume	Volume Change**	Volume	Volume Change**
No Project Condition	2,602	-	3,385	-	3,785	-	4,167	-	4,557	-	4,780	-
Achieve SPFC Design Flow Capacity Approach	2,602	0	3,506	121	3,979	195	4,436	270	5,015	459	5,513	733
Protect High Risk Communities Approach	2,602	0	3,385	0	3,782	-3	4,161	-5	4,554	-3	4,899	120
Enhance Flood System Capacity Approach	2,507	-95	3,249	-136	3,647	-138	3,974	-193	4,625	69	5,498	718
State Systemwide Investment Approach	2,601	-1	3,388	3	3,813	28	4,113	-54	4,634	78	4,986	206

Notes:
* based on the sum of volume of Sacramento River downstream from American River and Yolo Bypass at Lisbon during 1/18 -1/21
** Volume Change (TAF) is the difference between each approach and the No Project Condition
Key:
AEP = annual exceedence probability
SPFC = State Plan of Flood Control
TAF = Thousand acre-feet

- **No Project Condition, PHRC approach, and SSIA** – Levee breaks occur on Paradise Cut upstream from this location at the 0.5 percent AEP event, allowing a large amount of flow to leave the San Joaquin River, which reduces flow and stage compared to other approaches (for the 0.2 percent AEP event, the same levee breaks occur in other approaches). For the 0.2 percent AEP event, there are also levee breaks just downstream from this location, which result in a higher peak flow rate compared to other approaches, without a significant increase in stage relative to the other approaches.

- **SPFC Approach** – Restoring SPFC levees reduces the number of upstream levee breaks for all events compared to the No Project condition, so peak flows and stages tend to be higher. For the 0.2 percent AEP event, levee restoration also prevents a levee break downstream from this location, so the flow is slightly reduced compared to the No Project condition despite a higher stage.

- **EFSC Approach** – Restoring SPFC levees reduces upstream levee failures similar to the SPFC approach. Flows and stages at this location are generally similar to or lower than the No Project condition and other approaches due to increased upstream reservoir storage and floodplain storage areas, which tend to reduce peak flows and stages. At the 0.5 percent AEP flood, levee restoration reduces the number of levee breaks immediately upstream from this location (both along the San Joaquin River and in Paradise Cut) compared to the No Project condition, so flows and stages are higher. Similar to the SPFC approach, at the 0.2 percent AEP event levee restoration prevents a significant levee break downstream from this location, so the flow is slightly reduced compared to the No Project condition.

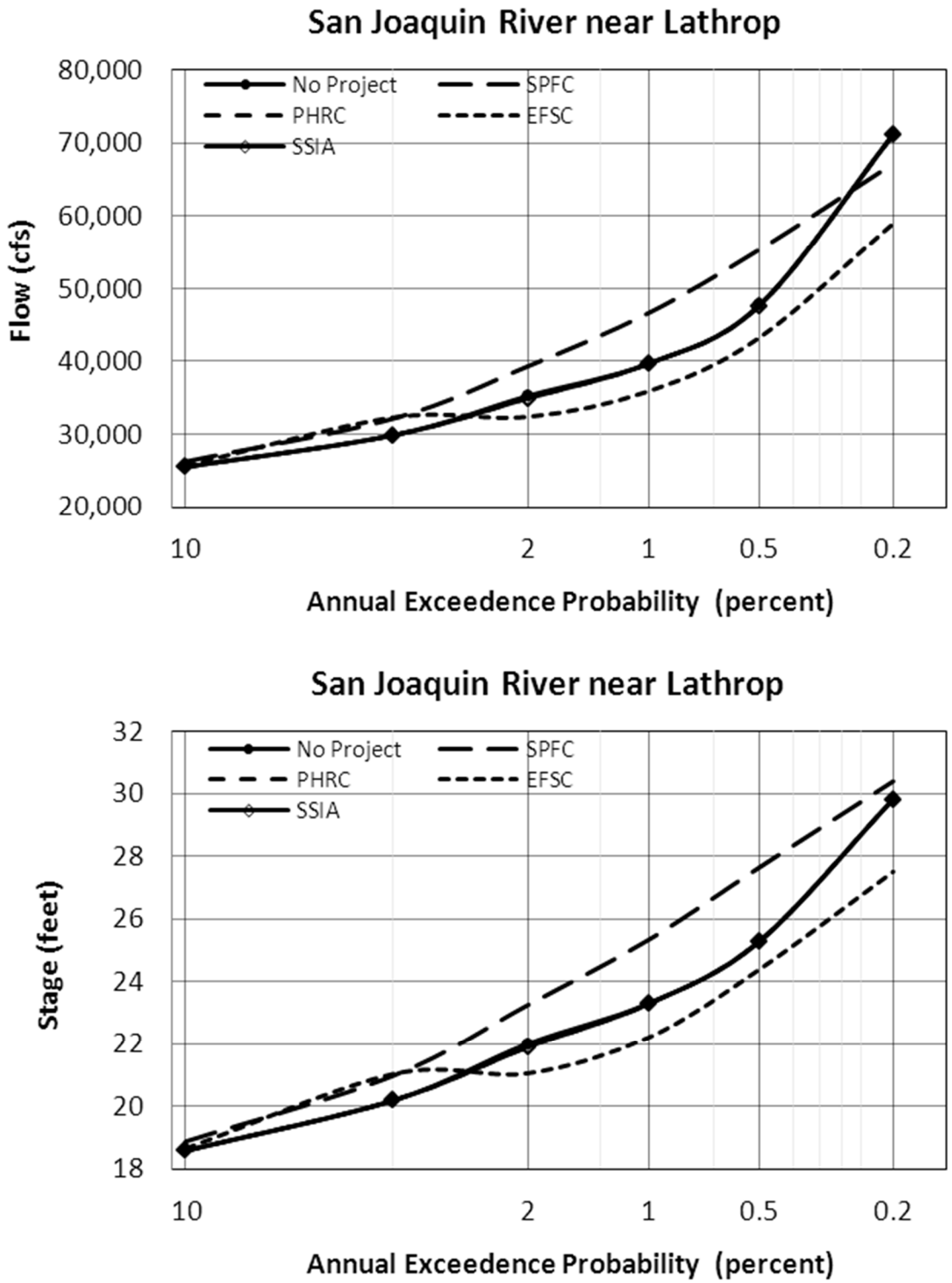


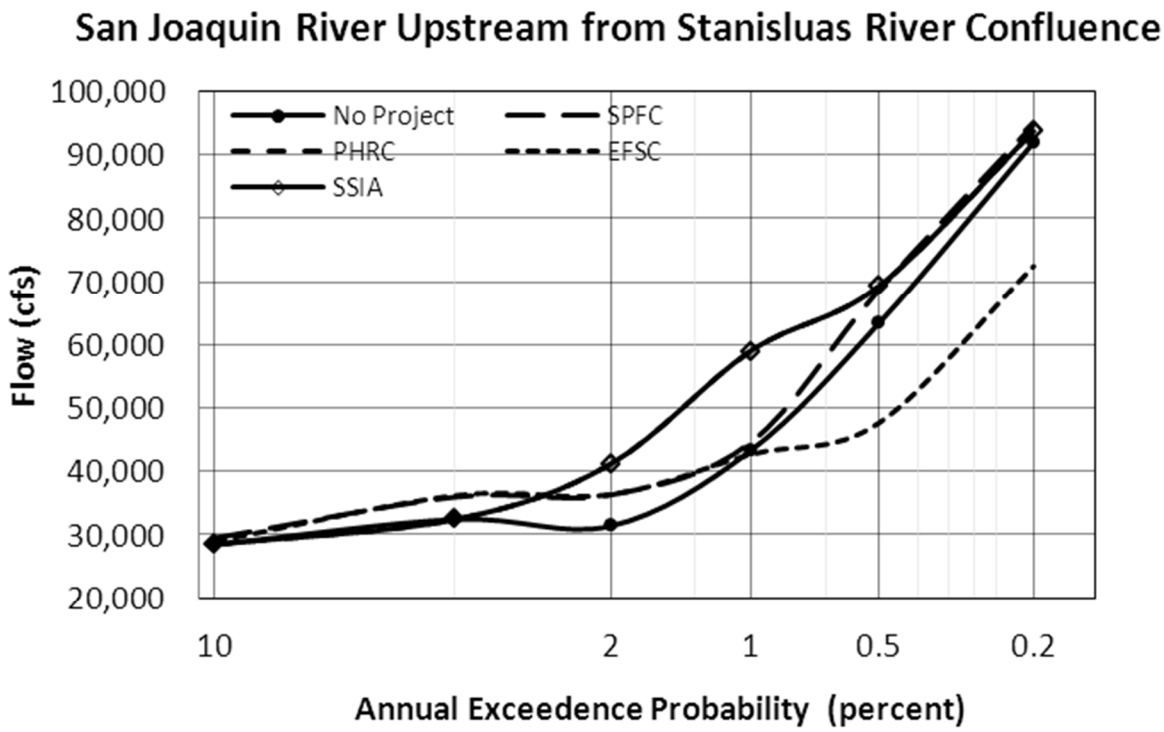
Figure 4-16. Flow- and Stage-Frequency Curves: San Joaquin River Near Lathrop [1]

- **No Project Condition** – Significant levee breaks occur upstream from this location for all floods larger than the 4 percent AEP event. At the 2 percent AEP event, stage continues to rise despite decreasing flows because of the backwater effect from the inflows from the Stanislaus River immediately downstream from this location. For larger events, inflows from the Tuolumne River upstream from this location and the Stanislaus River downstream from this location cause significant increases in flow and stage despite levee breaks along the San Joaquin River.

- **SPFC Approach** – Restoring SPFC levees reduces the number of upstream levee breaches, increasing stage and flow compared to the No Project condition for all events. Levee restoration also prevents a levee break immediately downstream from this location through the 1 percent AEP event, increasing downstream river stage compared to the No Project condition and all other approaches. The resulting backwater effect increases the peak stage for the SPFC approach for these events. At the 0.5 percent AEP event and greater, this levee breaches, so peak stage converges with the No Project condition, but flows continue to increase.

- **PHRC Approach and SSIA** – Levee restoration prevents levee breaches along the Tuolumne River through the 1 percent AEP flood, and as a result, the flow in the San Joaquin River between the Tuolumne and Stanislaus river confluences is greatest for these approaches. However, there is no appreciable difference in stage compared to the No Project condition because levees immediately downstream from this location break in the same location. For the 0.5 percent and 0.2 AEP events, levees along the Tuolumne River fail, so flows tend to converge with the No Project and SPFC approaches.

- **EFSC Approach** – Restoring SPFC levees reduces the number of upstream levee breaks compared to the No Project condition, increasing peak flow through the 1 percent AEP event. However, at the 0.5 percent and 0.2 percent AEP floods, the combined effects of an increased flood management pool in New Don Pedro Reservoir and transitory storage areas along the Tuolumne and San Joaquin Rivers act to keep flows lower than the No Project condition and all other approaches. River stage is lower than the No Project condition for all flood events as a result of levee setbacks along the San Joaquin River, which increase the conveyance capacity at any given stage.



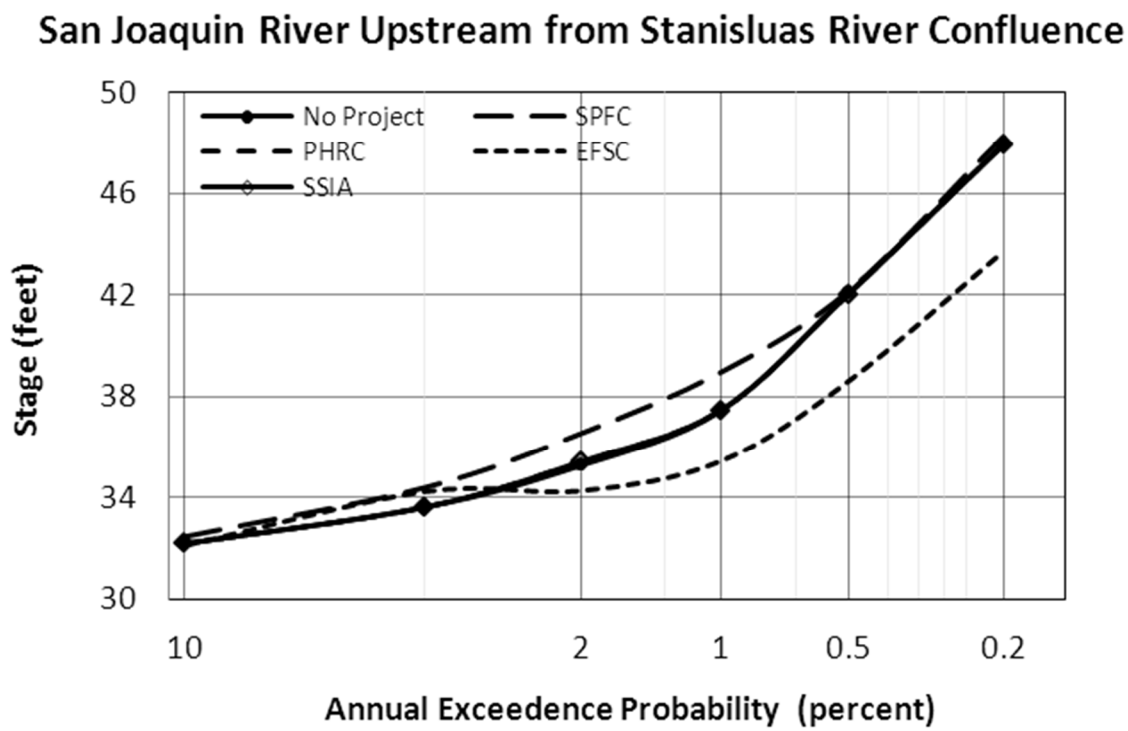


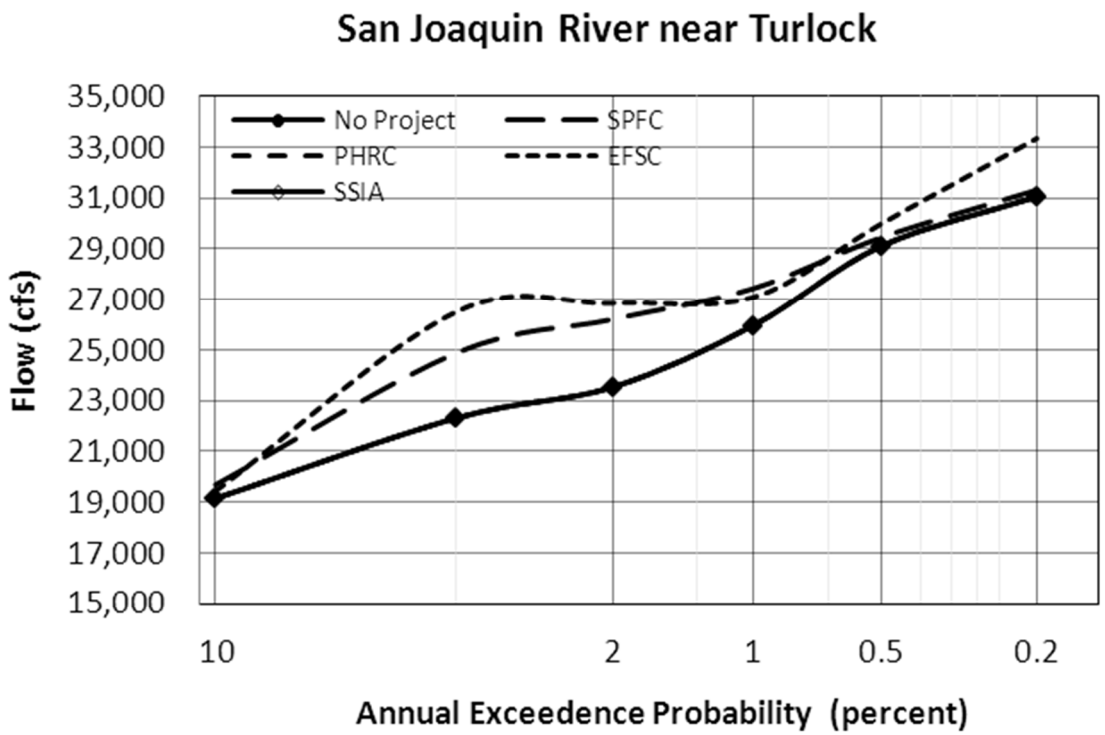
Figure 4-17. Flow- and Stage-Frequency Curves: San Joaquin River Upstream from Stanislaus River Confluence [2]

- **No Project Condition, PHRC Approach, and SSIA** – No modifications to the flood management system upstream from this location, and significant upstream levee breaks occur for all events larger than the 10 percent AEP flood. Levee breaks release flow into adjacent floodplains, so in channel flows and river stages are lower than for approaches with strengthened levees for all flood events. For large flood events (0.1 percent AEP and larger), the magnitude of Merced River inflows (which mostly enter from the surrounding floodplain) is much larger than San Joaquin River flow, and tends to cause flows to increase significantly, converging with SPFC and EFSC flows.

- **SPFC Approach** – Restoring SPFC levees reduces the number of upstream levee breaks through the 1 percent AEP event, increasing in-channel flow and stage compared to the No Project condition. At the 0.5 percent AEP event and greater, inflows from the Merced River (which mostly enter from the surrounding floodplain) tend to cause flows to converge with the No Project condition.

- **EFSC Approach** – Restoring SPFC levees results in higher flows and stages compared to the No Project condition in most cases. In-channel peak flows are higher and stages are lower than in No Project Condition and all other approaches for the 0.2 percent AEP because there is significantly less flow entering the San Joaquin River from the Tuolumne River downstream from this location, as a result of floodplain storage areas along the Tuolumne River and an increased flood management pool at New Don Pedro Reservoir. The lower downstream flow leads to a reduced backwater effect, which travels upstream to this location and tends to reduce river stage while also allowing for faster flows and higher flow rates.

  Peak inflows to the Merced River are reduced in the EFSC approach for all events larger than the 4 percent AEP flood event by the increased flood pool in New Exchequer Dam. However, levee breaks occur along the Merced River for the No Project condition and all approaches except EFSC for the 2 percent AEP flood and greater, releasing significant amounts of flow to the surrounding floodplain, such that flows in the Merced River at the San Joaquin River confluence are approximately equal for all approaches. Much of this flow re-enters the San Joaquin River through levee breaches along the San Joaquin River.



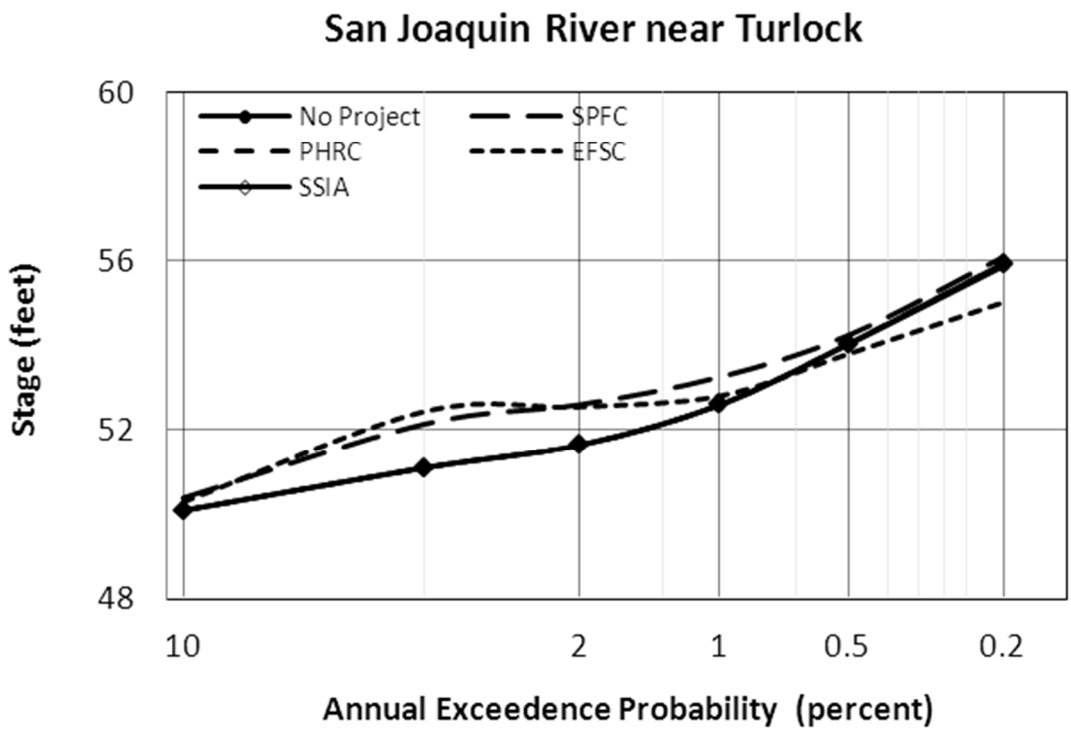
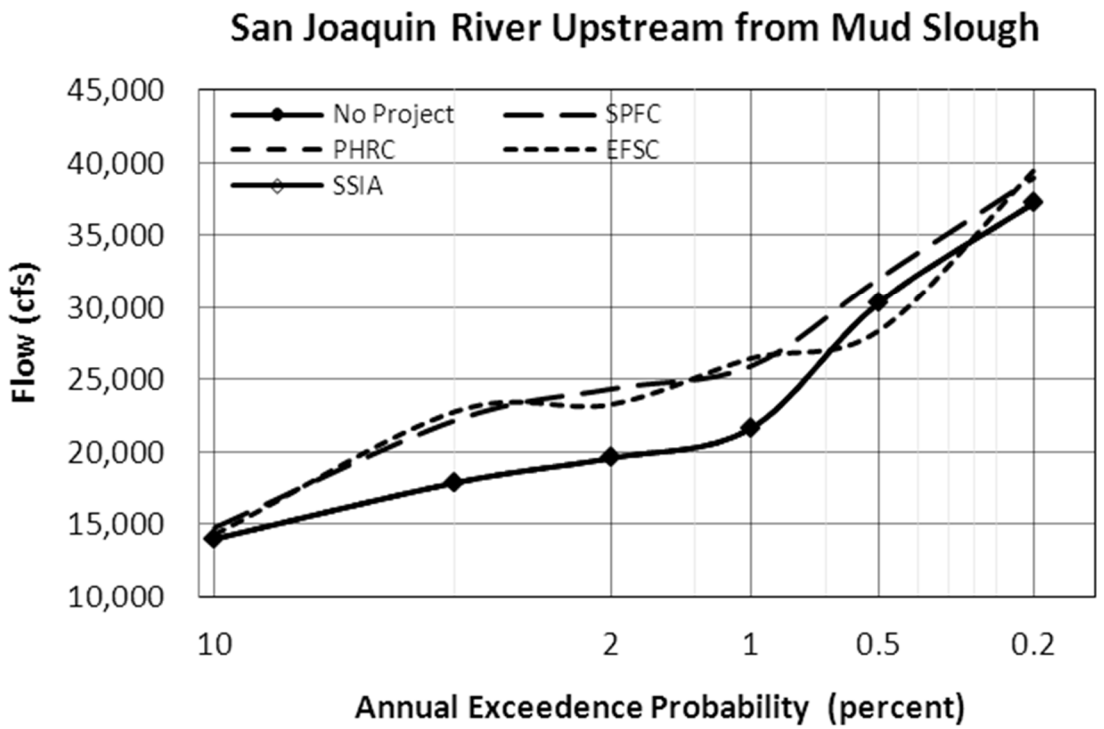


Figure 4-18. Flow- and Stage-Frequency Curves: San Joaquin River near Turlock [3]

- **No Project Condition, PHRC Approach, and SSIA –** No modifications to the flood management system upstream from this location, and significant levee breaks occur for all events larger than the 10 percent AEP flood, both along the San Joaquin River and along the Chowchilla/Eastside Bypass. Levee breaks release flow into adjacent floodplains, so in channel flows and river stages are generally lower than for approaches with strengthened levees. However, for larger flood events (0.5 percent AEP and larger), there are significant inflows to the San Joaquin River from the surrounding floodplain upstream from this location, significantly increasing river flows and stages. The majority of these inflows originate from the Merced River.

- **SPFC Approach** – Restoring SPFC levees reduces the number of upstream levee breaks for all events, increasing in-channel flow and stage compared to the No Project condition. However, for the 0.5 percent AEP event and larger events, overflows from the Merced River enter the San Joaquin River just upstream from this location, as in the No Project condition, so flow and stage tend to converge with those in the No Project condition at the 0.5 percent AEP event.

- **EFSC Approach** – As for the SPFC approach, restoration of SPFC levees results in higher flows and stages compared to the No Project condition in most flood events. For the 0.5 percent AEP flood, flows are lower than in the No Project condition and all other approaches because peak flows along the Merced River are reduced as a result of the increased flood pool at New Exchequer Dam, levee breaks along the Merced River occur later in the storm, which reduces the volume of flows into the floodplain area around the San Joaquin River, thereby, reducing the stage in the floodplain, resulting in reduced outflow from the floodplain to the river. At the 0.2 percent AEP event, flows overwhelm levees along the Merced River, so flow and stage is similar to the No Project condition.



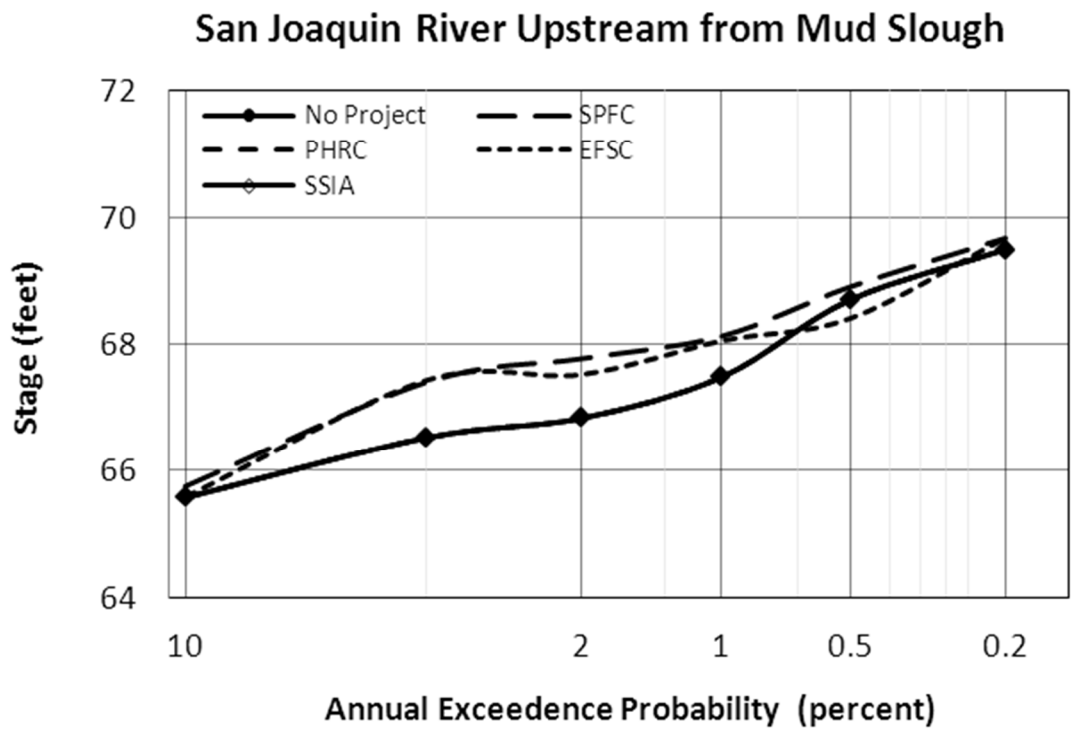


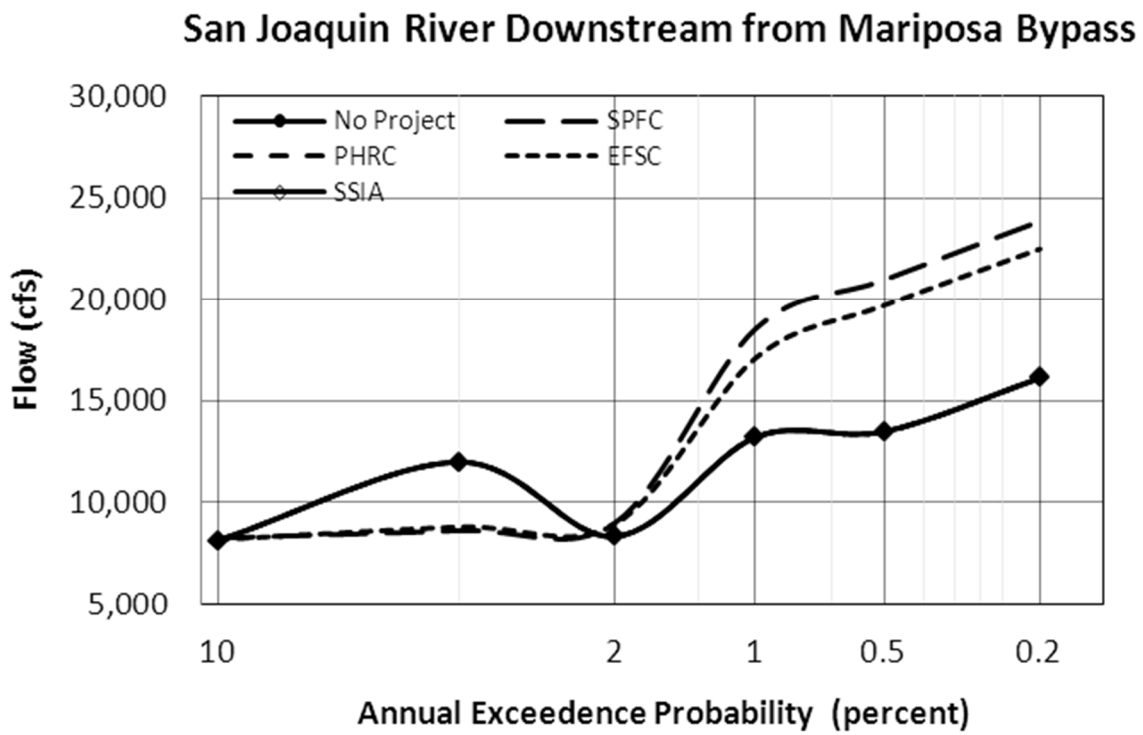
Figure 4-19. Flow- and Stage-Frequency Curves: San Joaquin River Upstream from Mud Slough [4]

- **No Project Condition, PHRC Approach, and SSIA –** Existing levees along the Chowchilla Bypass fail for all events greater than the 10 percent AEP flood, releasing a portion of the Chowchilla Bypass flow into the San Joaquin River upstream from the Mariposa Bypass. However, significant levee breaks also occur along the San Joaquin River for the 1 percent AEP event with these approaches, significantly reducing in-channel flow and stage compared to the 2 percent AEP event. In-channel flows for larger events increase only moderately because of the levee failures.

- **SPFC Approach** – Restored SPFC levees contain flow in the Chowchilla Bypass through the 4 percent AEP event, reducing San Joaquin River flows. At the 2 percent AEP event and greater those levees fail, releasing a portion of the bypass flow into the San Joaquin River upstream from the Mariposa Bypass. However, because there are fewer upstream levee breaks compared to the No Project condition, a larger volume of flow is available in the Chowchilla Bypass when levees fail, so higher flows are released into the San Joaquin River compared to the No Project condition. Improved levees along the San Joaquin River also maintain higher in-channel flows, breaking only at the 0.2 percent AEP event.

- **EFSC Approach** – Increased flood management pool at Friant Dam and restored SPFC levees allow Chowchilla and Mariposa Bypass flows to be managed through the 2 percent AEP flood event. Above the 2 percent AEP event, Chowchilla bypass levees break and release flow into the San Joaquin River. As in the SPFC approach, higher flows in the Chowchilla bypass at the time of the levee break lead to greater flows into the San Joaquin River compared to the No Project condition. Improved levees along the San Joaquin River maintain higher in-channel flows in the river compared to the No Project condition.

  River stage is consistently lower for the EFSC approach than for all other approaches despite higher flow rates at the 1 percent AEP event and greater because, when Chowchilla levees fail and release water into the floodplain the bypass and the San Joaquin River, stage in the San Joaquin River is lower than in other approaches. This lower stage (effects resulting from increased Friant flood management pool) increases the water surface slope between the floodplain and the San Joaquin River and results in more water flowing into the river channel.



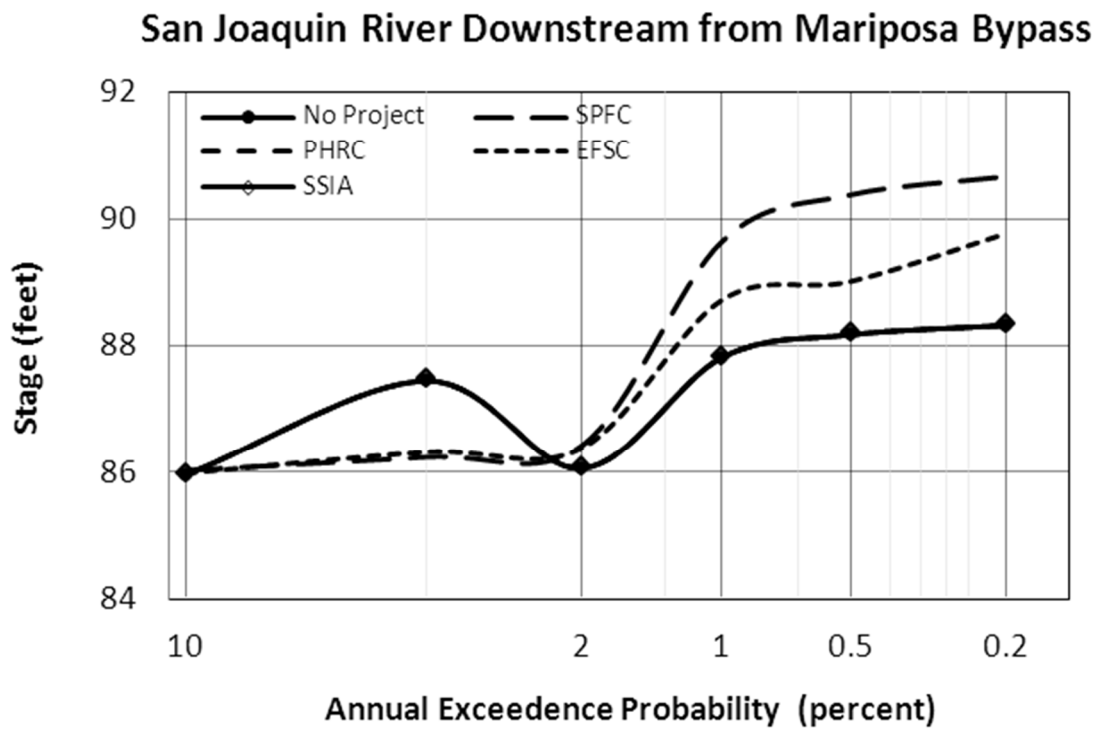
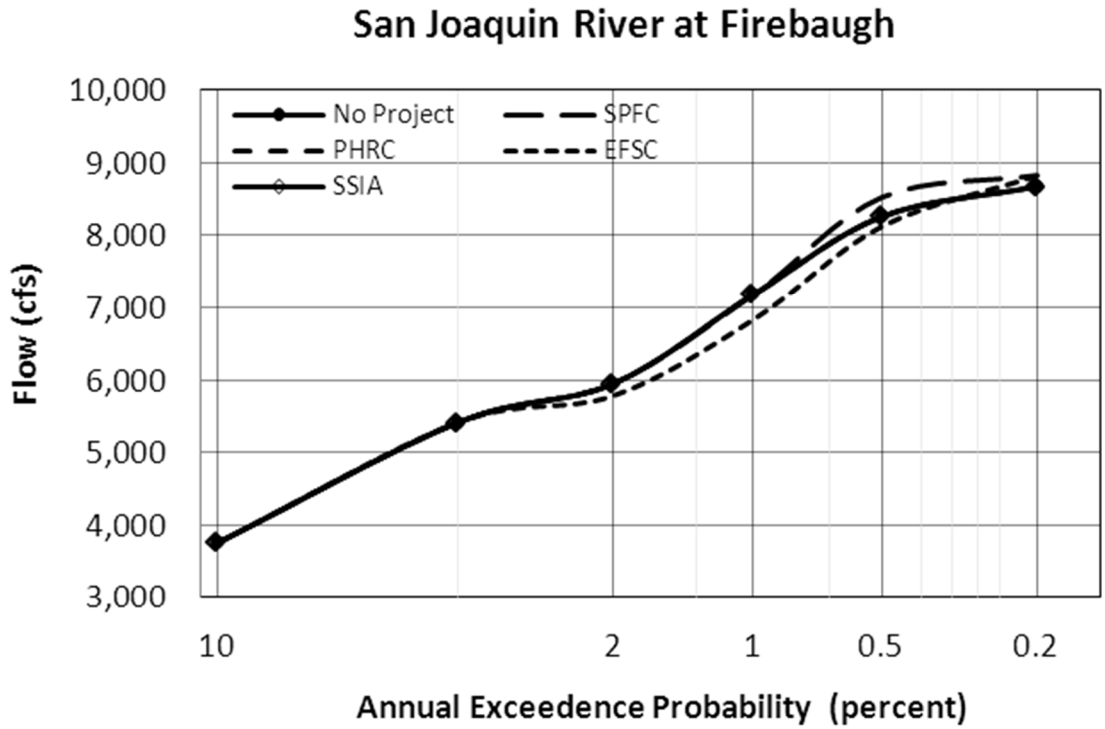


Figure 4-20. Flow- and Stage-Frequency Curves: San Joaquin River Downstream from Mariposa Bypass [5]

- **No Project Condition, PHRC Approach, and SSIA** – The stage in the San Joaquin River above the Chowchilla Bifurcation Control Structure for events greater than 4 percent AEP is high enough that significant levee breaks occur. These breaks allow large volumes of water to enter the surrounding floodplain shortly after the start of the flood event. The stage in the floodplain soon becomes great enough to breach levees along the San Joaquin River and allow water in the floodplain to enter the river. Although flow in the San Joaquin River immediately downstream from the Chowchilla Bifurcation Control Structure is the same for all scenarios, these flows re-enter the river upstream from Firebaugh and increase flow and stage. For the 1 percent AEP event through the 0.2 percent AEP event, peak flow increases slightly while the peak stage remains constant because there is a levee break immediately downstream from this location

- **SPFC Approach** – Peak flows are slightly higher for the 1 percent AEP flood event and greater compared to the No Project condition due to levee rehabilitation, and higher than in the EFSC approach due to the absence of any changes to flood storage at Friant Dam.

- **EFSC Approach** – Peak flows in the San Joaquin River are reduced by increased flood management storage at Friant Dam to the point that significant levee breaks are reduced or delayed through the 1 percent AEP event. As a result, the floodplain does not fill and there is little to no inflow into the San Joaquin River from the surrounding floodplain upstream from Firebaugh. However, at the 0.5 percent AEP flood and greater, although there is some reduction in peak flows below Friant Dam, even the reduced flows are too great to prevent significant levee breaks upstream from the control structure, and there are significant inflows to the San Joaquin River from the floodplain as in the No Project condition and other approaches.



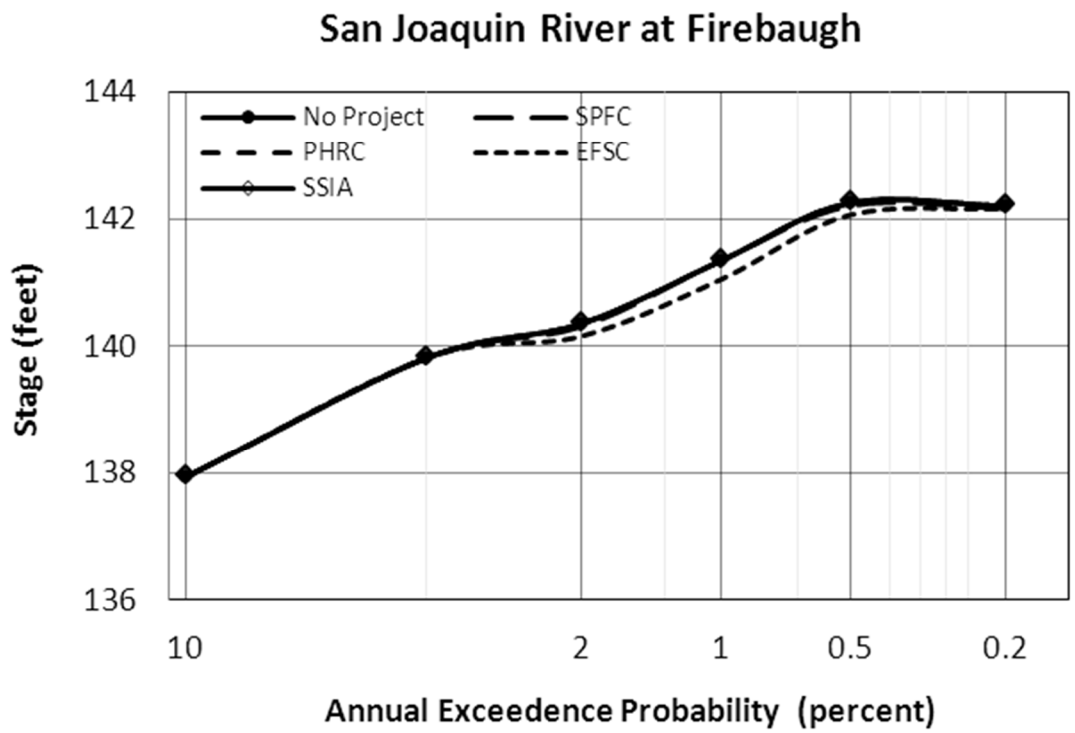
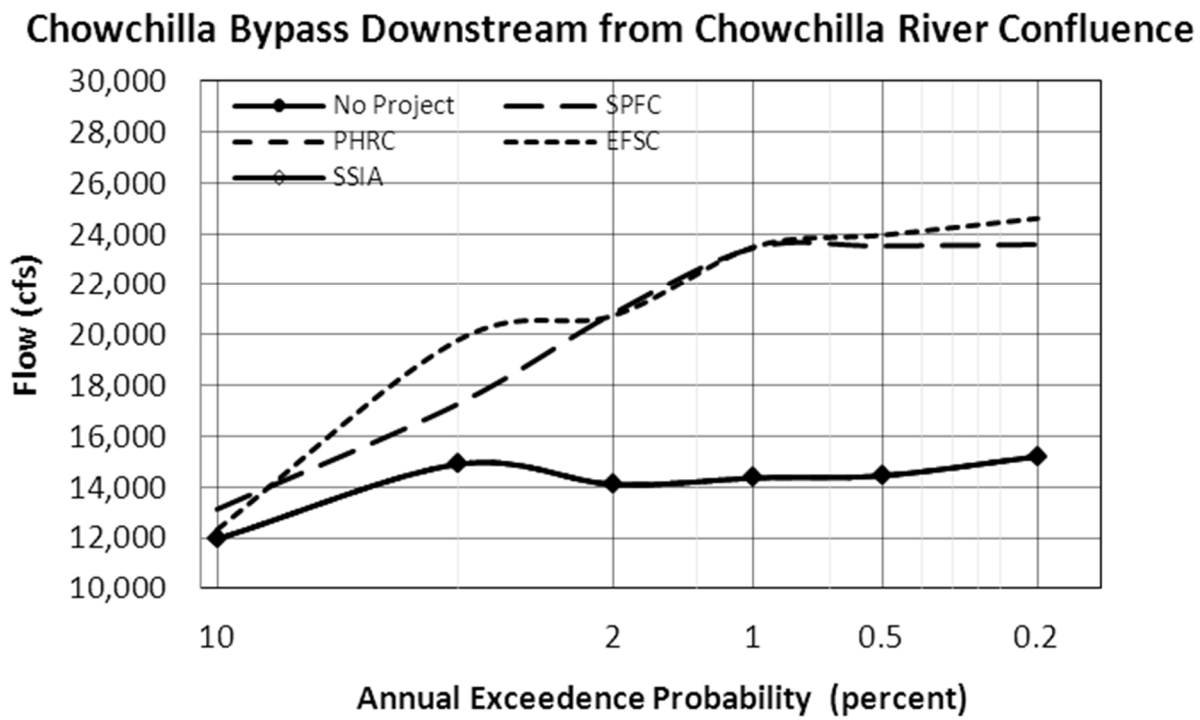


Figure 4-21. Flow- and Stage-Frequency Curves: San Joaquin River at Firebaugh [6]

- **No Project Condition** –Levee breaks occur upstream from this location, and levee breaks occur for all events larger than the 10 percent AEP flood. Levee breaks release flow into adjacent floodplains, so in-channel flows and river stages are lower than for approaches with strengthened levees for all flood events. Larger flood events (0.1 percent AEP and larger) greatly exceed the channel capacity, and cause virtually no increases in in-channel stage and flow.

- **SPFC Approach** – Restoring SPFC levees reduces the number of upstream levee breaks through the 2 percent AEP event, increasing in-channel flow and stage compared to the No Project condition. At the 1 percent AEP event and greater, significant upstream levee breaks occur, so peak flow and stage remains relatively constant; however, the improved levees maintain higher in-channel flow and stage compared to the No Project condition.

- **EFSC Approach** – Restoring SPFC levees results in higher flows and stages compared to the No Project condition. In-channel peak flows are lower for the EFSC approach compared to the SPFC approach for the 4 percent and 2 percent AEP events because of increased flood management pool at Friant Dam. Friant Dam continues to provide some management of flood peaks at the 1 percent AEP event and greater in the EFSC approach.



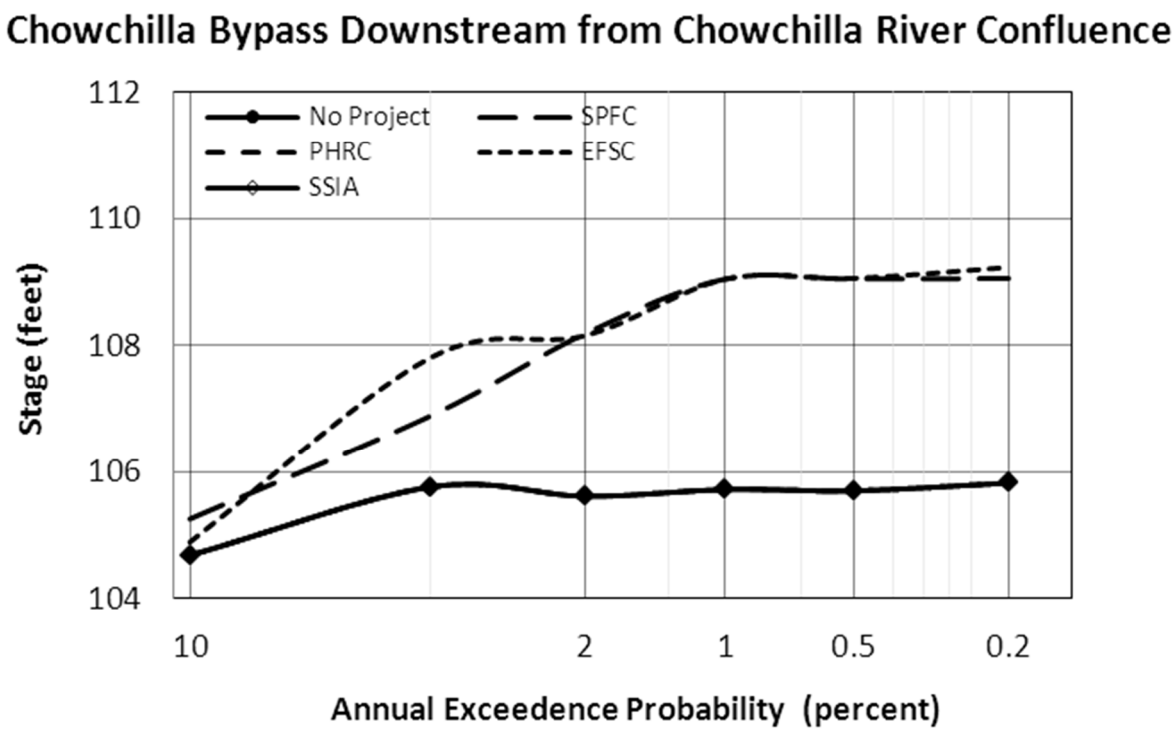
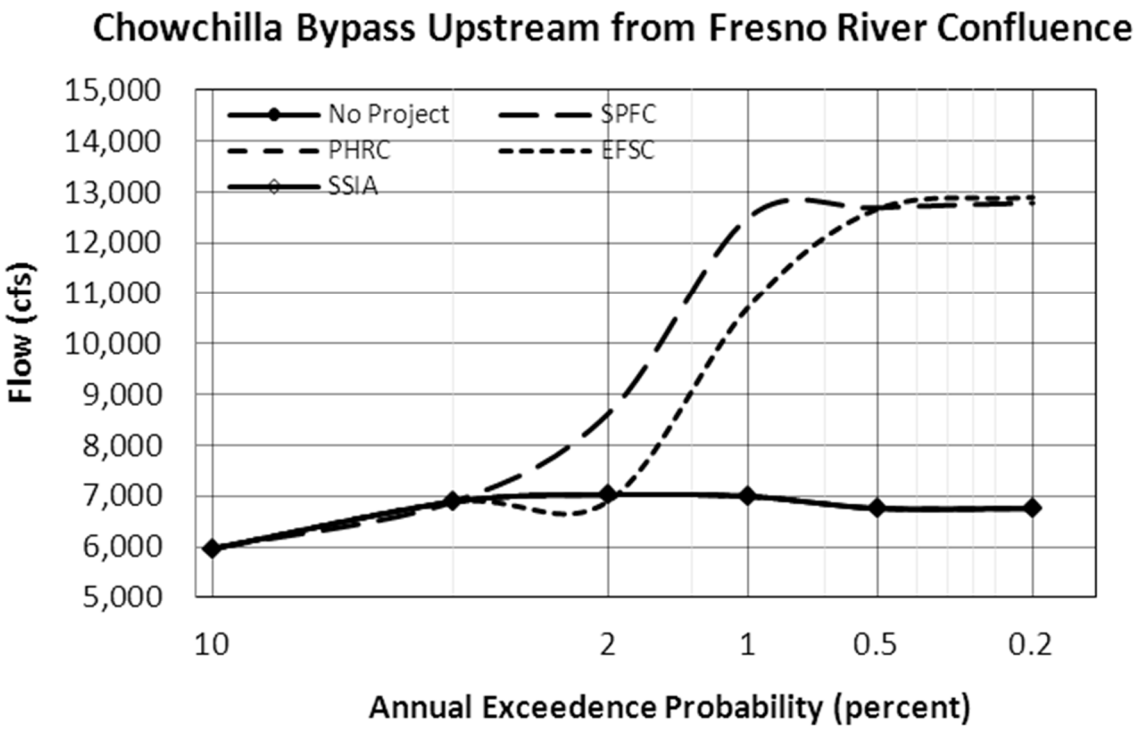


Figure 4-22. Flow- and Stage-Frequency Curves: Chowchilla Bypass Upstream from Chowchilla River Confluence [7]

- **No Project Condition, PHRC Approach, and SSIA –** No modifications to the flood management system upstream from this location, and significant levee breaks occur for all events larger than the 4 percent AEP flood. Levee breaks release flow into adjacent floodplains, so in channel flows and river stages are lower than for approaches with strengthened levees for all flood events. Maximum flows and stages decrease slightly with larger flood events because increased flows cause more upstream levee breaks.

- **SPFC Approach** – Restoring SPFC levees reduces the number of significant upstream levee breaks for all events, increasing in-channel flow and stage compared to the No Project condition. Peak flows remain nearly constant beyond the 2 percent AEP event because inflows to the bypass are reduced by upstream levee failures. Peak stage continues to increase up to the 0.5 percent AEP event because of increasing backwater effects from higher downstream flows (flows reenter the channel from the floodplain through a levee breach immediately downstream from this location).

- **EFSC Approach** – Restoring SPFC levees results in higher flows and stages compared to the No Project condition. In-channel peak flows are lower for the EFSC approach compared to the SPFC approach for the 4 percent and 2 percent AEP events because the increased flood management pool at Friant Dam reduces peak discharge rates to the San Joaquin River. Peak flow and stage is approximately equal to the SPFC approach beyond the 2 percent AEP because inflows to the Bypass are reduced by upstream levee failures.



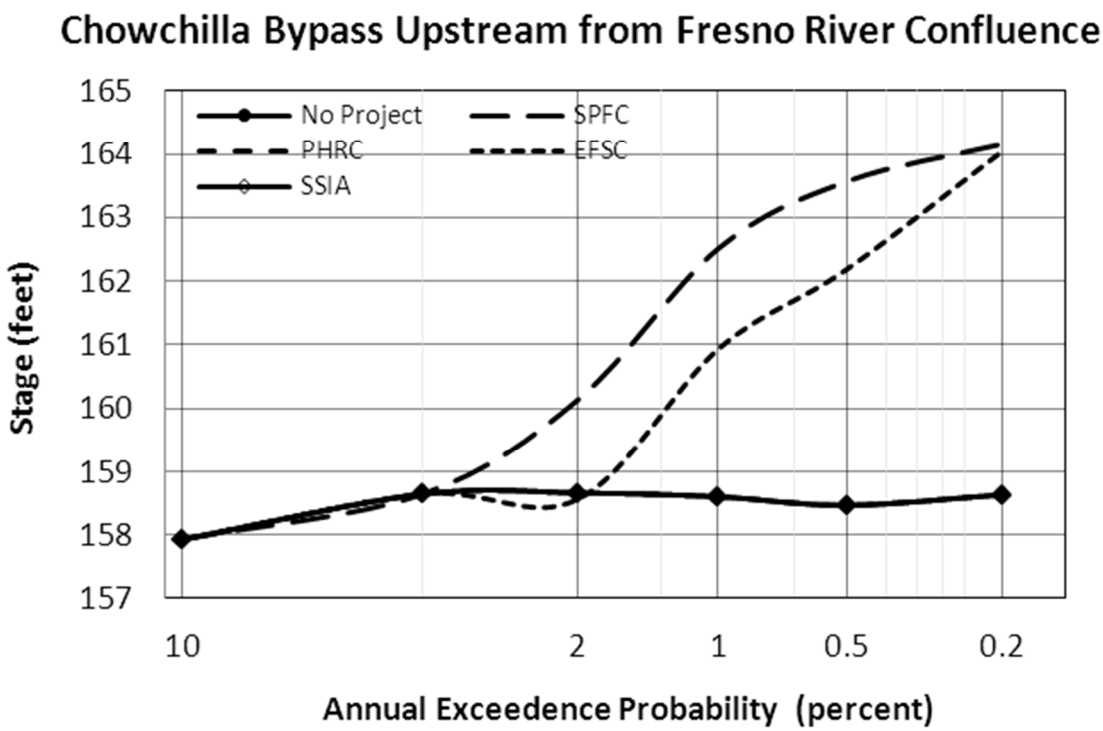
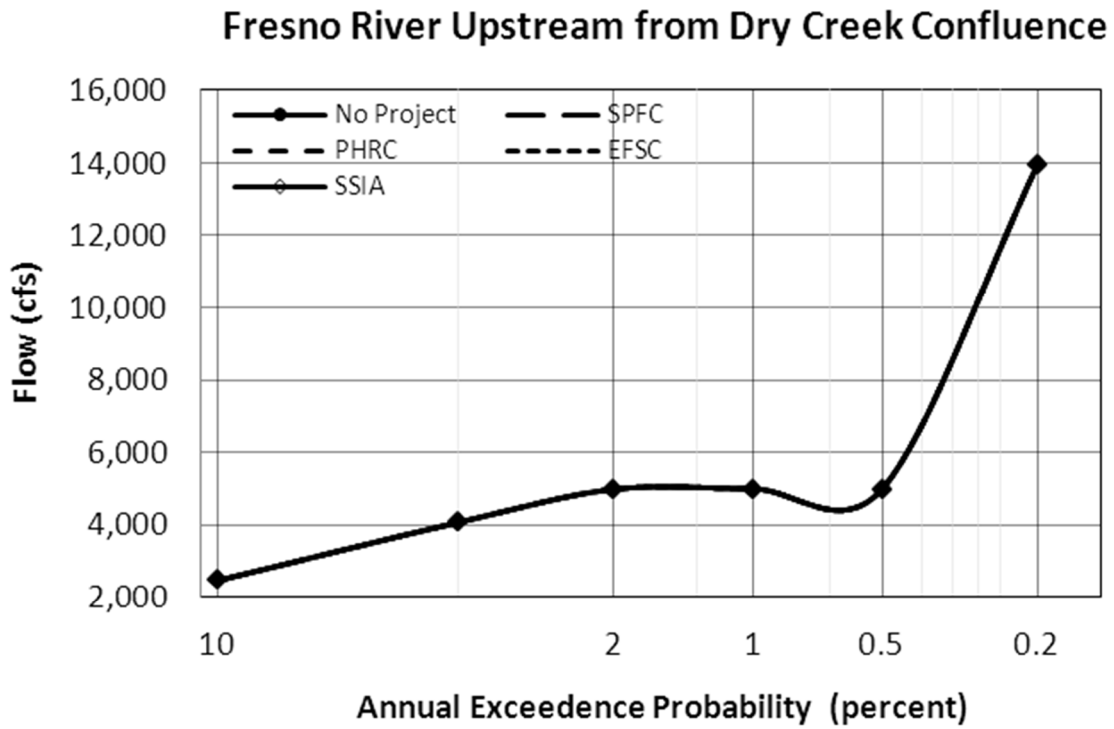


Figure 4-23. Flow- and Stage-Frequency Curves: Chowchilla bypass Upstream from Fresno River Confluence [8]

- **No Project Condition, PHRC Approach, and SSIA –** No modifications to the existing flood management system along the Fresno River, and significant levee breaks occur along the Fresno River downstream from this location for all events larger than the 2 percent AEP. Levee breaks release flow into adjacent floodplains, resulting in a decreased backwater effect and a subsequent drop in the river stage. The peak flow upstream from this location is the same for the 1 percent and 0.5 events, due to upstream reservoir operations, so there is no change in river flow or stage at these events.

- **SPFC and EFSC Approaches** – Reduce the number of downstream levee breaks on the Chowchilla Bypass for all events, resulting in an increased backwater effect and higher stages compared to the No Project condition. Because there are no modifications to the flood management system upstream from this point, peak flows are approximately equal in all events (this location is very close to a boundary point in the model, so flows are mostly controlled by boundary inflows). The peak flow upstream from this location is the same for the 1 percent and 0.5 events, due to upstream reservoir operations, so there is no change in river flow or stage at these events.



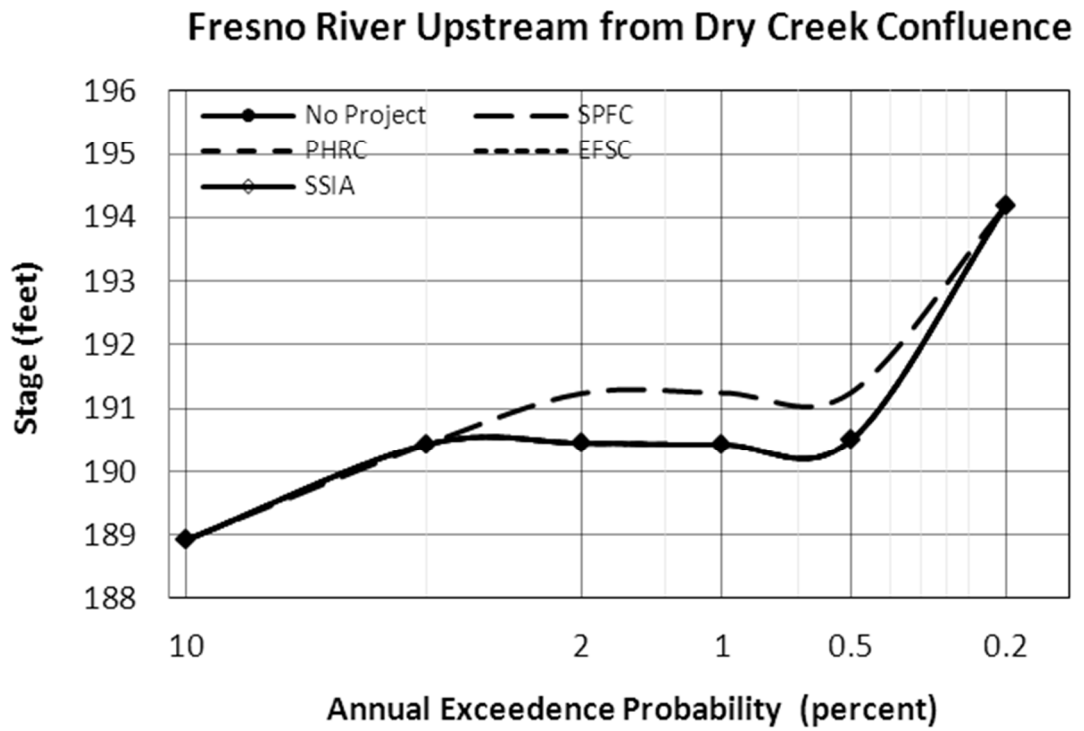


Figure 4-24. Flow- and Stage-Frequency Curves: Fresno River Upstream from Dry Creek Confluence [9]

Table 4-3. San Joaquin River Basin Simulated Out-of-System Volumes by Return Period for No Project Condition, and Change in Out-of-System Volumes for CVFPP Approaches

Approach	Return Period (years)	Flood Zones												
		1	2	3	4	5	6	7	8	9	10	11	12	13
No Project Volume (acre-feet)	10	0	0	0	0	27	0	24	0	0	0	0	0	0
	25	0	0	0	0	41	0	42	234	0	0	0	0	0
	50	0	0	0	192	82	0	58	0	0	198	0	0	0
	100	113	0	0	186	98	0	44	427	0	311	31	46	64
	200	148	0	0	301	113	69	50	485	0	370	38	86	420
	500	183	0	0	376	173	245	55	606	101	372	45	181	642
Achieve SPFC Design Flow Capacity Volume (acre-feet)	10	0	0	0	0	-27	0	-24	0	0	0	0	0	0
	25	0	0	0	0	-41	0	-42	-234	0	0	0	0	0
	50	0	0	0	-192	-82	0	-58	0	0	3	0	0	0
	100	1	0	0	-186	-98	0	-44	-76	0	2	-31	-29	-64
	200	-1	0	0	-210	-113	-69	-29	-65	0	-6	-9	0	-310
	500	-1	0	0	-241	-49	-49	-9	-73	-101	-8	-3	12	-15
Protect High Risk Communities Volume (acre-feet)	10	0	0	0	0	0	0	0	0	0	0	0	0	0
	25	0	0	0	0	0	0	0	0	0	0	0	0	0
	50	0	0	0	0	0	0	0	0	0	-194	37	0	0
	100	0	0	0	0	0	0	0	0	0	-310	-11	8	0
	200	0	0	0	0	0	0	0	0	0	-369	-1	-1	-1
	500	0	0	0	0	0	0	0	0	0	-371	0	5	-1
Enhance Flood System Capacity Volume (acre-feet)	10	0	0	0	0	-27	0	-24	0	0	0	0	0	0
	25	0	0	0	0	-41	0	-42	-234	0	0	0	0	0
	50	0	0	0	-192	-82	0	-58	0	0	-198	0	0	0
	100	-109	0	0	-186	-98	0	-44	-118	0	-189	-31	-46	-64
	200	-38	0	0	-301	-113	-69	-26	-69	0	-94	-38	-71	-411
	500	-26	0	0	-256	-49	-245	2	-61	-101	9	-20	-77	-205
State Systemwide Investment Approach Volume (acre-feet)	10	0	0	0	0	0	0	0	0	0	0	0	0	0
	25	0	0	0	0	0	0	0	0	0	0	0	0	0
	50	0	0	0	0	0	0	0	0	0	-194	37	0	0
	100	0	0	0	0	0	0	0	0	0	-310	-11	8	0
	200	0	0	0	0	0	0	0	0	0	-369	-1	-1	-1
	500	0	0	0	0	0	0	0	0	0	-371	0	5	-1

Notes: 10 percent AEP = 10-year return period; 4 percent AEP = 25-year return period; 2 percent AEP = 50-year return period; 1 percent AEP = 100-year return period; 0.5 percent AEP = 200-year return period; 0.2 percent AEP = 500-year return period

Table 4-4. San Joaquin River Basin Simulated Flow Volume into Delta

Approach	*Flow Volume Entering Delta (TAF)											
	10% AEP (10-year)		4% AEP (25-year)		2% AEP (50-year)		1% AEP (100-year)		0.5% AEP (200-year)		0.2% AEP (500-year)	
	Volume	Volume Change**	Volume	Volume Change**	Volume	Volume Change**	Volume	Volume Change**	Volume	Volume Change**	Volume	Volume Change**
No Project Condition	251	-	312	-	338	-	378	-	463	-	590	-
Achieve SPFC Design Flow Capacity Approach	252	1	321	9	352	14	404	26	483	20	605	15
Protect High Risk Communities Approach	251	0	312	0	337	-1	379	1	464	1	590	0
Enhance Flood System Capacity Approach	253	2	323	11	316	-22	382	4	457	-6	566	-24
State Systemwide Investment Approach	251	0	312	0	337	-1	379	1	464	1	590	0

Notes:
*based on the volume of San Joaquin River at Vernalis during 1/18 - 1/21
** Volume Change (TAF) is the difference between each approach and the No Project Condition
Key:
AEP = annual exceedence probability
SPFC = State Plan of Flood Control
TAF = Thousand acre-feet

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5.0 Stockton Area Analysis

This section provides a description of the hydrology, hydraulic modeling, and floodplain modeling assumptions and methodology for the No Project condition and each CVFPP approach in the Stockton area. The section also contains the results from the Stockton area hydraulic and floodplain modeling.

It is important to note that the hydraulic modeling described below is a deterministic process that simulates levee breaches based on data provided regarding levee stability. Hydraulic modeling cannot and does not predict the location of actual levee breaches.

5.1 Methodology

An overview of overall CVFPP hydraulic modeling was given in Section 3.1 and Figure 3-1. As explained there, hydraulic models of the river systems are one of the tools used to evaluate the CVFPP planning approaches. As shown in Figure 5-1, input to the economic analysis models for comparison of approaches also requires floodplain modeling.

While the CVFPP used existing tools (i.e., Comprehensive Study hydrology and hydraulic models) as much as possible for evaluating the planning approaches, no models for the Calaveras River (including Mormon Slough and the Stockton Diverting Canal) and Bear Creek were developed for the Comprehensive Study. Hence, it was necessary to develop hydrology and hydraulic models for those two streams in the Stockton area as described in the following sections.

5.1.1 Hydrology Development

As described previously, riverine hydraulic models require flow hydrographs (a time-series of flows) as upstream boundary conditions. The Sacramento and San Joaquin UNET models were used to determine river stage, velocity, depth, and levee breaches, as well as breakout and return flows from overbank areas for each CVFPP approach, but these models do not cover the Stockton area. Each set of hydrographs represents either unregulated or regulated flow conditions (simulated reservoir releases from reservoir models) under different storm centerings (a centering is a set of synthetic storms covering a range of AEPs) that will result in peak flows at a given location.

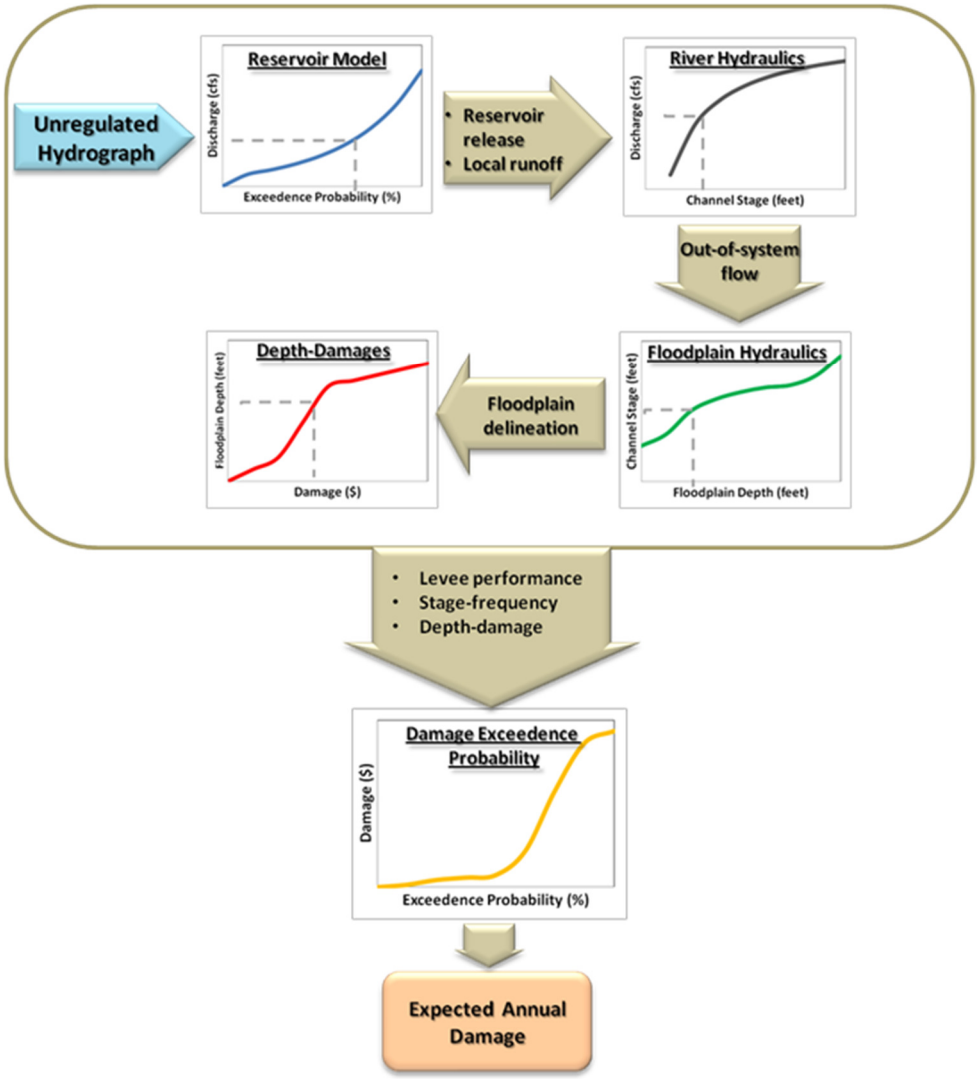

Figure 5-1. Schematic of Overall Modeling Framework

Comprehensive Study hydrology was available for the Calaveras River out of New Hogan Reservoir, leading to the upper end of Mormon Slough and then the Stockton Diverting Canal. But, Comprehensive Study hydrology was not developed for the Calaveras River downstream from the Mormon Slough Diversion, Bear Creek, Paddy Creek, Upper Mosher Creek, Pixley Slough, or Mosher Slough. To provide input data for the hydraulic analyses of reaches of the streams listed above that are protected by SPFC facilities, the following steps were taken:

1. Obtain peak flows for each stream using data from past studies.

2. Scale Duck Creek hydrology from the Comprehensive Study to produce flow hydrographs for each stream.

The peak flows used were from SJAFCA's Flood Protection Restoration Project (SJAFCA, 1998a). As part of SJAFCA's effort, hydrologic models were developed for the 50, 1 and 0.5 percent AEP (2-, 100-, and 200-year return period) storm events using the USACE HEC-1 rainfall-runoff model. Those peak flows were plotted on log-probability paper and the peak flows for the 50, 10, 4, 2, and 0.2 percent AEP storm events were interpolated or extrapolated from the curves. The peak flows are shown in Table 5-1, with the HEC-1 flows underlined.

Table 5-1. Peak Flows for Various AEP Storm Events (Percent)

Stream	50% AEP	10% AEP	4% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Bear Creek	1,137	3,100	4,300	5,300	6,367	7,279	9,300
Paddy Creek	88	210	290	360	434	510	640
Pixley Slough	121	305	430	530	667	778	980
Upper Calaveras River[1]	161	480	720	920	1,170	1,433	1,800
Mosher Slough	294	410	460	500	532	580	620
Upper Mosher Cr.	156	380	540	670	851	966	1,200
Duck Creek[2]	238	533	729	855	1,006	1,106	1,257

Source: SJAFCA, 1998s except where noted.

Notes:
Peak flows from HEC-1 are underlined.
[1] Downstream from the Mormon Slough Diversion.
[2] Peak flow taken from Comprehensive Study hydrology.

Key:
AEP = Annual Exceedence Probability

To be consistent with the other hydrology used for the 2012 CVFPP, the hydrology for these six streams was developed to match the Comprehensive Study pattern of flows (i.e., 34-day event, hourly flows, with the largest peak flow occurring around Day 17). To accomplish this, it was first assumed that the hydrology for these three streams would have peak flows approximately equal to the flows shown in Table 5-1 for a given AEP. Secondly, it was assumed that the shape of the flow hydrograph would match the shape the Comprehensive Study's hydrographs for Duck Creek near Farmington. This is because the characteristics of floods would be similar in the sub-watersheds.

Duck Creek near Farmington was selected as the base pattern for the Stockton area streams because its watershed is at similar elevations to the other streams and it is geographically the closest stream included in the Comprehensive Study. Hence, while Duck Creek may not be the same size as the Stockton area watersheds, it would likely experience similar precipitation patterns and is appropriate to develop hydrology for other streams for use in preliminary evaluations for the 2012 CVFPP.

In each watershed for which flood hydrographs were developed, and for each storm AEP, Duck Creek hourly flows were multiplied by a constant to develop the particular stream's hourly flows. The constant was the ratio of each stream's peak flow to Duck Creek's peak flow for a storm with a given AEP. For example, the constant for a 10 percent AEP storm on Bear Creek would be 5.8 (3,100 cubic feet per second (cfs) divided by 533 cfs). Hence, for this particular AEP flood, Duck Creek hourly flows were multiplied by 5.8 to obtain the inflow hydrograph for Bear Creek.

The Calaveras River storm centering was used for both Bear Creek and the Calaveras River because it resulted in the highest flow flood events. The hourly flows for six AEP flood events developed for the Comprehensive Study (10, 4, 2, 1, 0.5, and 0.2 percent) were used. If the flow at any hour was 0 cfs, it was changed to 1 cfs for better continuity when run in the unsteady HEC-RAS model.

5.1.2 Hydraulic Model Development

HEC-RAS Version 4.1 was used to develop the Stockton area hydraulic models for the CVFPP by translating existing HEC-2 models from SJAFCA. Two separate HEC-RAS models, Calaveras River and Bear Creek, were created in this manner (Figure 1-3). The following sections describe model settings specific to the CVFPP evaluation for the Stockton area. For more information about the capabilities of the HEC-RAS model, refer to the January 2010 HEC-RAS User's Manual (USACE, 2010).

Model Selection

The available riverine hydraulic models for the Stockton area were from the SJAFCA Flood Protection Restoration Project (SJAFCA, 1998b) and from the SJAFCA Provisionally Accredited Levee binder submittal to FEMA for the Lower Calaveras River (SJAFCA, 2010a). This project developed a HEC-RAS model set for the Calaveras River from existing models as detailed below:

- **Upper Calaveras River** – HEC-2 model (SJAFCA, 1998b)
- **Lower Calaveras River** – HEC-RAS model (SJAFCA, 2010a)
- **Mormon Slough** – HEC-2 model (SJAFCA, 1998b)
- **Stockton Diverting Canal** – HEC-2 model (SJAFCA, 1998b)

Similarly, an HEC-RAS model of Bear Creek was developed using three HEC-2 models–Bear Creek, Mosher Diversion, and Upper Mosher Creek (SJAFCA, 1998b).

The HEC-2 models from the SJAFCA Flood Protection Restoration Project (SJAFCA, 1998b) were converted to HEC-RAS using the HEC-2 import feature in HEC-RAS. Some of the model inputs were also updated to include changes to the system since 1998.

Levee Breach Modeling

In HEC-RAS, the top of a levee is defined as a station and elevation point in each cross section. At a designated cross section, a breach elevation may be entered into the model and when the computed water surface elevation equals or exceeds this breach elevation, flood flows are diverted into the floodplain. The simulated levee breach is 100 feet wide. When the levee breaches, water will flow through the breach into a storage area associated with that cross section. The storage area will continue to fill until either the stage in the river decreases below the stage in the storage area or the stage in the storage area reaches the same elevation as the stage in the river.

Boundary Conditions

The four primary types of HEC-RAS model boundary conditions used for the Stockton area are:

- **Upstream Boundary Conditions** – Upstream boundary conditions for the Stockton area HEC-RAS models are flow hydrographs (i.e., discharge in cfs over time) for each particular flood for all reaches that are not connected to another reach at their upstream end. For the Calaveras River Model, there are two upstream hydrographs: Calaveras River just east of Highway 99 and Mormon Slough at Jack Tone Road. For the Bear Creek Model, there are three upstream hydrographs: Bear Creek, South Paddy Creek at Jack Tone Road, Mosher Creek Diversion to Bear Creek, and Pixley Slough. See Figure 5-2 for the upstream boundary hydrograph locations.

  Flows with an AEP of greater than 10 percent (10-year return period) were not modeled because the Stockton area flood management systems are designed to manage flood events with AEPs less than 10 percent. Therefore, it is anticipated that storms with greater than a 10 percent AEP would not cause serious impacts.

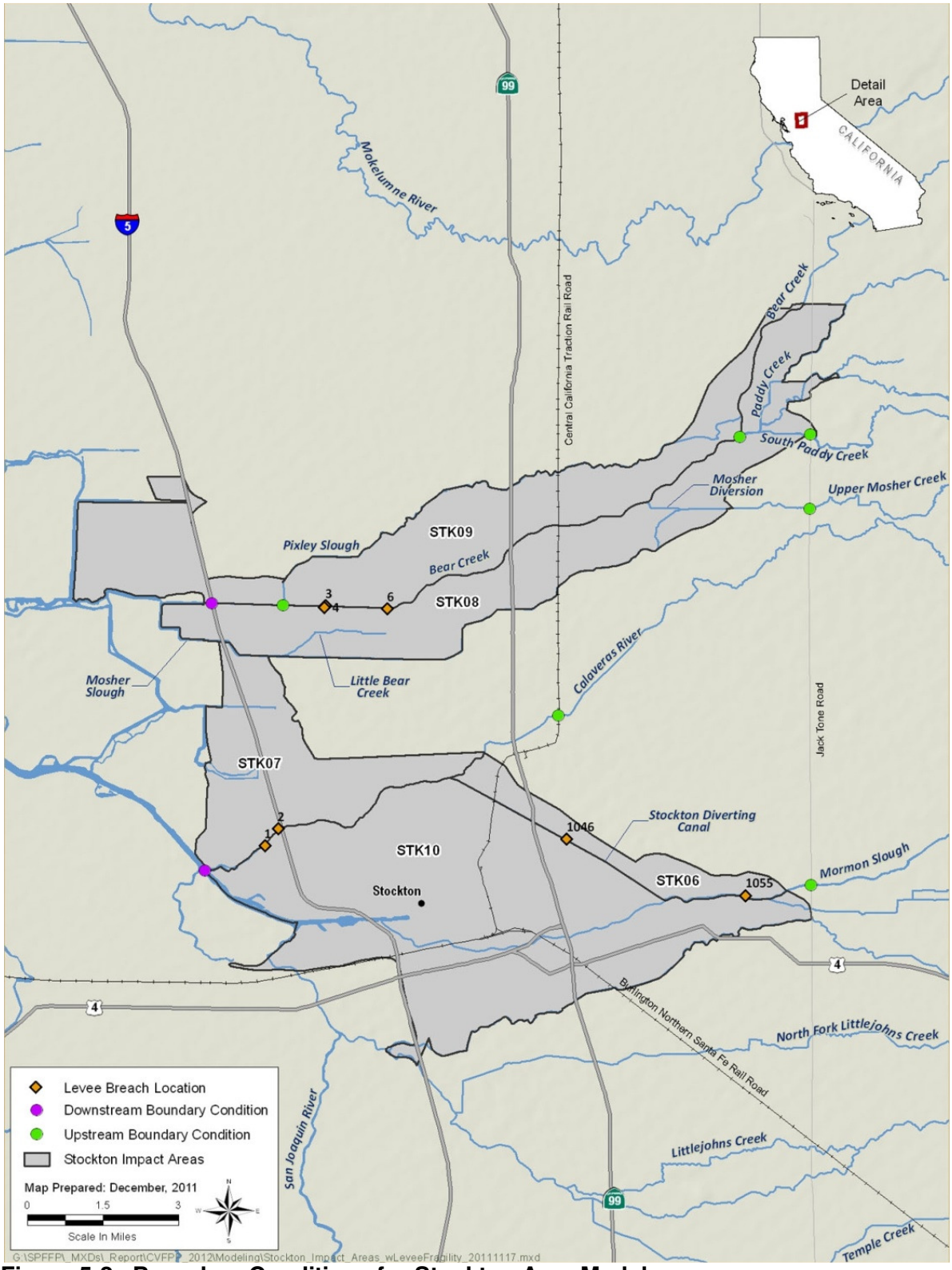

Figure 5-2. Boundary Conditions for Stockton Area Models

- **Interior Boundary Conditions –** Interior boundary conditions define the connections between stream reaches, as well as between stream reaches and other parts of the model. Interior boundary conditions ensure continuity of flow by defining river channel alignment, cross sections, and bridge geometries.

- **Downstream Boundary Conditions –** To function properly, a hydraulic model of a river system must define the water surface elevation at the downstream end of all model reaches not connected to another reach or river. Downstream boundary conditions are usually in the form of tailwater stage hydrographs that describe the variation of the downstream water surface elevation over time.

  River stage time series from the RMA Delta Model for (1) Calaveras River at San Joaquin River, and (2) Bear Creek at Disappointment Slough define the tailwater conditions for the Calaveras River Model and Bear Creek Model, respectively. See Figure 5-2 for downstream boundary hydrograph locations. Details of the RMA Delta Model are in Attachment 8D: Estuary Channel Evaluations.

- **Internal Boundary Conditions –** Internal boundary conditions are coded in the model to represent levee failure scenarios or floodplain interactions, spillways or weir overflow/diversion structures, bridge or culvert hydraulics, or pumped diversions. To simulate water leaving the river into the floodplain through breaches, storage areas representing floodplains were added to the HEC-RAS models; three for the Calaveras River Model (STK06, STK07, and STK10), and two for the Bear River Model (STK08 and STK09) (see Figure 5-2). Rating curves for the relationship between water stage and floodplain volume were developed as inputs to the models using the topographic data developed for CVFED.

Simulation Period

The simulation period for the Stockton area models was chosen to be 35 days and extends from 1:00 a.m., January 1, to midnight, February 4. This calendar period matches the time period for the UNET models.

5.1.3 Levee Performance Curves

The ULE Project and NULE Project under the DWR Levee Evaluations Program developed performance curves for levees in the Sacramento and San Joaquin river basins. Levee performance curves provide geotechnical relationships between river stage and the probability that a levee segment will breach (water from the water side of the levee flows in an uncontrolled manner to the landside of the levee) at a specific stage. Details on levee

performance curve development are contained in Attachment 8E: System/Levee Performance.

Levee performance curves from ULE and NULE were used to identify two water surface elevations of interest for the hydraulic analyses. These water surface elevations and the corresponding probability of levee failure at a particular levee location are as follows:

- **Probable failure point (PFP)** – 85 percent probability of failure
- **Top of levee (TOL)** – 100 percent probability of failure

These two water surface elevations were incorporated into UNET models to simulate conditional levee failure. This means that once the simulated river stage at a specific levee location reaches either the PFP or TOL elevation, depending on the CVFPP approach being modeled, a levee breach would begin to develop in UNET. Water from the river would then enter into the adjacent floodplain through the levee breach, and the downstream river stage and flow would be reduced. On the other hand, if TOL is used in the simulation, the water surface elevation and flow would be higher both upstream and downstream before the levee breach.

The water surface elevations of interest described above are not intended to represent or predict how levees would fail under an actual flood event. For example, under the PFP scenario, all levees assigned a PFP would fail in a simulated flood event once the water surface was equal to or higher than the PFP elevation. In reality, many of these levees would not fail even when the stage exceeds the PFP elevations, while others might fail before the stage reaches the PFP elevations. Further, floodfighting and other emergency actions (conditions that are not simulated in the hydraulic models) could result in very different levee failure probabilities.

In addition to simulating the PFP and TOL scenarios, model simulations were also conducted that considered infinitely tall levees along the river channels. These “infinite channel” simulations helped estimate the maximum potential flood flows and stages at various locations in the system.

5.1.4 Floodplain Model Development

The Comprehensive Study applied FLO-2D, a two-dimensional flood routing model, to simulate the inundation of floodplains in the Sacramento and San Joaquin river basins. For 2012 CVFPP development, FLO-2D was applied to simulate the inundation of floodplains in the Stockton area that were not previously covered by the Comprehensive Study.

Preliminary LiDAR topographic data developed by CVFED were used to set the terrain elevations for the five Stockton area floodplains (damage areas) in FLO-2D.

The levee breach time-series hydrographs output from HEC-RAS were used as input to the FLO-2D models at the corresponding breach locations. FLO-2D then simulated the area of inundation and water depth of each floodplain grid over the entire simulation period. The maximum depth at each grid point in each of the impact areas was used in the HEC-FDA model of the impact area to determine flood damages (see Attachment 8F: Flood Damage Analysis).

Damage areas STK07 and STK10 did not have levee breaches up to and including the 0.2 percent AEP (500-year) flood. To provide flows to use in the FLO-2D model to develop the grid depth information, a special 0.2 percent AEP (500-year) model run was made where the breach elevations at the levee breach locations for STK07 and STK10 were lowered such that a levee breach occurred. The flow hydrographs generated from these forced levee breaches were then used as input for STK07 and STK10 when determining the depth grids for use in HEC-FDA.

5.2 Stockton Area Results

The general Stockton area hydrology and floodplain assumptions for the No Project condition are described in Section 5.1. This section contains the HEC-RAS modeling results as flow-frequency, stage-frequency tables, and out-of-system volume-frequency tables. The tables were developed for four locations based on HEC-RAS model outputs (see Figure 5-2). These locations, which are shown in Figure 5-2 include:

- Stockton Diverting Canal at Highway 99
- Bear Creek at Highway 99
- Bear Creek at Interstate 5
- Calaveras River at Interstate 5

5.2.1 Riverine Hydraulics

The results from the riverine hydraulics analysis for the Stockton area Analysis contained in Tables 5-2 and 5-3 show the flows and stages for each AEP at the locations listed above and shown on Figure 5-2.

Results are shown only for the No Project condition and the Protect High Risk Communities because in the Stockton area the Achieve SPFC Design

Flow Capacity, Protect High Risk Communities, and State Systemwide Investment approaches are essentially the same. All of the levees are treated as urban levees for Protect High Risk Communities and State Systemwide Investment approaches, and the levee heights are nearly the same as those set for the Achieve SPFC Design Flow Capacity Approach and the levee breaches function the same in the hydraulic models. The Enhance Flood System Capacity Approach in the Stockton area is the same as the No Project Condition.

5.2.2 Out-of-System Volumes to FLO-2D

Estimates of out-of-system flood flow volumes into floodplains for modeling using the two-dimensional hydraulic computer model FLO-2D are shown in Table 5-4 for the damage/storage areas shown on Figure 5-3. The depth grid results from the FLO-2D modeling, based on the volumes shown in Table 5-4, are used in the Hydrologic Engineering Center Flood Damage Analysis (HEC-FDA) model analysis described in Attachment 8F: Flood Damage Analysis.

Results are shown only for the No Project condition and the Protect High Risk Communities as described in Section 5.2.1.

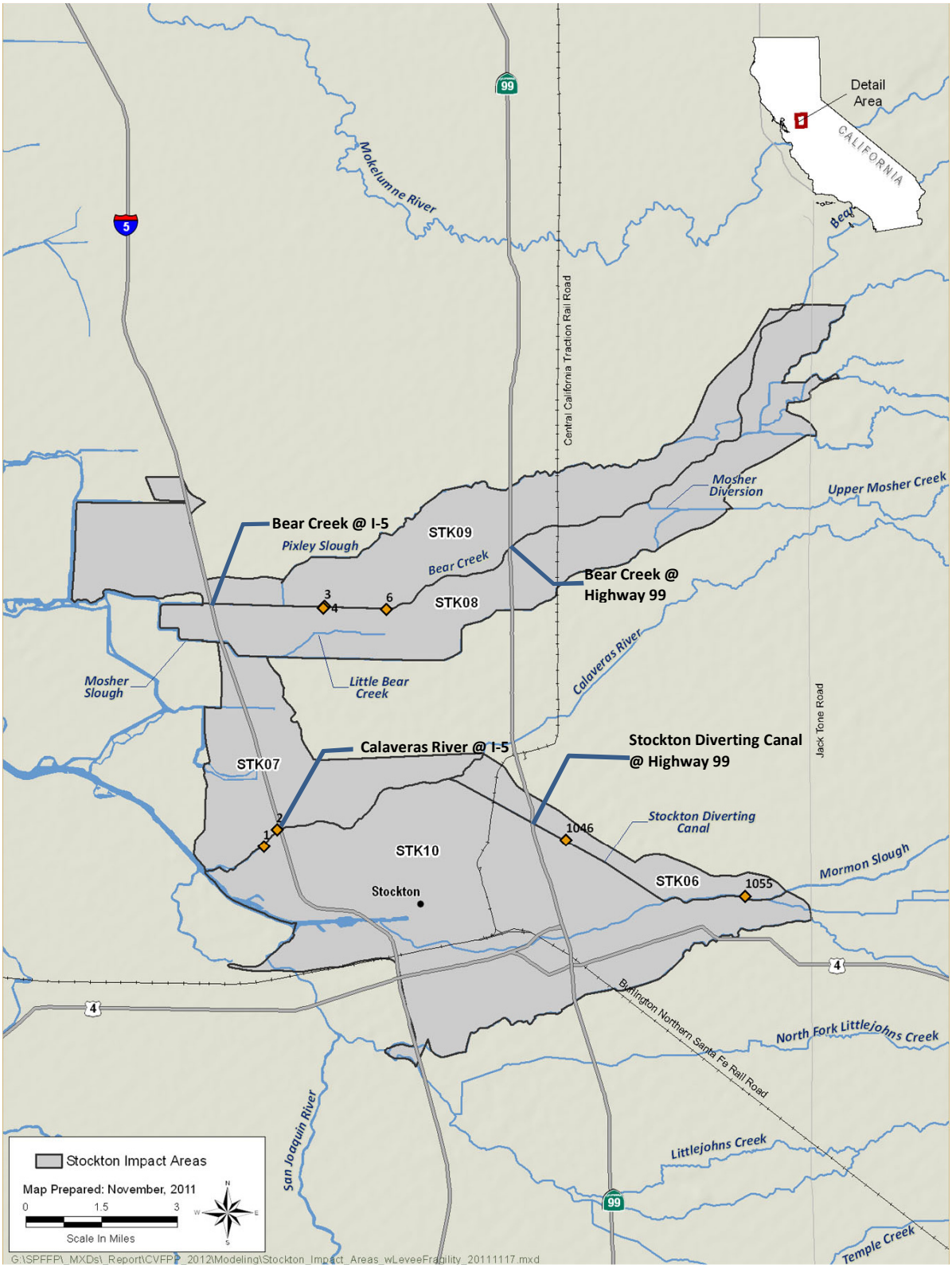

Figure 5-3. Stockton Area HEC-RAS Model Output Locations and FLO-2D Floodplains/Damage Areas

Table 5-2. Simulated Flows at Output Locations in Stockton Area

Location		Flow (cfs)					
		10% AEP	4% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Bear Creek @ Interstate 5	No Project	3,736	5,309	6,405	7,768	8,835	9,326
	Protect High Risk Communities	3,736	5,309	6,405	7,768	8,839	9,410
Bear Creek @ Highway 99	No Project	3,532	4,921	6,052	7,367	8,360	8,625
	Protect High Risk Communities	3,479	3,761	6,053	7,369	8,362	8,625
Calaveras River @ Interstate 5	No Project	12,441	12,011	12,485	12,489	12,339	21,408
	Protect High Risk Communities	12,441	12,011	12,485	12,489	12,339	21,415
Stockton Diverting Canal @ Highway 99	No Project	12,400	12,400	12,400	12,400	13,058	21,376
	Protect High Risk Communities	12,400	12,400	12,400	12,400	13,058	21,383

Key:
AEP = Annual Exceedence Probability
NGVD29 = National Geodetic Vertical Datum of 1929

Table 5-3. Simulated Stages at Output Locations in Stockton Area

Location		Stage (feet NGVD29)					
		10% AEP	4% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Bear Creek @ Interstate 5	No Project	6.1	7.0	7.7	8.5	9.2	9.5
	Protect High Risk Communities	6.1	7.0	7.7	8.5	9.2	9.5
Bear Creek @ Highway 99	No Project	39.3	39.3	39.8	41.9	42.4	42.8
	Protect High Risk Communities	39.3	39.3	39.8	41.9	42.4	42.8
Calaveras River @ Interstate 5	No Project	9.2	9.4	9.6	9.8	10.0	12.7
	Protect High Risk Communities	9.2	9.4	9.6	9.8	10.0	12.7
Stockton Diverting Canal @ Highway 99	No Project	31.2	31.2	31.2	31.2	32.0	38.9
	Protect High Risk Communities	31.2	31.2	31.2	31.2	32.0	38.9

Key:
AEP = Annual Exceedence Probability
NGVD29 = National Geodetic Vertical Datum of 1929

Table 5-4. Simulated Out-of-System Volumes in Stockton Area Floodplains (Damage Areas)

Damage Area		Out-of-System Volume (acre-feet)					
		10% AEP	4% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
STK06	No Project	-	-	-	-	-	15,773
	Protect High Risk Communities	-	-	-	-	-	13,027
STK07	No Project	-	-	-	-	-	-
	Protect High Risk Communities	-	-	-	-	-	-
STK08	No Project	-	-	-	-	978	1,188
	Protect High Risk Communities	-	-	-	-	-	-
STK09	No Project	-	-	-	-	13,933	14,712
	Protect High Risk Communities	-	-	-	-	-	-
STK10	No Project	-	-	-	-	-	-
	Protect High Risk Communities	-	-	-	-	-	-

Key:
AEP = Annual Exceedence Probability

5.2.3 Findings

The major findings from the tabulated data described above are presented in the following sections.

No Project Condition

The No Project condition assumes that levee breaches occur when the river stage reaches the Probable Failure Point (PFP) on a levee performance curve. No simulated levee breaches occurred on either the Calaveras River system or Bear Creek at AEP more frequent than 0.5 percent.

Achieve SPFC Design Flow Capacity Approach

The Achieve SPFC Design Flow Capacity Approach assumes that levee breaches occur when the river stage reaches the top of SPFC levees that have been raised to equal the 55/57 design profile. No simulated levee breaches occurred on either the Calaveras River system or Bear Creek at AEP more frequent than 0.5 percent.

Protect High Risk Communities Approach

The Protect High Risk Communities Approach assumes levee breaches occur when the river stage reaches the top of urban levees that have been set to be the existing levee elevation or the 200-year flood plus freeboard (3 feet). No simulated levee breaches occurred on either the Calaveras River system or Bear Creek at AEP more frequent than 0.5 percent.

Enhance Flood System Capacity Approach

The Enhance Flood System Capacity Approach in the Stockton area is the same as the Achieve SPFC Design Flow Capacity Approach. No simulated levee breaches occurred on either the Calaveras River system or Bear Creek at AEP more frequent than 0.5 percent.

State Systemwide Investment Approach

The State Systemwide Investment Approach is the same as the Protect High Risk Communities Approach in the Stockton area. No simulated levee breaches occurred on either the Calaveras River system or Bear Creek at AEP more frequent than 0.5 percent.

5.2.4 Limitations

The results of the hydrologic, riverine hydraulic, and floodplain modeling for the Stockton area Analysis are suitable for use in high-level planning studies such as the CVFPP. With significant additional work and field verification and data collection, the hydraulic and floodplain models could be adapted for use in more detailed project studies.

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7.0 Acronyms and Abbreviations

1-D	one-dimensional
AEP	annual exceedence probability
Board	Central Valley Flood Protection Board
cfs	cubic feet per second
Comprehensive Study	Sacramento and San Joaquin River Basins Comprehensive Study
CVFED	Central Valley Floodplain Evaluation and Delineation Program
CVFPP	Central Valley Flood Protection Plan
Delta	Sacramento-San Joaquin Delta
DWR	California Department of Water Resources
EFSC	Enhanced Flood System Capacity Approach
HEC-RAS	Hydrologic Engineering Center River Analysis System
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NPRJ	No Project Condition
NULE	Non-Urban Levee Evaluations
PFP	probable failure point
PHRC	Protect High Risk Communities Approach
Q-F	discharge-frequency
Reclamation	U.S. Department of the Interior, Bureau of Reclamation
RM	River mile
SAFCA	Sacramento Area Flood Control Agency
S-F	stage-frequency
SPA	Systemwide Planning Area
SPFC	State Plan of Flood Control or Achieve SPFC Design Flow Capacity Approach
SSIA	State Systemwide Investment Approach
State	State of California

TAF ..... thousand acre-feet

TOL ..... top of levee

TRLIA ..... Three Rivers Levee Improvement Authority

ULE ..... Urban Levee Evaluations

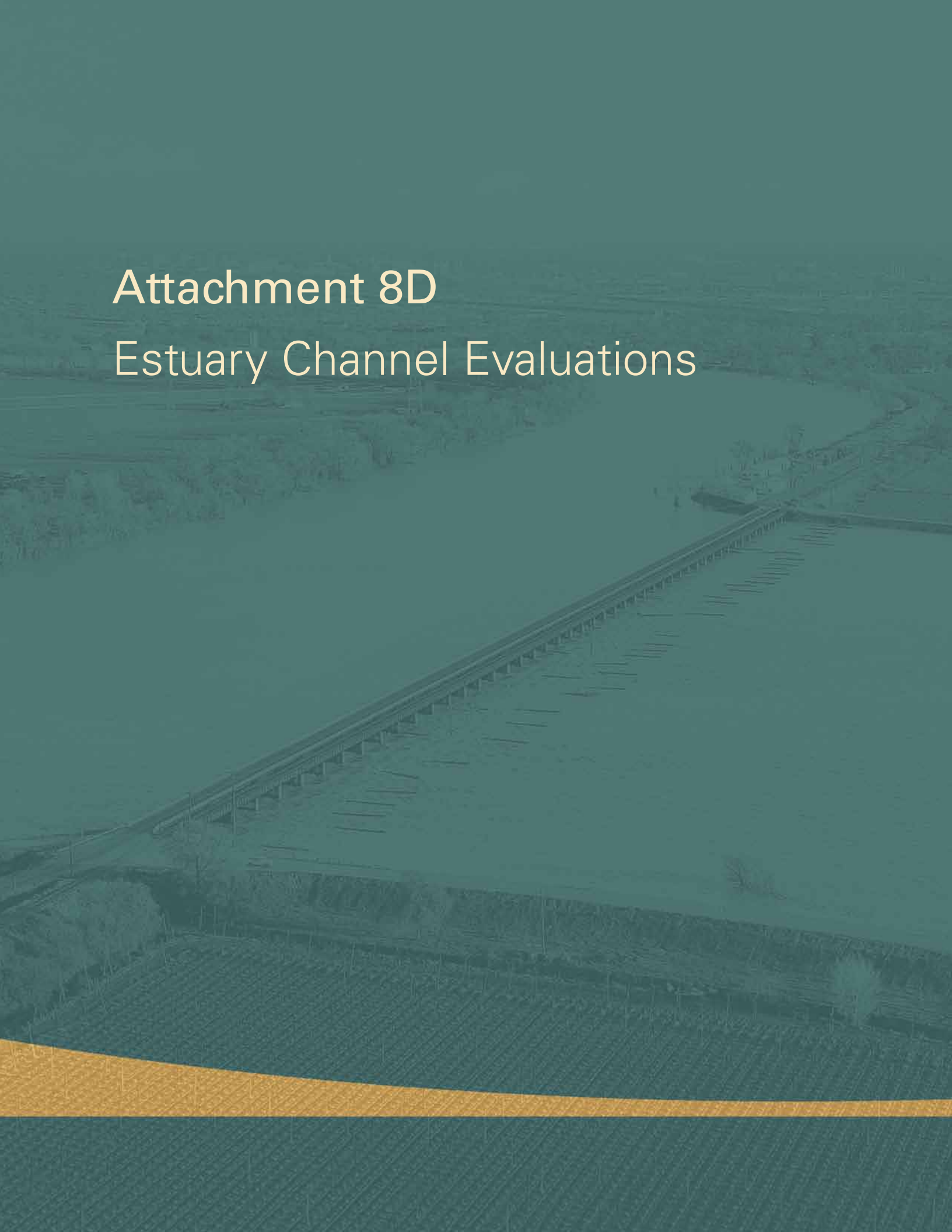
UNET ..... Unsteady flow Through a Full NETwork of open channels computer model

USACE ..... U.S. Army Corps of Engineers

STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

Attachment 8D

Estuary Channel Evaluations



CENTRAL VALLEY
FLOOD MANAGEMENT
PLANNING PROGRAM

FloodSAFE
CALIFORNIA

Public Draft

2012 Central Valley Flood Protection Plan

Attachment 8D: Estuary Channel Evaluations

January 2012

STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

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1.0 Introduction

This section states the purpose of this attachment, gives background information (including a description of planning areas, goals, and approaches), introduces the Sacramento-San Joaquin Delta (Delta) region, and provides an overview of the report organization.

1.1 Purpose of this Attachment

As part of development of the 2012 Central Valley Flood Protection Plan (CVFPP), hydraulic modeling was performed for the Sacramento River Basin, San Joaquin River Basin, Stockton area, and Delta region to support flood management system evaluations. Results from the hydraulic modeling were used to describe the hydraulic performance of the existing flood management system (No Project) and to simulate management actions for various approaches to improving the system. Hydraulic modeling results were also used as input to flood damage evaluation models to estimate economic values of flood damages (Attachment 8F: Flood Damage Analysis).

This attachment documents estuary hydraulic modeling methodology and results for the Delta for each of the following CVFPP approaches:

- No Project
- Achieve State Plan of Flood Control (SPFC) Design Flow Capacity
- Protect High Risk Communities
- Enhance Flood System Capacity
- State Systemwide Investment

Riverine channel hydraulic modeling of the Sacramento and San Joaquin river basins using UNET models (documented in Attachment 8C: Riverine Channel Evaluations) provided the upstream boundary conditions for the Resource Management Associates, Inc. (RMA) Delta Model used to simulate estuary channel hydraulics in the Delta.

This attachment documents the following modeling results:

- Stage-frequency (S-F) relationship for in-river locations inside estuarine channels of the Delta. Frequency for storm events of various annual exceedence probabilities (AEP) is expressed in percentage (i.e., 1 percent AEP, or a storm with a 100-year return period).

- Out-of-system volume from river reaches in the Delta. This represents the total volume of water that would leave Delta channels and enter into an island through levee breaches due to levee overtopping. Out-of-system volume is in thousand acre-feet (TAF).

After completion of the 2012 CVFPP, new riverine and floodplain models developed by the Central Valley Floodplain Evaluation and Delineation Program (CVFED) will be become available for use in the 2017 CVFPP.

1.2 Background

As authorized by Senate Bill 5, also known as the Central Valley Flood Protection Act of 2008, the California Department of Water Resources (DWR) has prepared a sustainable, integrated flood management plan called the CVFPP, for adoption by the Central Valley Flood Protection Board (Board). The 2012 CVFPP provides a systemwide approach to protecting lands currently protected from flooding by existing facilities of the SPFC, and will be updated every 5 years.

As part of development of the CVFPP, a series of technical analyses were conducted to evaluate hydrologic, hydraulic, geotechnical, economic, ecosystem, and related conditions within the flood management system and to support formulation of system improvements. These analyses were conducted in the Sacramento River Basin, San Joaquin River Basin, and Delta.

1.3 CVFPP Planning Areas

For planning and analysis purposes, and consistent with legislative direction, two geographical planning areas were important for CVFPP development (Figure 1-1):

- **SPFC Planning Area –** This area is defined by the lands currently receiving flood protection from facilities of the SPFC (see *State Plan of Flood Control Descriptive Document* (DWR, 2010a)). The State of California's (State) flood management responsibility is limited to this area.

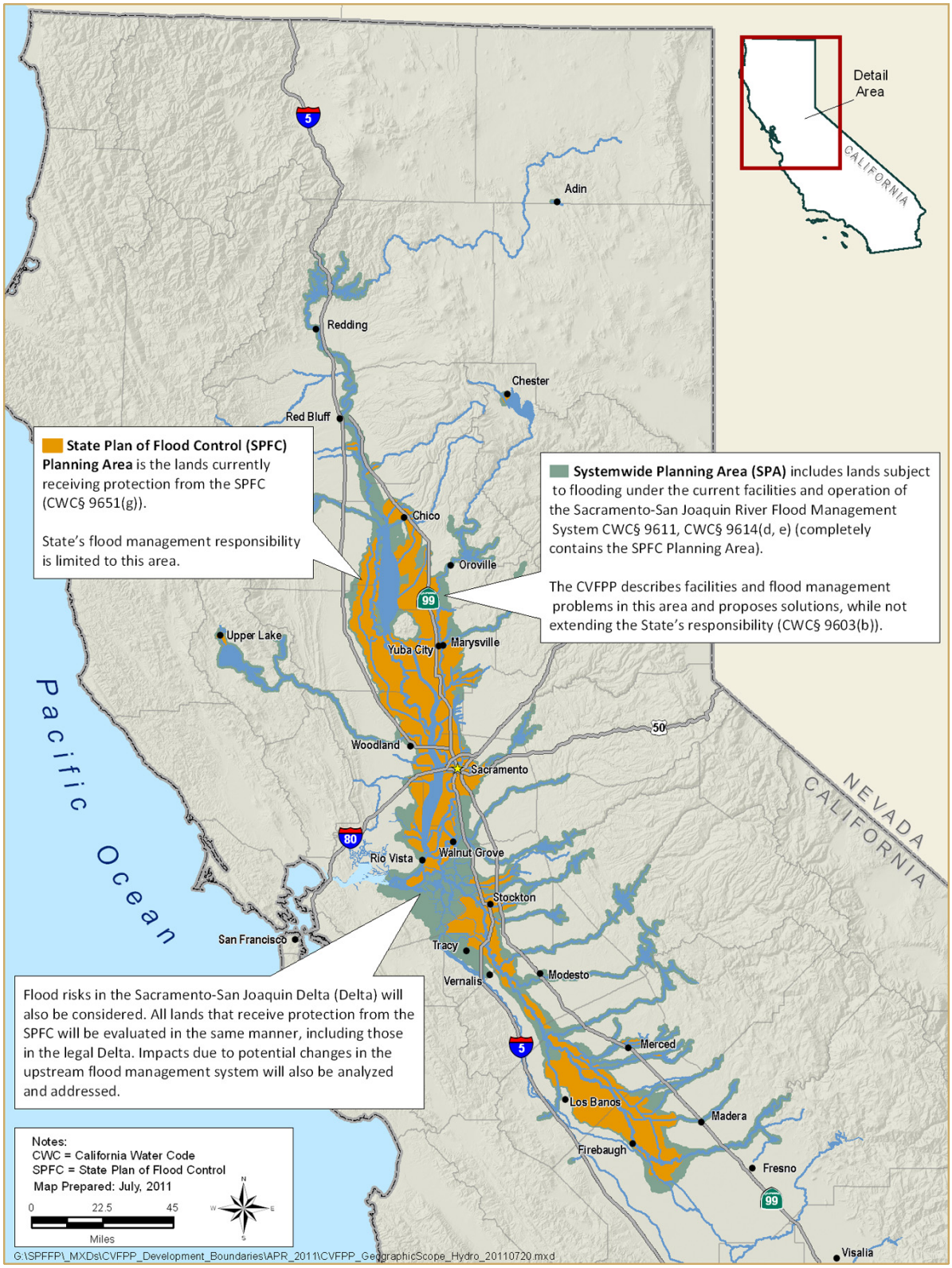

Figure 1-1. Central Valley Flood Protection Plan Planning Areas

- **Systemwide Planning Area –** This area includes the lands that are subject to flooding under the current facilities and operation of the Sacramento-San Joaquin River Flood Management System (California Water Code Section 9611). The SPFC Planning Area is completely contained within the Systemwide Planning Area which includes the Sacramento River Basin, San Joaquin River Basin, and Delta regions.

Planning and development for the CVFPP occurs differently in these planning areas. The CVFPP focused on SPFC facilities; therefore, evaluations and analyses were conducted at a greater level of detail within the SPFC Planning Area than in the Systemwide Planning Area.

Hydraulic modeling was performed for the SPFC Planning Area and Delta. This attachment focuses on the Delta. Hydraulic modeling of the Sacramento and San Joaquin river basins and the Stockton area was conducted separately and is described in Attachment 8C: Riverine Channel Evaluations. Riverine hydraulic modeling results from the Sacramento and San Joaquin river basins provided the upstream boundary conditions (inputs) for the Delta hydraulic modeling described in this attachment.

1.4 2012 CVFPP Planning Goals

To help direct CVFPP development to meet legislative requirements and address identified flood-management-related problems and opportunities, a primary and four supporting goals were developed:

- **Primary Goal** – Improve Flood Risk Management
- Supporting Goals:
  - Improve Operations and Maintenance
  - Promote Ecosystem Functions
  - Improve Institutional Support
  - Promote Multi-Benefit Projects

Modeling results in this attachment demonstrate how each of the approaches (described below) meets the primary goal.

1.5 2012 CVFPP Planning Approaches

In addition to **No Project**, three fundamentally different approaches to flood management were initially compared to explore potential improvements in the Central Valley. These approaches are not alternatives; rather, they bracket a range of potential actions and help explore trade-offs in costs, benefits, and other factors important in decision making. The approaches are as follows:

- **Achieve SPFC Design Flow Capacity** – Address capacity inadequacies and other adverse conditions associated with existing SPFC facilities, without making major changes to the footprint or operation of those facilities.
- **Protect High Risk Communities** – Focus on protecting life safety for populations at highest risk, including urban areas and small communities.
- **Enhance Flood System Capacity** – Seek various opportunities to achieve multiple benefits through enhancing flood system storage and conveyance capacity.

Comparing these approaches helped identify the advantages and disadvantages of different combinations of management actions, and demonstrated opportunities to address the CVFPP goals to different degrees.

Based on this evaluation, a **State Systemwide Investment Approach** was developed that encompasses aspects of each of the approaches to balance achievement of the goals from a systemwide perspective, and includes integrated conservation elements. Figure 1-2 illustrates this plan formulation process.

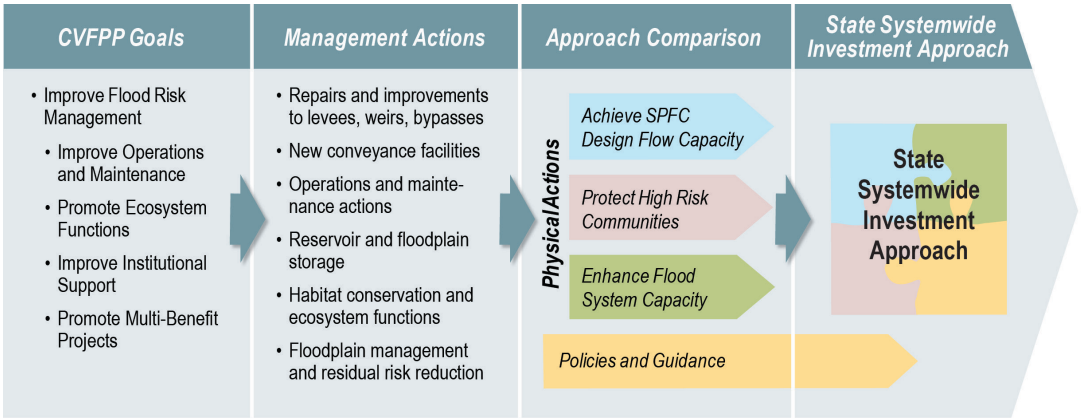

Figure 1-2. Formulation Process for State Systemwide Investment Approach

1.6 Delta Region

The Delta is the West Coast's largest estuary, encompassing approximately 1,153 square miles of waterways through which passes more than 40 percent of the freshwater in California. Sixteen of California's major rivers provide flow to the Delta as tributaries of the Sacramento River, California's largest river, or of the San Joaquin River. The Sacramento and San Joaquin rivers flow from low-lying inland valleys into the Delta – a labyrinth of islands, sloughs, canals, and channels – continuing through Grizzly Bay, Suisun Bay, and San Pablo Bay, before emptying into San Francisco Bay and then finally the Pacific Ocean. The Cosumnes, Mokelumne, and Calaveras rivers, Yolo Bypass, and numerous smaller creeks and sloughs enter the Delta in addition to the Sacramento and San Joaquin rivers. The largest source of water for the Delta is the Sacramento River, which transports about 18.3 million acre-feet (MAF) into the Delta in an average year. Additional flows from the Yolo Bypass and the San Joaquin River contribute an average of 5.8 MAF, with precipitation adding about another 1 MAF.

Freshwater from the rivers mixes with saltwater from ocean tides, creating a rich and diverse estuarine ecosystem. Because of its geographical location, the Delta serves as the collection point for much of Northern California's runoff and resulting water supplies. It is through the channels of the Delta that this water must pass to satisfy the water supply needs of the Delta, San Francisco Bay Area (Bay Area), agricultural lands of the San Joaquin River Basin, and densely populated southlands.

The flood management system in the Delta manages flows from the Sacramento and San Joaquin river basins, tributaries, and tides from San Francisco and San Pablo bays. Water management facilities in the Delta include levees around most of the islands, pumping plants, control gates, port facilities, gages used in flood and water quality forecasting, and diversion and inlet structures.

1.7 Report Organization

Organization of this document is as follows:

- Section 1 describes the purpose of this attachment, and provides an overview of the CVFPP and the Sacramento and San Joaquin river basins.

- Section 2 summarizes results and findings for CVFPP estuary hydraulic modeling.

- Section 3 describes overall CVFPP hydraulic modeling methodology, estuary model selection, and RMA Delta Model specifications.
- Section 4 provides complete results for the estuary hydraulic analysis by CVFPP approach.
- Section 5 contains references for the sources cited in this document.
- Section 6 lists abbreviations and acronyms used in this document.

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2.0 Results Summary and Findings

Results from hydraulic modeling of the Delta are summarized in Figures 2-1 through 2-12, which map the changes in stage between the No Project condition and the four CVFPP approaches throughout the system.

Maps are only included for AEPs of 2 percent, 1 percent, and 0.5 percent (50-, 100-, and 200-year return period) because the flood management system doesn't exhibit significant differences between the No Project and the four approaches for the 10 percent and 4 percent (10- and 25-year return period), and similarly the 0.2 percent AEP flood (500-year return period) overwhelms the flood management system in all cases.

2.1 Achieve SPFC Design Flow Capacity Approach

Figures 2-1 through 2-3 indicate the changes in Delta stages that would result from repairing or improving all SPFC levees to meet their design flows (Section 3.5, Tables 3-1 and 3-2) as specified by the 55/57 design profiles. Overall, for all of the AEPs there would be fewer upstream levee breaks, resulting in increased flows and higher water surface elevations in the Delta, particularly in the areas where the Sacramento and San Joaquin rivers enter the Delta.

2.2 Protect High Risk Communities Approach

Figures 2-4 through 2-6 indicate the changes in stage that would result from repairing or improving all urban levees to meet the 0.5 percent AEP (200-year) design criteria (Section 3.6, Tables 3-1 and 3-2), and providing increased protection to selected small communities. Since this approach would improve only urban and small community levees, other levees would be untouched and function as in the No Project condition. Stage increases of a foot or less would be seen on the Delta as a result of increased protection for upstream urban areas.

2.3 Enhance Flood System Capacity Approach

Figures 2-7 through 2-9 indicate the changes in stage that would result from modifying the flood management system as described in Section 3.7 and shown in Tables 3-1 and 3-2. Key components of the approach are added upstream reservoir storage, improving SPFC levees to their design flow capacity, improving urban levees to pass the 0.5 AEP flood, widened and new bypasses, levee setbacks, and floodplain storage. The added upstream and floodplain storage in the Sacramento River Basin would result in lower stages entering and in the interior of the Delta for all AEPs. The Paradise Cut Bypass enlargement and Roberts Island floodplain storage lower stages on the San Joaquin River from Paradise Cut to Stockton.

2.4 State Systemwide Investment Approach

Figures 2-10 through 2-12 indicate the changes in stage resulting from repairing or improving all urban levees to meet the 0.5 percent AEP (200-year) design criteria and other improvements in the State Systemwide Investment Approach (Section 3.8, Tables 3-1 and 3-2). Because this approach would improve only urban levees, other levees would be untouched and function the same as the No Project condition. Stages in the Delta as a result of this approach would be the same as or lower than the No Project condition.

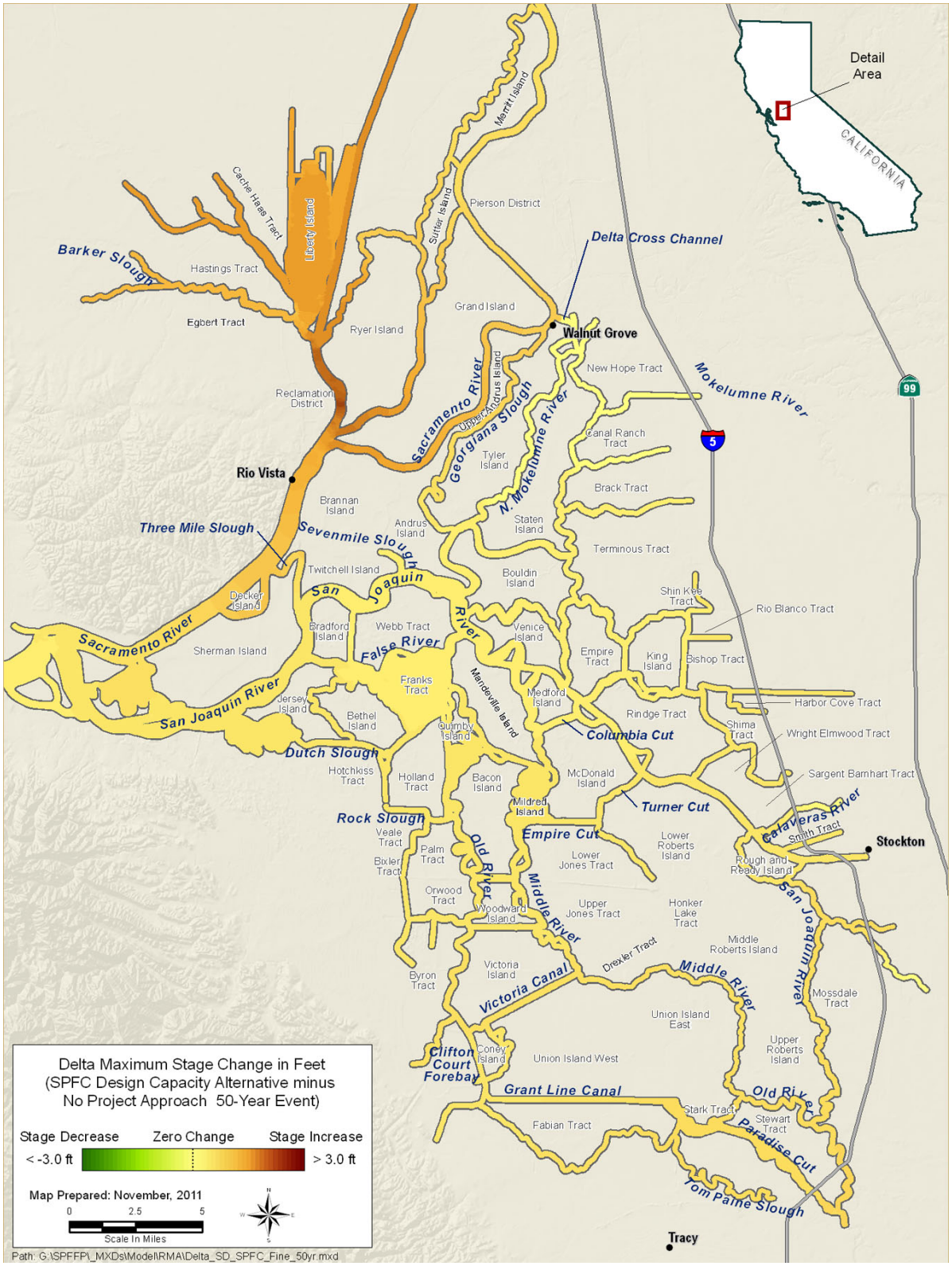


Figure 2-1. Stage Changes from No Project Condition to Achieve SPFC Design Flow Capacity Approach – 2 Percent AEP (50-year)

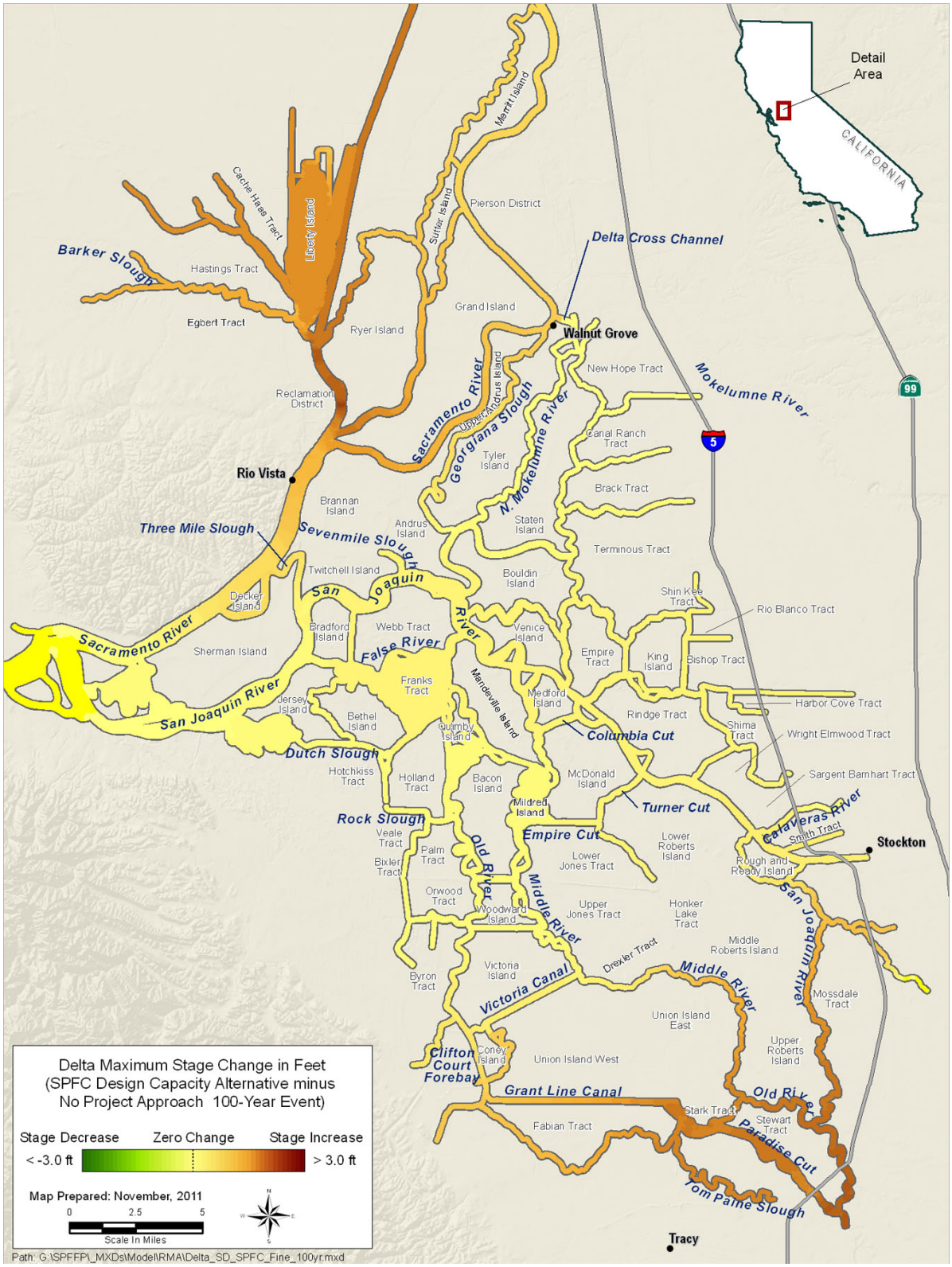


Figure 2-2. Stage Changes from No Project Condition to Achieve SPFC Design Flow Capacity Approach – 1 Percent AEP (100-year)

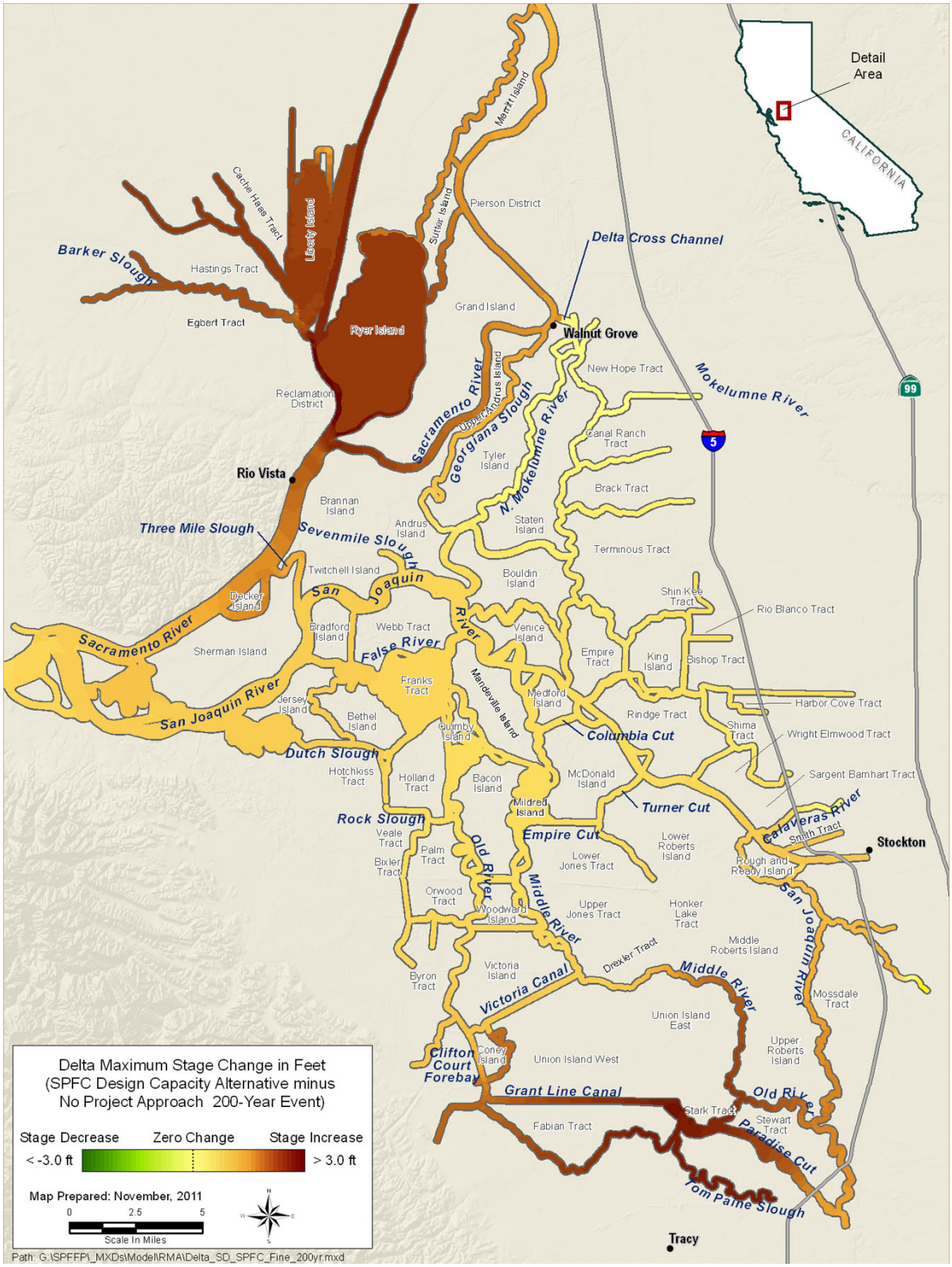


Figure 2-3. Stage Changes from No Project Condition to Achieve SPFC Design Flow Capacity Approach – 0.5 Percent AEP (200-year)

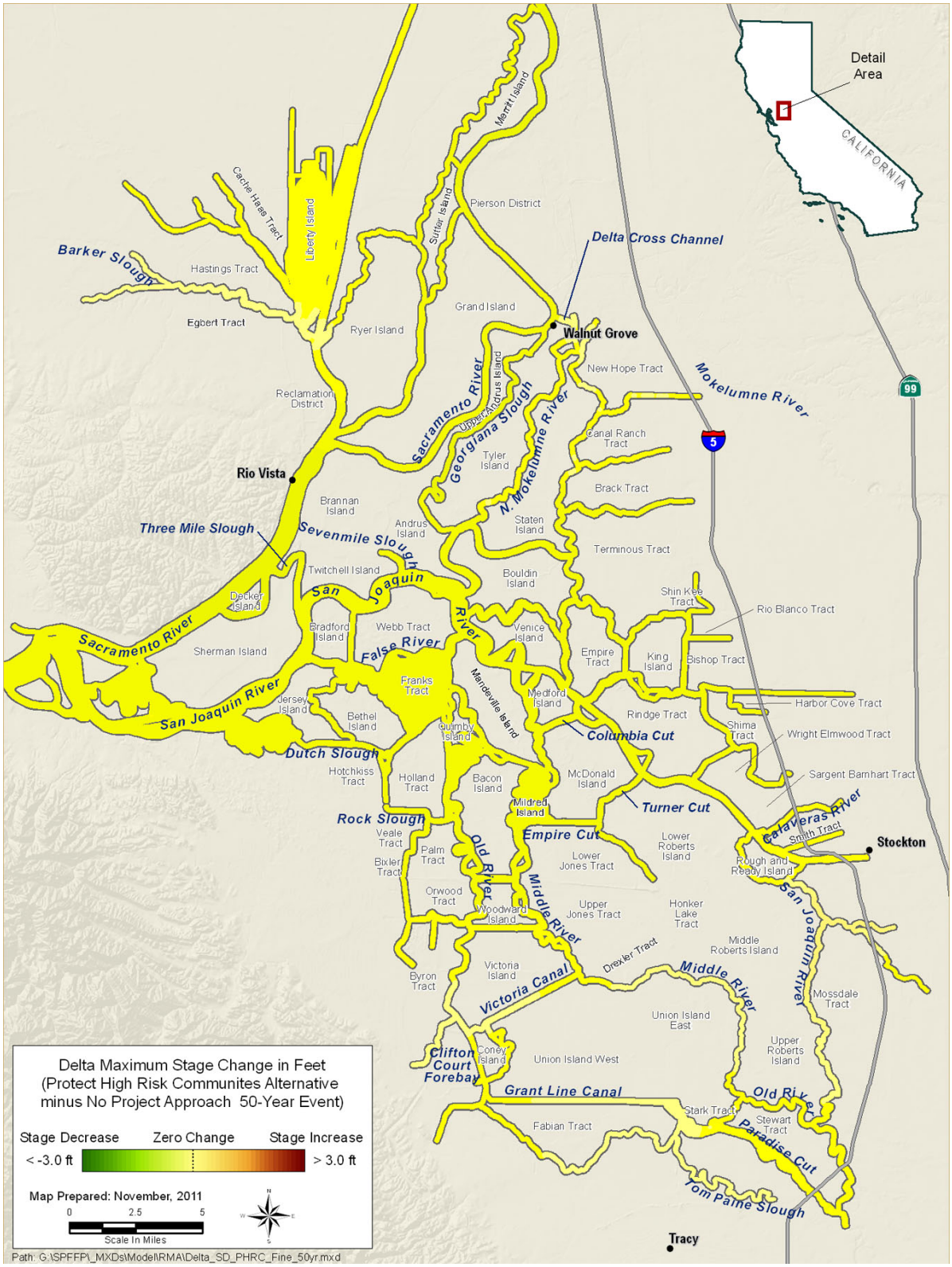


Figure 2-4. Stage Changes from No Project Condition to Protect High Risk Communities Approach – 2 Percent AEP (50-year)

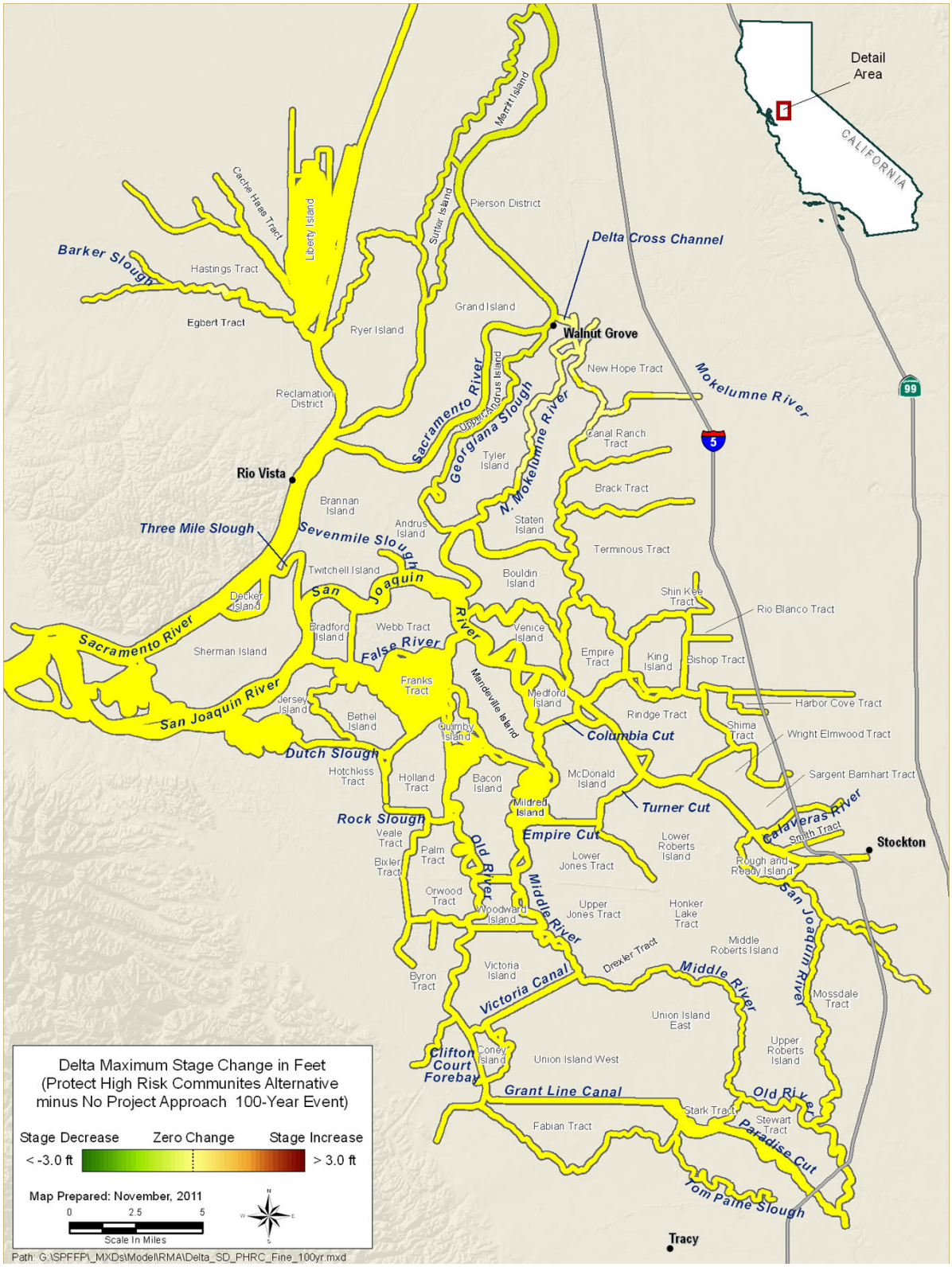


Figure 2-5. Stage Changes from No Project Condition to Protect High Risk Communities Approach – 1 Percent AEP (100-year)

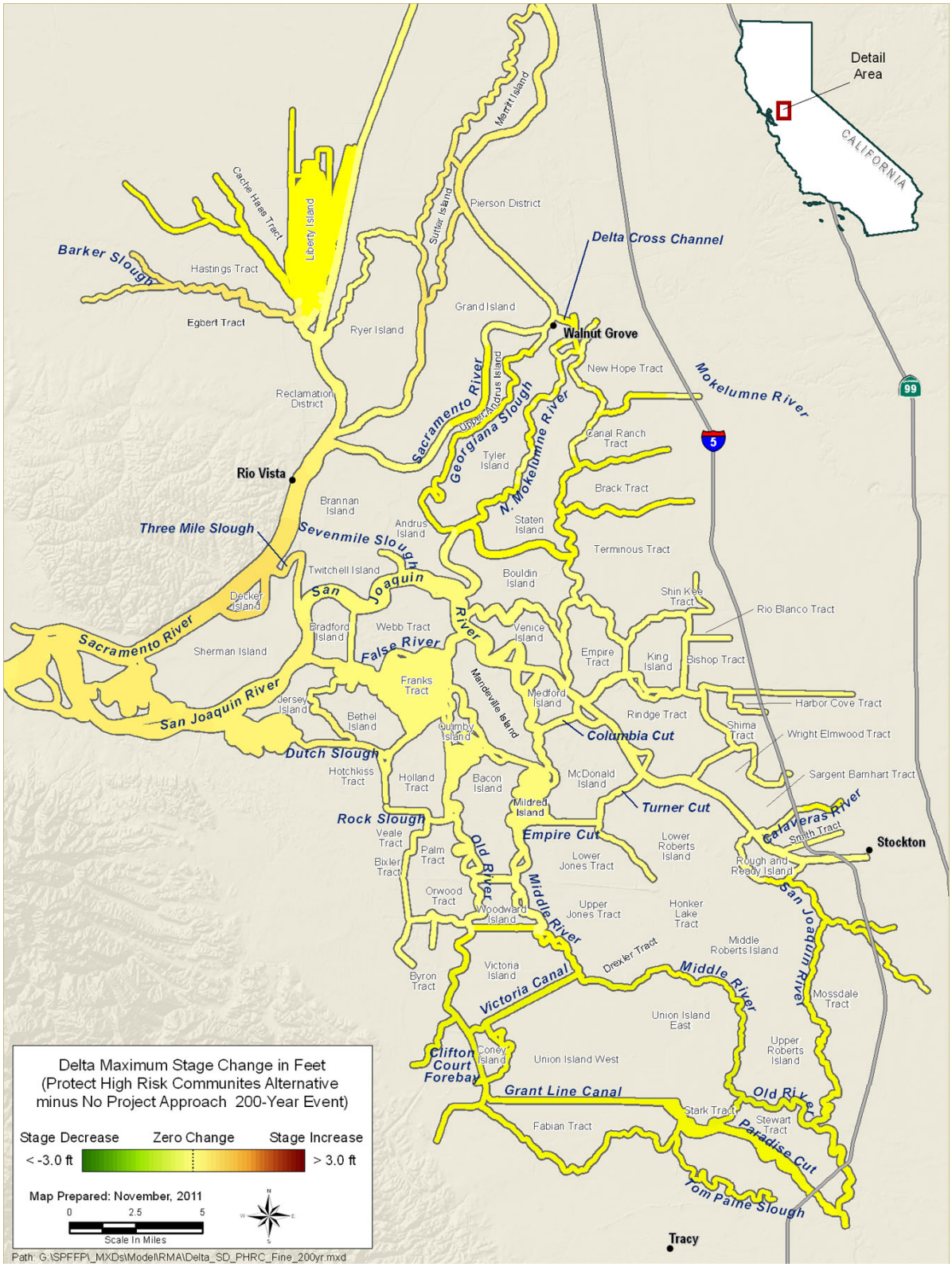


Figure 2-6. Stage Changes from No Project Condition to Protect High Risk Communities Approach – 0.5 Percent AEP (200-year)

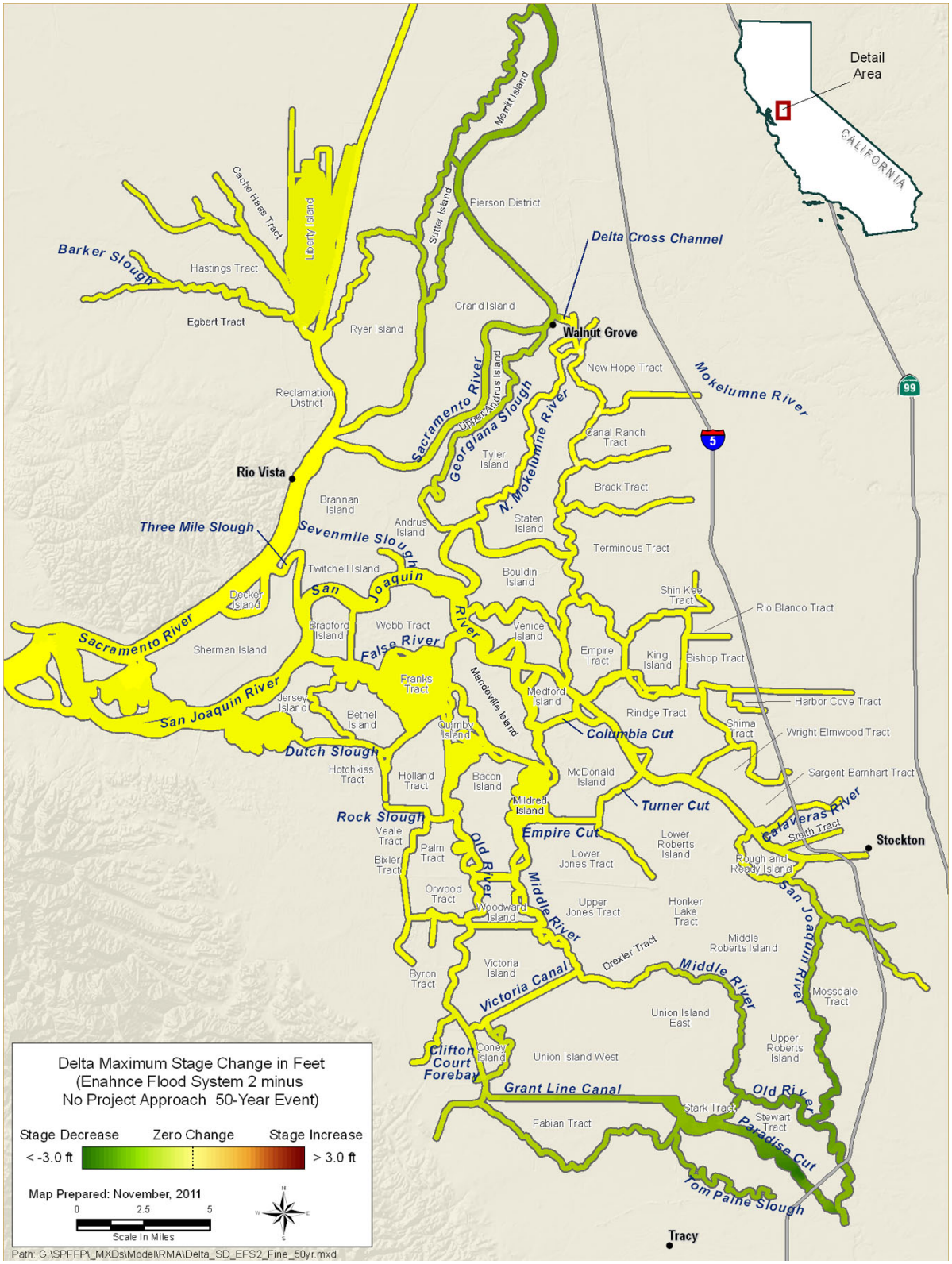


Figure 2-7. Stage Changes from No Project Condition to Enhance Flood System Capacity Approach – 2 Percent AEP (50-year)

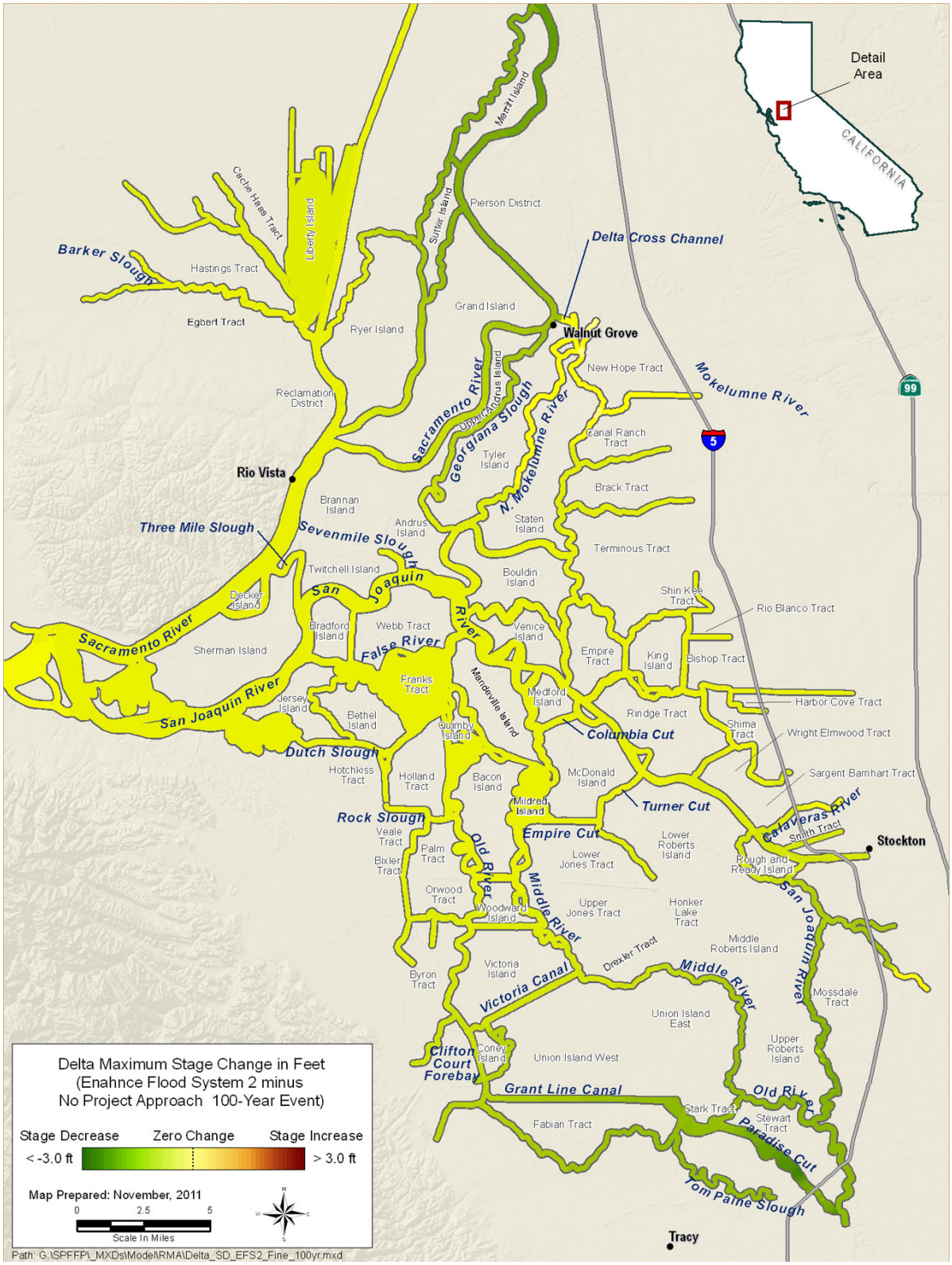


Figure 2-8. Stage Changes from No Project Condition to Enhance Flood System Capacity Approach – 1 Percent AEP (100-year)

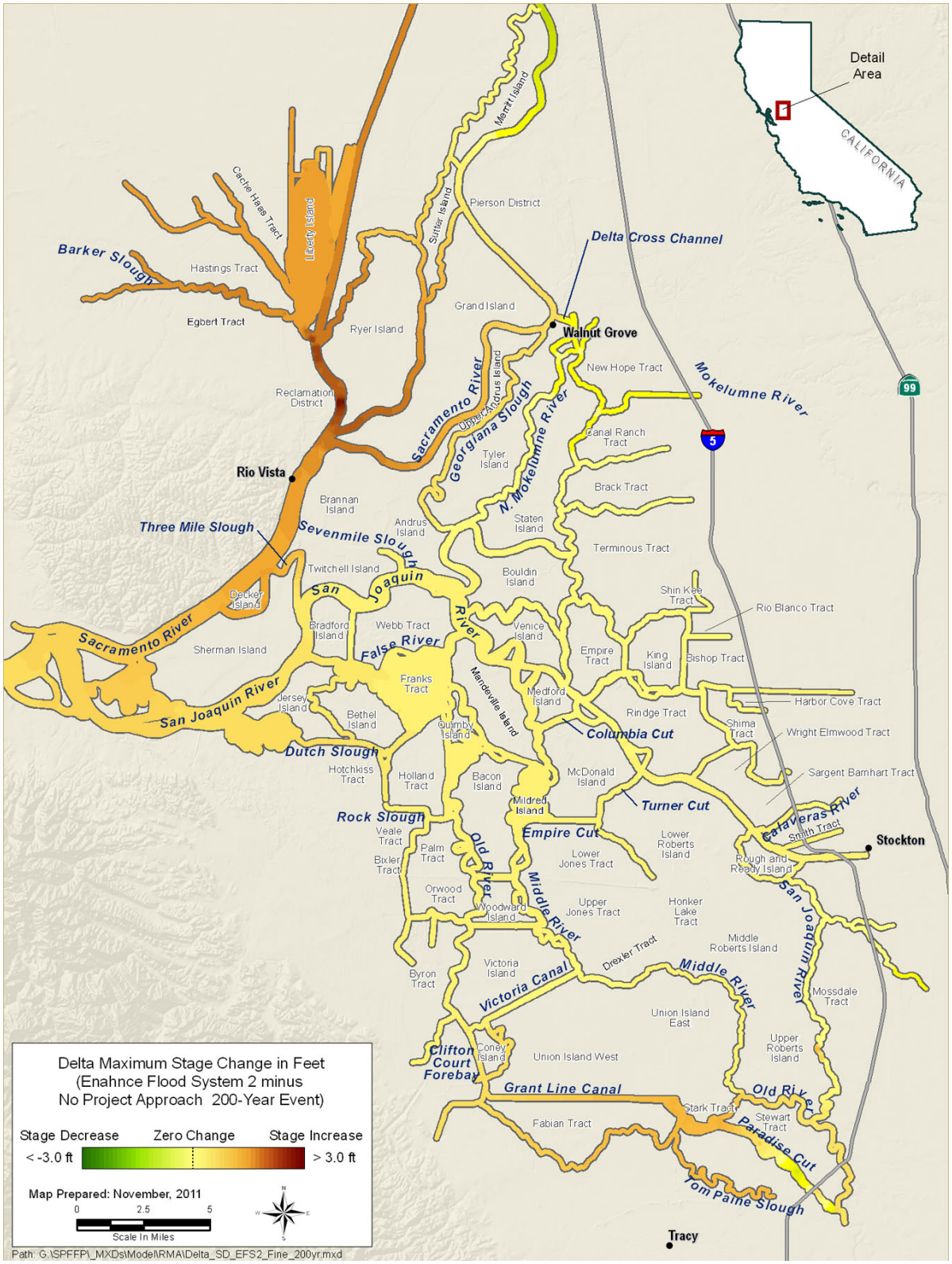


Figure 2-9. Stage Changes from No Project Condition to Enhance Flood System Capacity Approach – 0.5 Percent AEP (200-year)

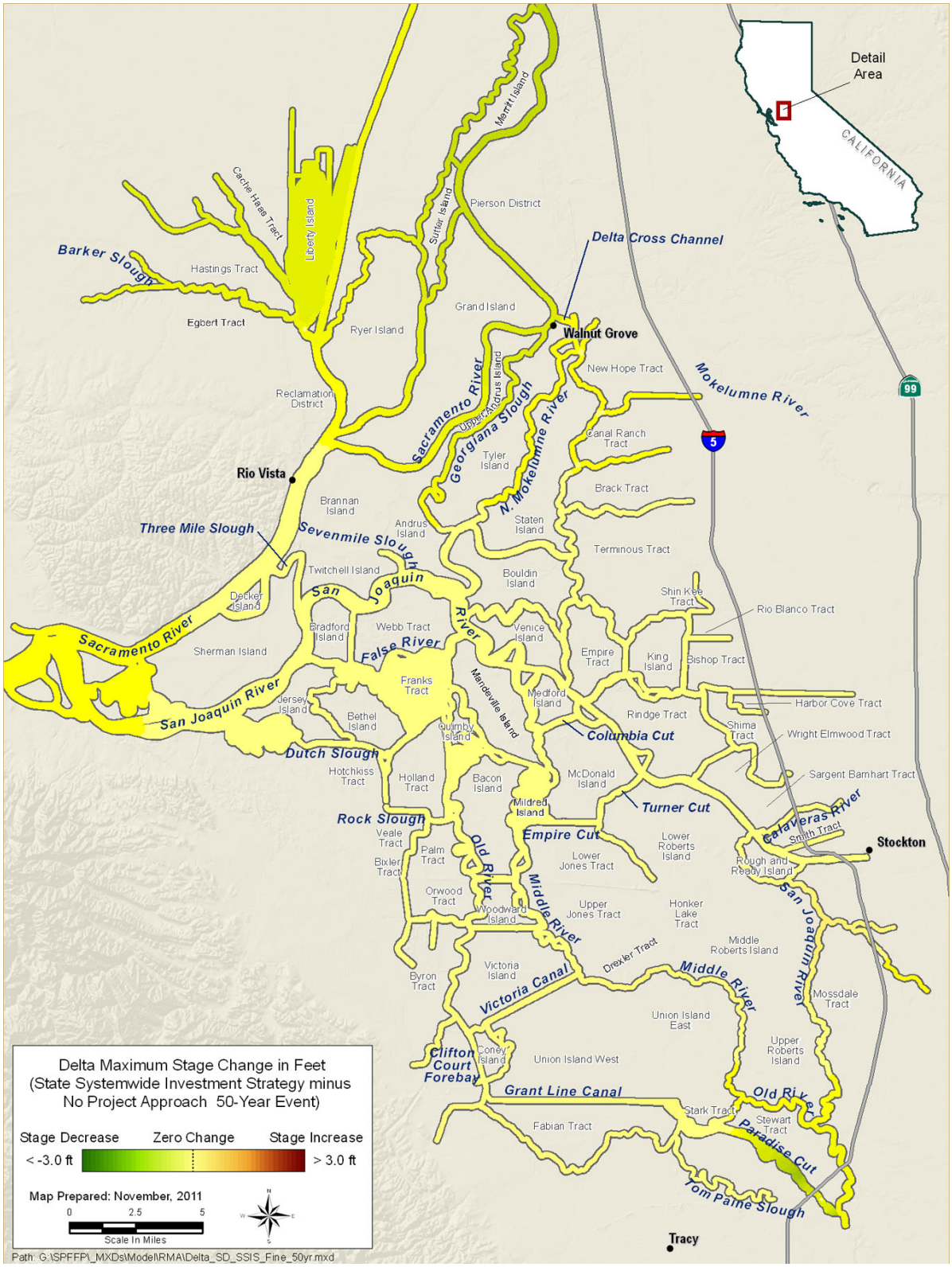


Figure 2-10. Stage Changes from No Project Condition to State Systemwide Investment Approach – 2 Percent AEP (50-year)

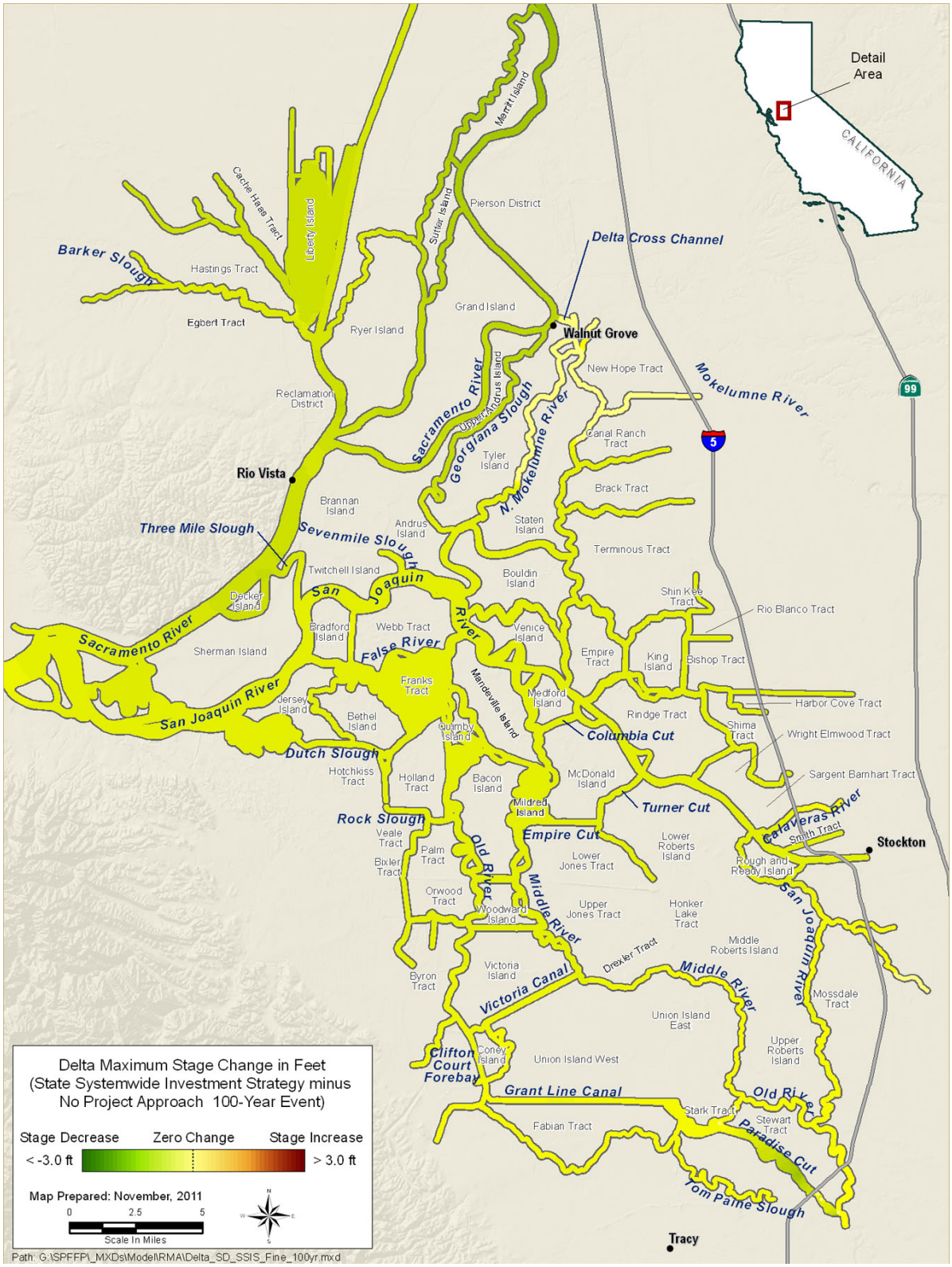


Figure 2-11. Stage Changes from No Project Condition to State Systemwide Investment Approach – 1 Percent AEP (100-year)

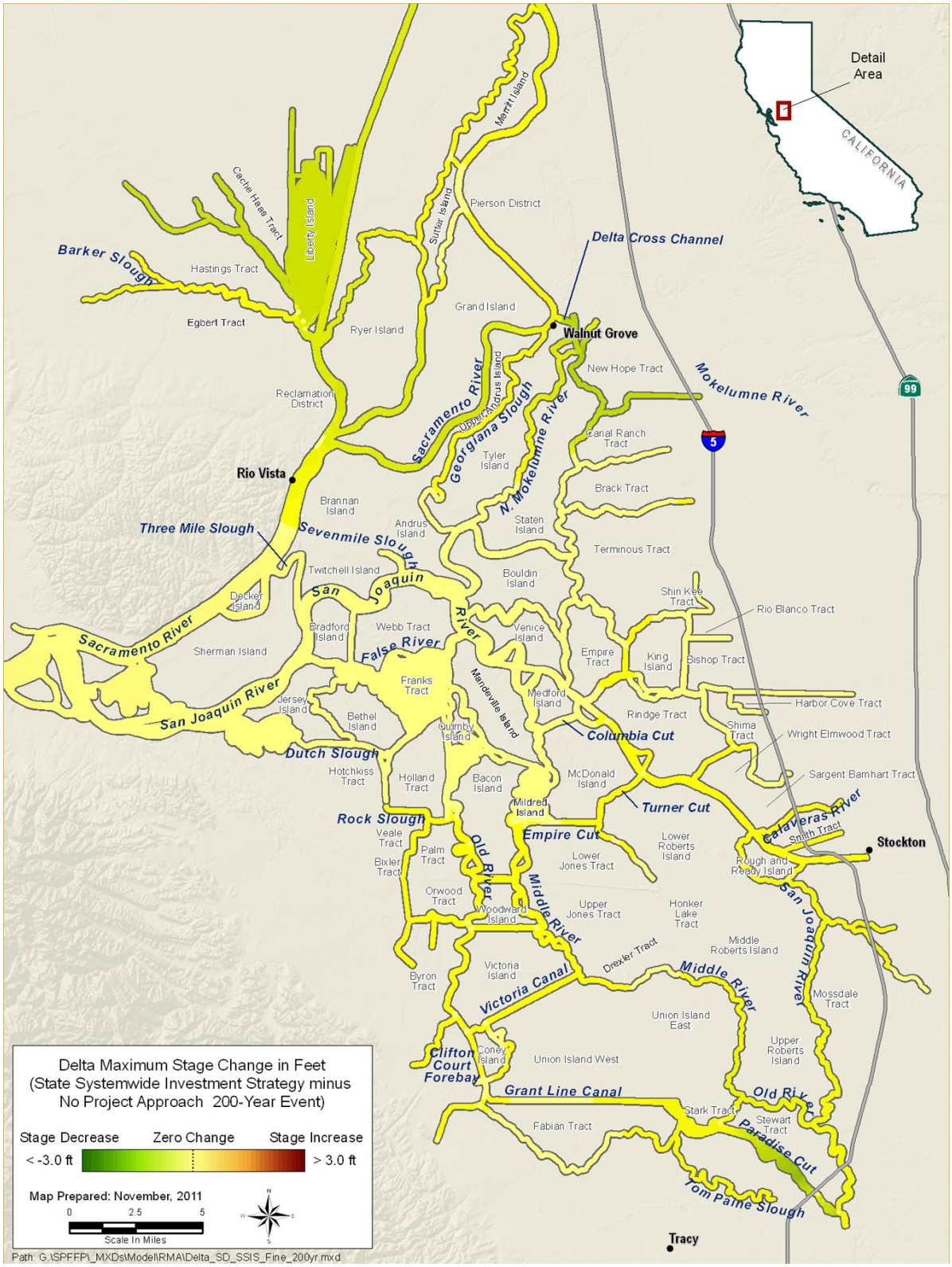


Figure 2-12. Stage Changes from No Project Condition to State Systemwide Investment Approach – 0.5 Percent AEP (200-year)

3.0 Methodology

This section provides an overview of the CVFPP modeling framework, model selection, the RMA Delta Model, and modeling assumptions for the No Project condition and each CVFPP approach.

It is important to note that the hydraulic modeling conducted using the RMA Delta Model is a deterministic process that simulates levee breaches based on data provided regarding levee performance. Hydraulic modeling cannot and does not predict the location of actual levee breaches.

3.1 CVFPP Modeling Overview

Figure 3-1 shows the overall hydraulic modeling schematic for the CVFPP. With defined boundary conditions (including upstream hydrographs to represent storm events, downstream tailwater stage, levee breach scenarios, etc.), riverine hydraulic conditions were simulated to generate hydrographs that would be the upstream boundary conditions for the Delta hydraulic model. The Delta hydraulic model was then used to estimate the water stage for locations inside the Delta. Details of the riverine hydraulic modeling are contained in Attachment 8C: Riverine Channel Evaluations. All flows from areas protected by the SPFC eventually pass through the Delta; therefore, estuary hydraulic modeling using existing tools was an important part of the hydraulic analyses needed to support the CVFPP development.

3.2 Model Selection

Two existing hydraulic models were evaluated for use in determining water stages in the Delta: the Delta Simulation Model II (DSM2) and the RMA Delta Model.

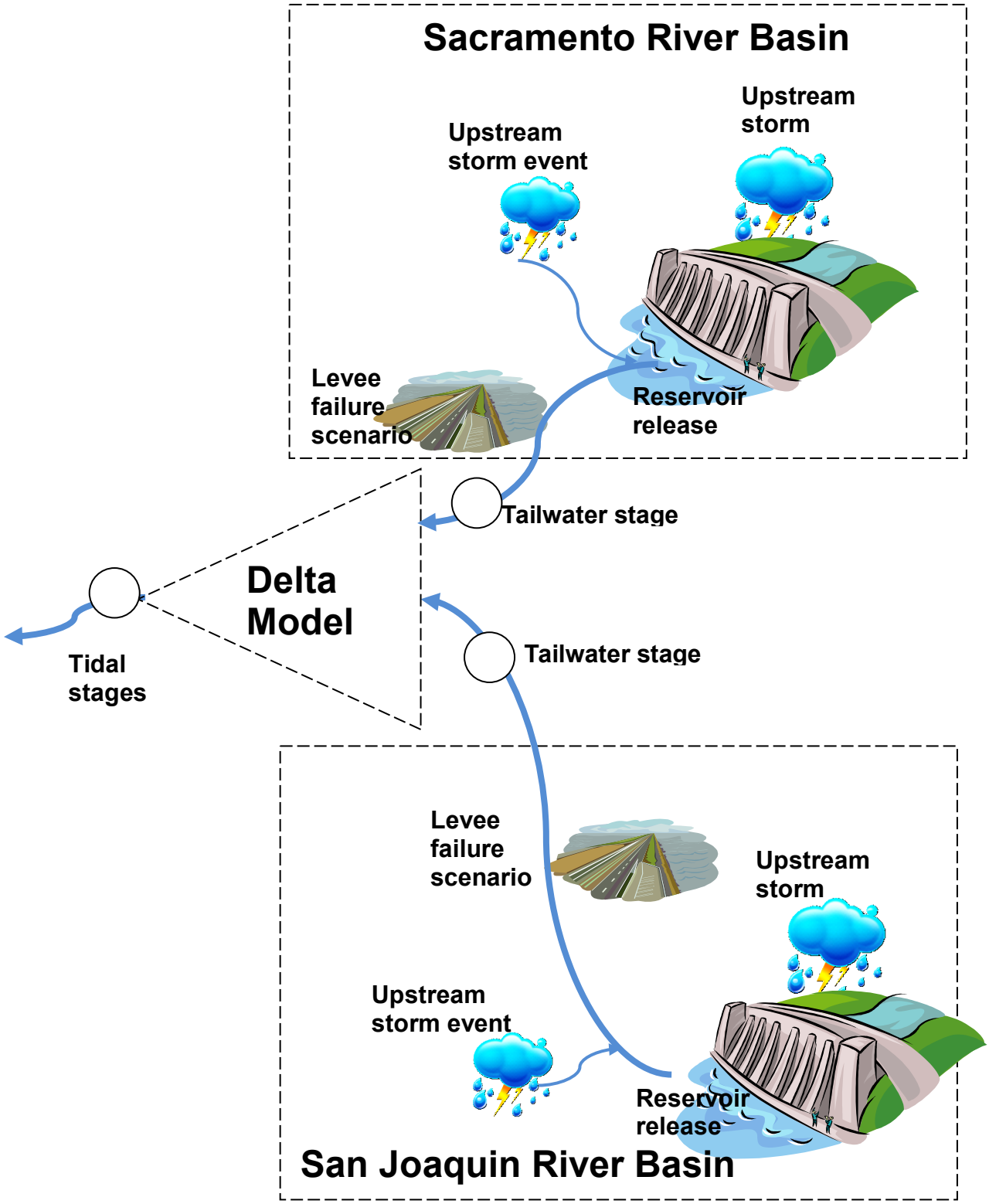


Figure 3-1. Schematic of CVFPP Hydraulic Modeling

DSM2, developed by DWR in the early 1990s, is a branched one-dimensional (1-D), physically based numerical model of the Delta. DSM2-Hydro, the hydrodynamics module, is derived from the U.S. Geological Survey (USGS) Four Point Model. Key DSM2 inputs for the hydrodynamic module include tidal stage at Martinez, boundary inflows (e.g., Sacramento and San Joaquin rivers, Yolo Bypass, eastside streams), and operations of flow-control structures (e.g., Clifton Court Forebay gates, Delta Cross Channels). DSM2 uses the Delta Island Consumptive Use (DICU) Model to develop agricultural diversions and return flow to each of 142 Delta subareas. The DICU follows the seasonal pattern of irrigation diversions during the summer and drainage return flows from winter runoff.

The RMA Delta Model uses finite element analysis to enable a mixed representation of two-dimensional (2-D) depth-averaged elements and 1-D channel elements. For systems such as the San Francisco Bay-Delta, the 2-D depth-averaged elements are typically used to represent the open waters of the bays and large river channels while the 1-D elements are used for reproducing flow and transport for simple channels in the Delta (RMA, 2005). Boundary conditions and model extents for the RMA Delta Model are similar to DSM2. The RMA Delta Model also uses DICU Model outputs for agricultural diversions and return flows into the Delta.

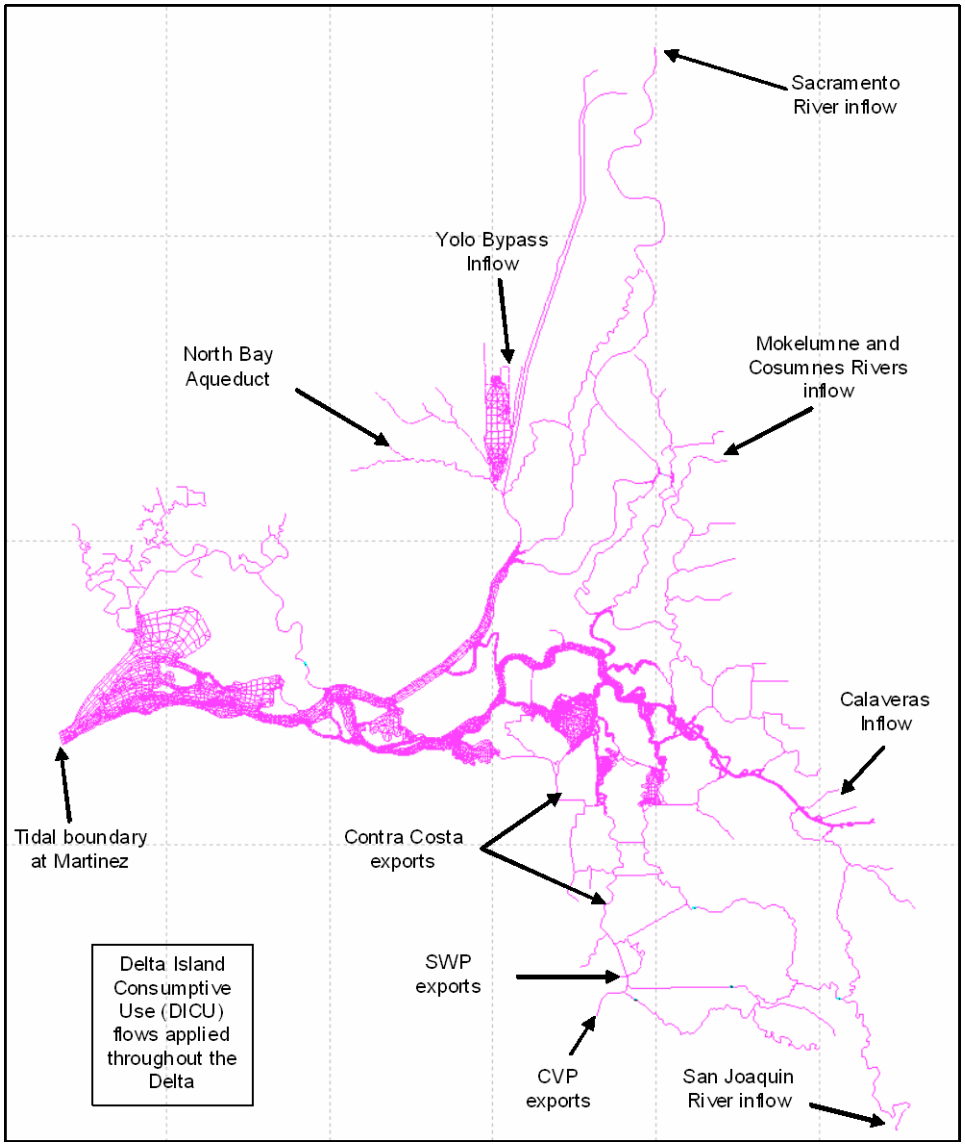
The RMA Delta Model can explicitly simulate levee breaches and inundation of islands to estimate interior flood depth using available elevation data for levee crest and Delta island topography. Therefore, the RMA Delta Model was selected for this CVFPP estuary channel evaluation to estimate Delta in-channel water stage and flooding inside islands.

3.3 RMA Delta Model Overview

The RMA Delta Model is a calibrated finite element model for surface water hydrodynamics simulation to compute 2-D depth-averaged velocity and water surface elevation. This model encompasses the major rivers and channels of the Delta system. Figure 3-2 shows a schematic of the RMA Delta Model (RMA, 2005).

The RMA Delta Model employs 2-D depth-averaged elements to represent large open water areas of the system, such as the area in and around Franks Tract, the Sacramento-San Joaquin River confluence, and Suisun Bay. For this CVFPP estuary channel evaluation, the 2-D depth-averaged elements were extended on the Sacramento River near Rio Vista, and on the San Joaquin River to the Port of Stockton.

Other channels of the Delta are represented by 1-D channel elements for simplified representations of channel cross sections in trapezoidal shape. The 1-D elements have a provision for off-channel storage or an ineffective flow area. This feature is typically used to represent shallow water or marsh areas bordering the main flow channel. Off-channel storage is also defined with a simplified geometry.



Source: RMA, 2005

Figure 3-2. Schematic of RMA Delta Model

By default, the outer boundary or shoreline encompassing the 2-D network elements are treated as "infinite walls" where no overtopping flow is allowed. This is also true for flow in the 1-D channel elements. Top of levee (TOL) elevations can be changed with time on a node-by-node basis to allow complete simulation of a breaching event and later levee repair.

3.4 Modeling Assumptions for No Project Condition

Tables 3-1 and 3-2 show modeling assumptions for the Sacramento and San Joaquin river basins, respectively, for the No Project condition and all of the CVFPP approaches. The following sections describe assumptions in the RMA Delta Model for the CVFPP No Project condition.

3.4.1 Paradise Cut Modifications

The Lower San Joaquin River (LSJR) model was developed and calibrated for River Islands at Lathrop using the Hydrologic Engineering Center River Analysis System (HEC-RAS). The model was constructed by converting a portion of the San Joaquin River Comprehensive Study UNET Model to the HEC-RAS platform, with additional refinements in floodplain geometry and hydraulic connections. Figure 3-3 shows the extent of the LSJR HEC-RAS model (MBK, 2006).

Geometry data in the RMA Delta Model were modified to reflect refinements made in the LSJR HEC-RAS Model, as follows:

- The junction of Paradise Cut (see Figure 3-3) and the San Joaquin River were modeled with 2-D features to better simulate weir flow.
- Channel representation for the junction of Grant Line Canal/Old River/Paradise Cut was refined and extended.

Table 3-1. Summary of Sacramento River Basin Modeling Assumptions

Element	Description	No Project (NPRJ)	Achieve SPFC Design Flow Capacity (SPFC)	Protect High Risk Communities (PHRC)	Enhance Flood System Capacity (EFSC)	State Systemwide Investment (SSIA)
Levee Setback	Sacramento River RM 199.5 to 197				√	
	Sacramento River RM 169.5 to 111.25				√	
	Feather River RM 24.5 to 0				√	
Levee Improvement	Restore 1955/1957 design levee: Assume levee breach at top of levee in hydraulic model		√		√	
	Fix urban area levee: Assume levee breach at top of levee in hydraulic model			√	√	√
	TRLIA levee improvement	√	√	√	√	√
	Marysville levee improvement	√	√	√	√	√
	Natomas levee improvement	√	√	√	√	√
Bypass	Widen Yolo Bypass[1]				√	√
	Widen Sacramento Bypass and Gates				√	
	Widen Sutter Bypass				√	√
	Feather to Butte Basin (Biggs) Bypass				√	√
Reservoir Storage and Operations	Folsom Dam Joint Federal Project	√	√	√	√	√
	Lake Oroville: Modify Lake Oroville release schedule				√	
	New Bullards Bar and Lake Oroville: Implement coordinated operation of the Feather-Yuba River Basin				√	
Floodplain Storage	Sutter Butte Basin				√	
	Feather River Basin				√	
	Elkhorn				√	
	Merritt Island				√	

Notes:
55/57 levee design profile was the design standard for the State Plan of Flood Control.
[1] Use off-stream storage to model levee setback.

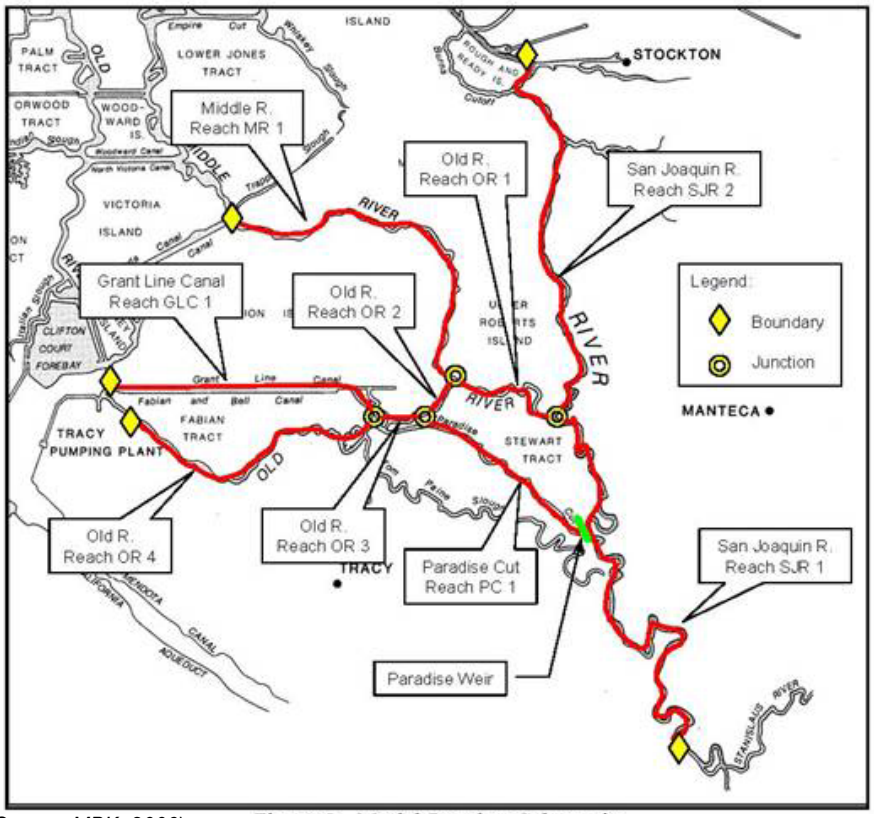
Key:
EFSC = Enhance Flood System Capacity Approach
NPRJ = No project
PHRC = Protect High Risk Communities Approach
RM = River Mile
SPFC = State Plan of Flood Control
SSIA = State Systemwide Investment Approach
TRILIA = Three Rivers Levee Improvement Authority

Table 3-2. Summary of San Joaquin River Basin Modeling Assumptions

Element	Description	No Project (NPRJ)	Achieve SPFC Design Flow Capacity (SPFC)	Protect High Risk Communities (PHRC)	Enhance Flood System Capacity (EFSC)	State Systemwide Investment (SSIA)
Levee Setback	SJR RM115 to 99				√	
	SJR RM 81.5 to 72.5				√	
Levee Improvement	Restore 55/57 levee design profile: Assume levees breach at top of levee in hydraulic model		√		√	
	Fix urban area levees: Assume levees breach at top of levee in hydraulic model			√	√	√
	Restore bypass levees: Assume levees breach at top of levee in hydraulic model		√		√	
Bypass	Widen Paradise Cut				√	√
Reservoir Storage and Operations	New Don Pedro Reservoir: Increase flood storage allocation by 230,000 acre-feet				√	
	Friant Dam and Millerton Lake: Increase flood storage allocation by 60,000 acre-feet				√	
	New Exchequer Dam and Lake: Increase flood storage allocation by 100,000 acre-feet				√	
Floodplain Storage	Roberts Island				√	
	San Joaquin River: between Merced and Tuolumne rivers				√	
	San Joaquin River: between Tuolumne River and Stanislaus River				√	

Note:
55/57 levee design profile was the design standard for the State Plan of Flood Control

Key:
EFSC = Enhance Flood System Capacity Approach
NPRJ = No project
PHRC = Protect High Risk Communities Approach
RM = river mile
SJR = San Joaquin River
SPFC = State Plan of Flood Control
SSIA = State Systemwide Investment Approach
TRLIA = Three Rivers Levee Improvement Authority



(Source: MBK, 2006)

Figure 3-3. Schematic of LSJR HEC-RAS Model for River Islands

3.4.2 Boundary Conditions

UNET model outputs for the Sacramento and San Joaquin river basins for the No Project condition were incorporated into the RMA Delta Model as upstream boundary conditions. These upstream boundary conditions from UNET models were applied into the RMA Delta Model at the following locations (see Figure 3-4):

- Sacramento River downstream from American River confluence
- Yolo Bypass at three locations near Liberty Island
- San Joaquin River at Vernalis[1]

[1] UNET flow for the San Joaquin River upstream from Paradise Cut Weir was applied in the RMA Delta Model at Vernalis (about 13 river miles upstream from the Paradise Cut Weir) by shifting the time-series 10 hours earlier to address the lag time. Also, the RMA Delta Model assumed there was no levee breach along the San Joaquin River between Vernalis and the Paradise Cut Weir. Such a levee breach was addressed in the UNET model.

Flows entering the Delta from eastside streams (collectively referred to as Delta tributaries) were incorporated into the RMA Delta Model based on hydrographs from the Sacramento and San Joaquin River Basins Comprehensive Study (Comprehensive Study)(USACE, 2002a) for six flood events (AEPs of 10, 4, 2, 1, 0.5, and 0.2 percent) to represent flows for: Mokelumne, Cosumnes, and Calaveras rivers, and French Camp Slough (see Figure 3-4).

Historical records from January 1997 were shifted 20 days forward to match the UNET model simulation period (i.e., historical records of January 1 were shifted to January 21 in the RMA Delta Model) and were used as boundary conditions for the following:

- Downstream tidal stage at Martinez
- Central Valley Project (CVP) and State Water Project (SWP) exports
- Operations of control structures in the Delta[2]

[2] Control structures in the Delta of interest include Suisun Marsh Salinity Control gate, Delta Cross Channel, Old River near Tracy barrier, temporary barrier at the head of Old River, Middle River temporary barrier, Clifton Court Forebay Gates, Grant Line Canal barrier, and Rock Slough tide gate.

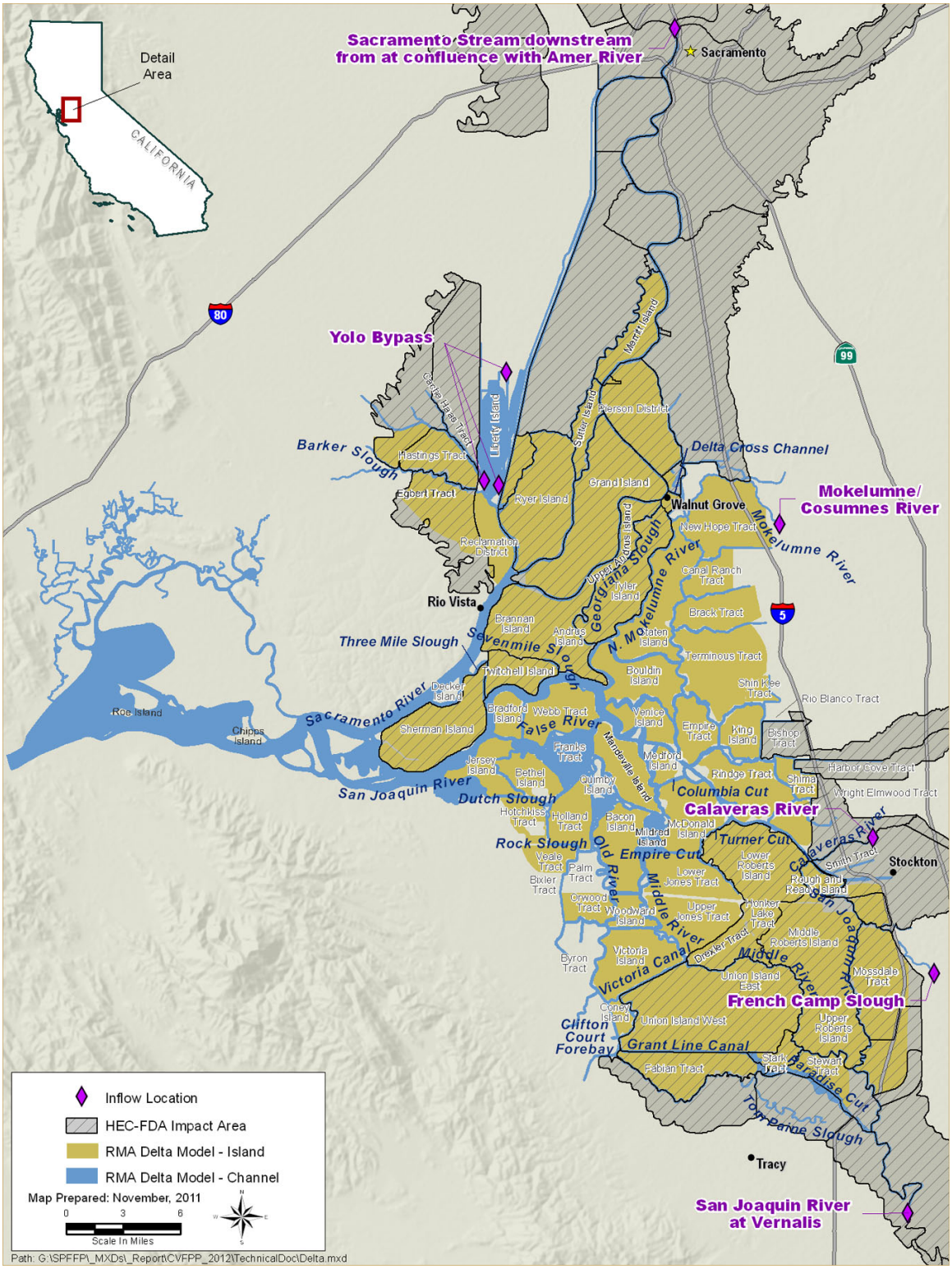


Figure 3-4. Upstream Boundary Inflow Locations for RMA Delta Model

3.4.3 Delta Inflow Annual Exceedence Probabilities

The riverine hydraulic model of the Sacramento River Basin has five storm centerings (Sacramento, Ord Ferry, Shasta, Yuba, and American River) and the San Joaquin River Basin also has five centerings (Vernalis, Newman, El Nido, Merced, and Friant) for six flood events (AEPs of 10, 4, 2, 1, 0.5, and 0.2 percent) (see Attachment 3: Riverine Channel Evaluations).

Only hydrographs from the Sacramento and Vernalis centerings were used as inputs into the RMA Delta Model. These two storm centerings generated the largest peak inflows into the Delta for flood events from the Sacramento and San Joaquin river basins.

The exceedence frequency of storm inflows from the Sacramento and San Joaquin river basins into the Delta from a given storm are not likely to be exactly the same, but inflows from the two basins do have some meteorological connectivity. To help identify a reasonable Sacramento-San Joaquin river inflow coincident probability to use for the Delta channel evaluation, two approaches were taken: review of historical inflows and hydraulic sensitivity analysis.

Historical Flow Review

Historical full natural daily flows from October 1, 1921, through November 18, 1997 (i.e., water years 1922 through 1997), were evaluated at the following locations:

- Sacramento River at latitude Sacramento
- San Joaquin River near Vernalis
- Cosumnes River at Michigan Bar
- Mokelumne River at Camanche Reservoir
- Calaveras River at New Hogan Reservoir

For the historical flow review, a summation of flows from the five sources listed above was used to represent total Delta inflows for each day. An analysis was made of the coincidence of Delta river source inflows with total Delta inflow; results are shown in Figure 3-5. For each water year, the date of maximum Delta total inflow was identified and the recurrence interval was calculated. Flows for the same day on the Sacramento and San Joaquin rivers were identified and the corresponding recurrence interval was then determined for each of those flows and plotted with the total Delta inflow recurrence (see Figure 3-5) to show the correlations.

Figure 3-5 shows that total Delta inflows historically had the highest correlation with Sacramento River flows; a 1 percent AEP event (100-year) in the Delta could be caused by a 1.11 percent AEP event (90-year) on the Sacramento River, which would coincide with a San Joaquin River flood of having an AEP of roughly 1.25 percent (80-year). The differences in coincident AEP are due in part to different timing of peak flows; San Joaquin River flow at Vernalis typically peaked 1 day later than the Sacramento River flow at latitude Sacramento while the Delta tributaries peaked 1 day earlier.

Hydraulic Sensitivity Analysis

A sensitivity analysis was performed using the RMA Delta Model to understand the sensitivity of Delta stages to varying Sacramento and San Joaquin river inflows at the following locations (Figure 3-6):

- Old River near Tracy Temporary Barrier
- Middle River near State Highway 4
- Middle River at Bacon Island
- San Joaquin River at Rindge Pump
- Head of Old River
- San Joaquin River at Jersey Point
- Sacramento River at Rio Vista
- Sacramento River above Delta Cross Channel

The sensitivity analysis shown in Figure 3-7 included the following storm events under the No Project condition:

- A 1 percent AEP flood (100-year) for the Sacramento and San Joaquin river basins
- A 1 percent AEP flood (100-year) for the Sacramento River Basin and a 2 percent AEP flood (50-year) for the San Joaquin River Basin
- A 2 percent AEP flood (50-year) for the Sacramento River Basin and a 1 percent AEP flood (100-year) for the San Joaquin River Basin

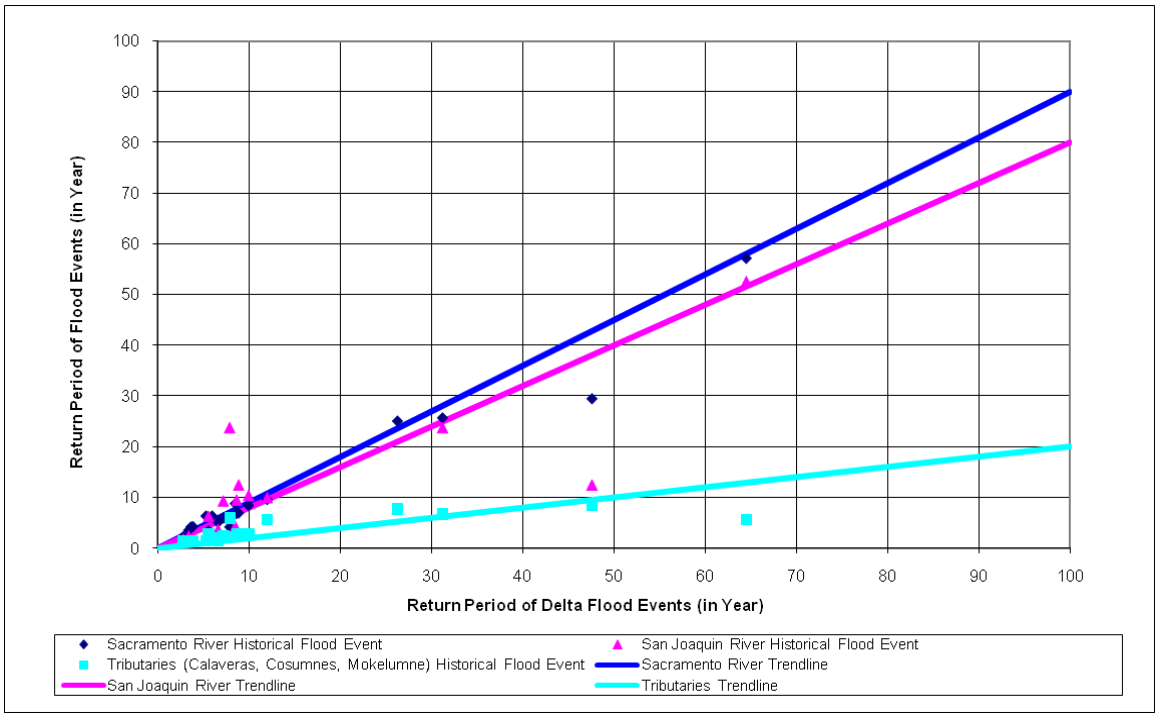
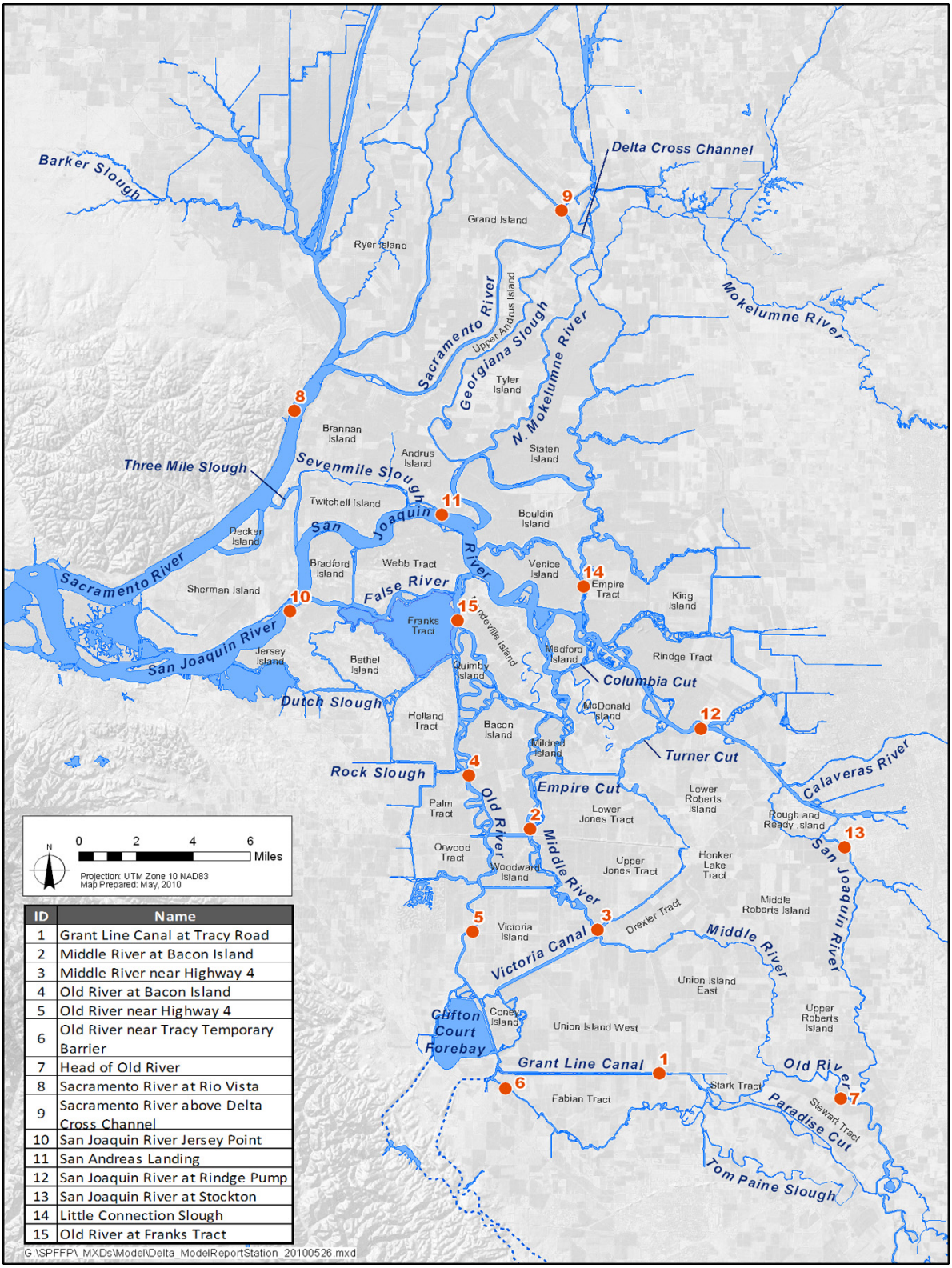


Figure 3-5. Correlation of Total Delta Inflow Recurrence with Source Inflows

As shown in Figure 3-8, for inflows into the RMA Delta Model from UNET results, the peak flow rate of the Sacramento River inflow (downstream from the American River confluence) to the RMA Delta Model for the 2 percent AEP event has a very similar magnitude to the 1 percent AEP event (Figure 3-8). For the Yolo Bypass inflow to the RMA Delta Model (Yolo Bypass at Lisbon), the difference between the flow rates of the 1 percent and 2 percent AEP events is less than 10 percent. For the San Joaquin River inflow to the RMA Delta Model (San Joaquin River upstream from the Paradise Cut Weir), the peak flow of the 1 percent AEP event is about 30 percent higher than the 2 percent AEP event.



ID	Name
1	Grant Line Canal at Tracy Road
2	Middle River at Bacon Island
3	Middle River near Highway 4
4	Old River at Bacon Island
5	Old River near Highway 4
6	Old River near Tracy Temporary Barrier
7	Head of Old River
8	Sacramento River at Rio Vista
9	Sacramento River above Delta Cross Channel
10	San Joaquin River Jersey Point
11	San Andreas Landing
12	San Joaquin River at Rindge Pump
13	San Joaquin River at Stockton
14	Little Connection Slough
15	Old River at Franks Tract

Figure 3-6. RMA Delta Model Output Locations

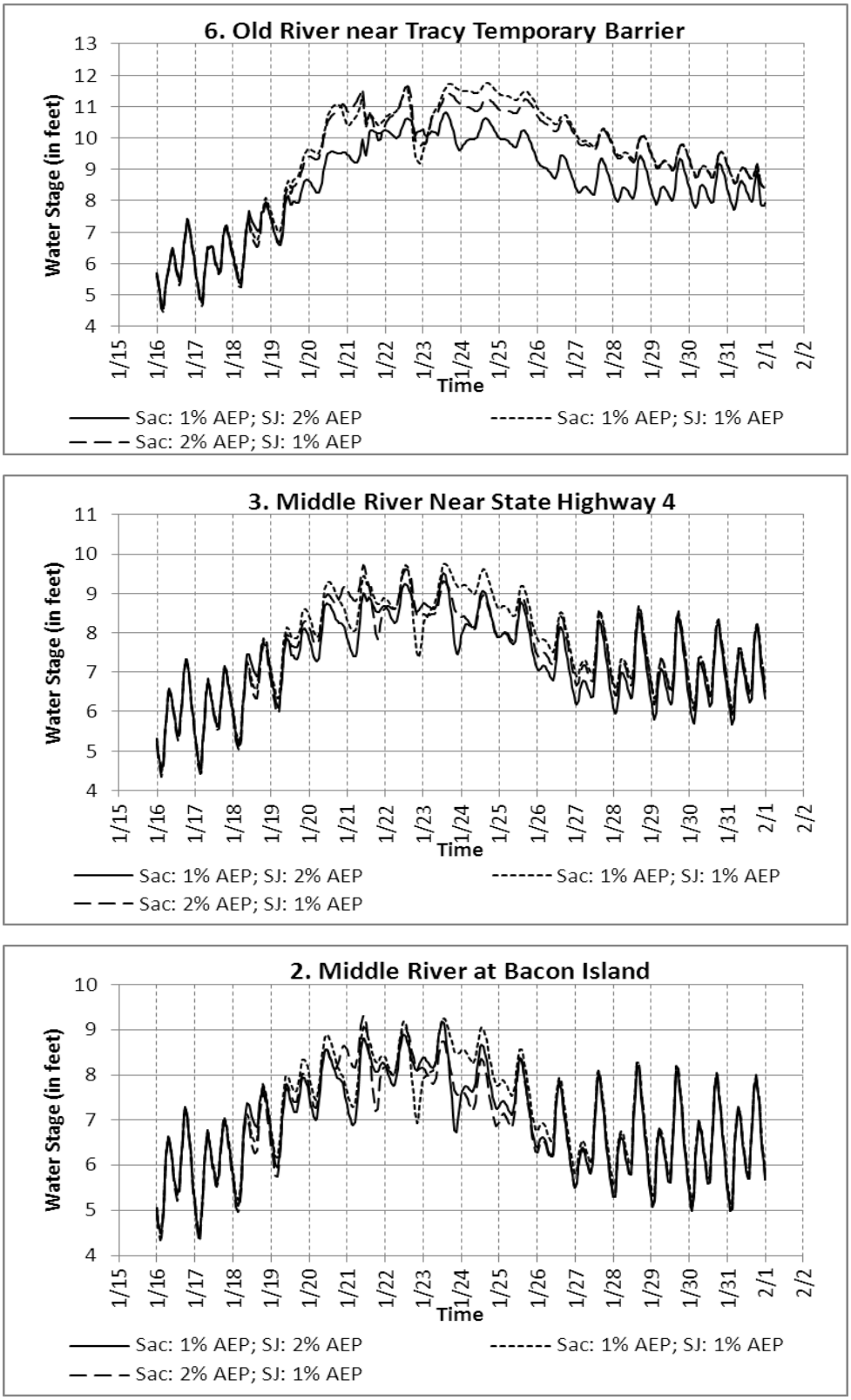
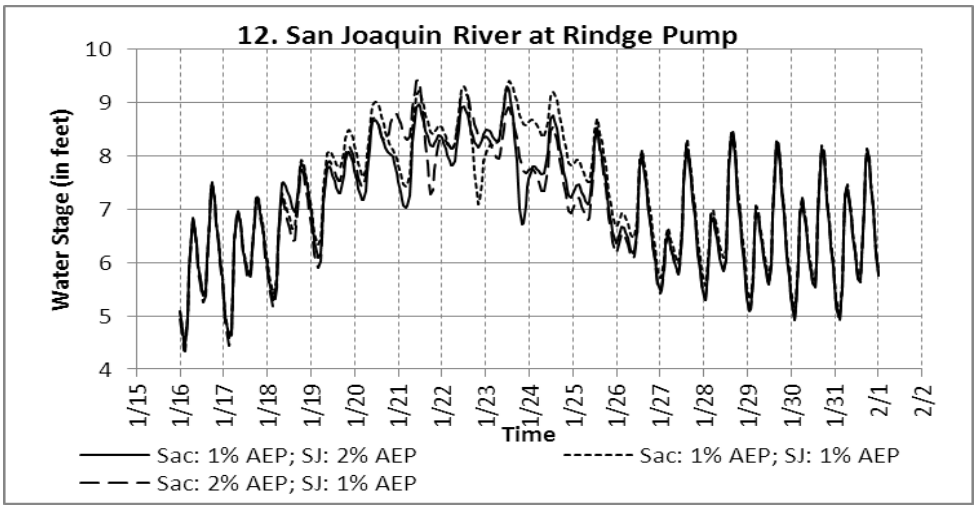
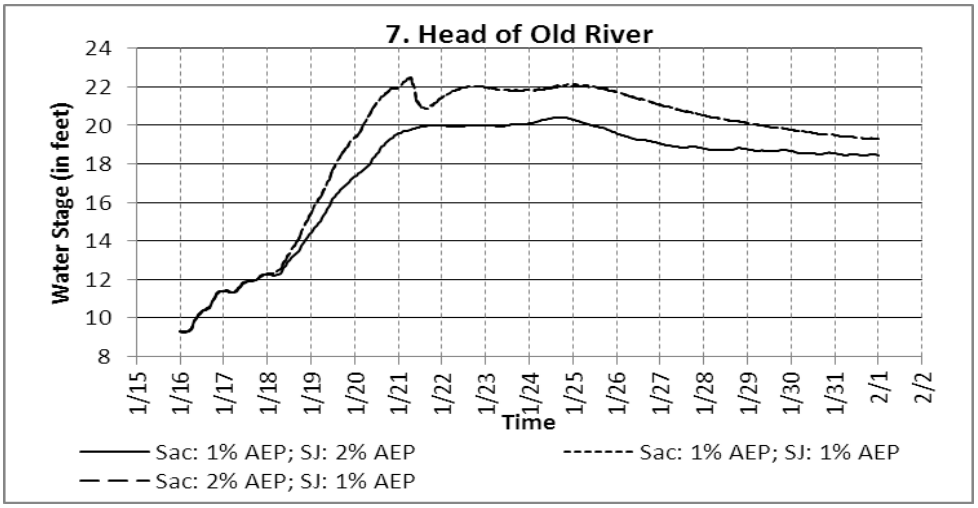


Figure 3-7. Simulated Delta Stages for Hydraulic Sensitivity Analysis





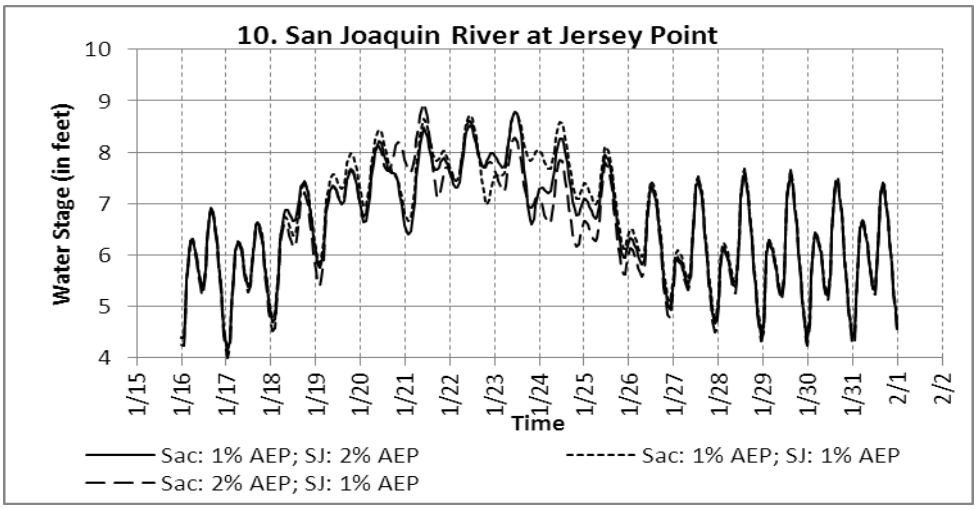
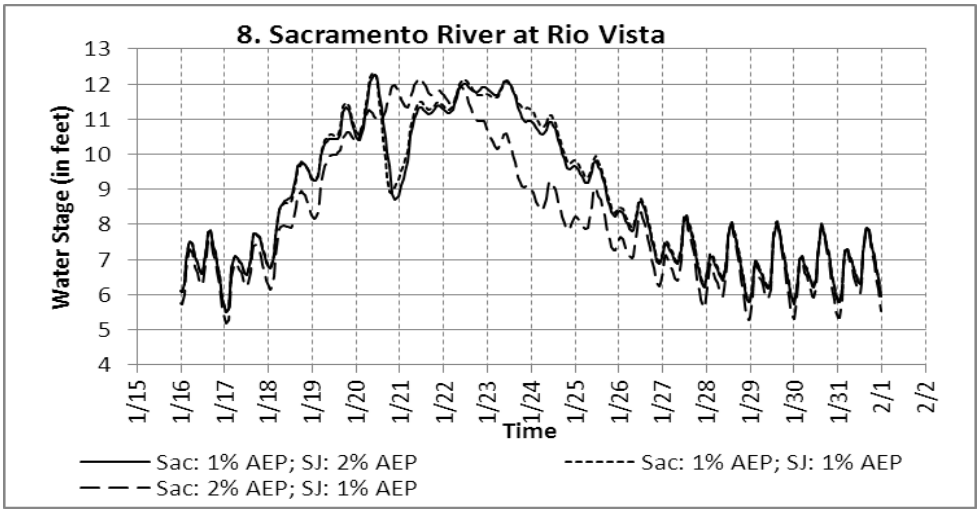


Figure 3-7. Simulated Delta Stages for Hydraulic Sensitivity Analysis (contd.)



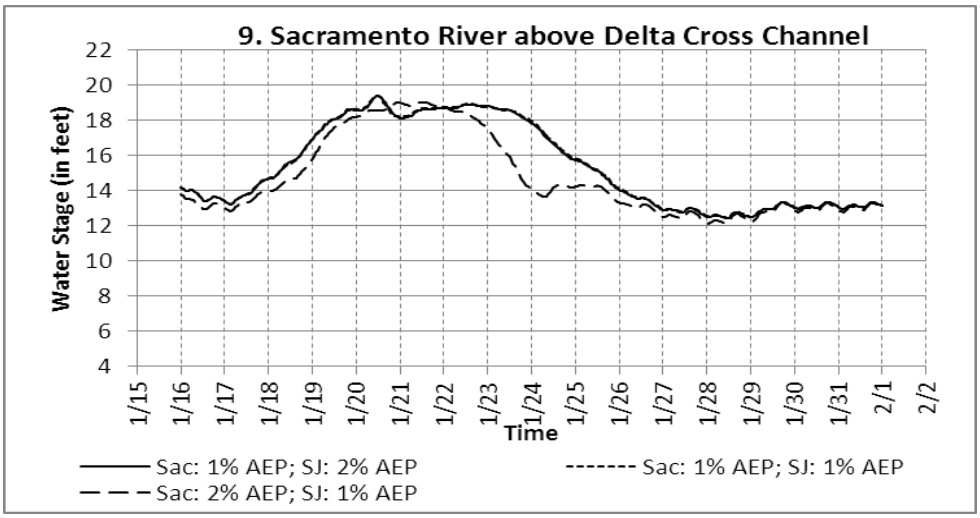


Figure 3-7. Simulated Delta Stages for Hydraulic Sensitivity Analysis (contd.)

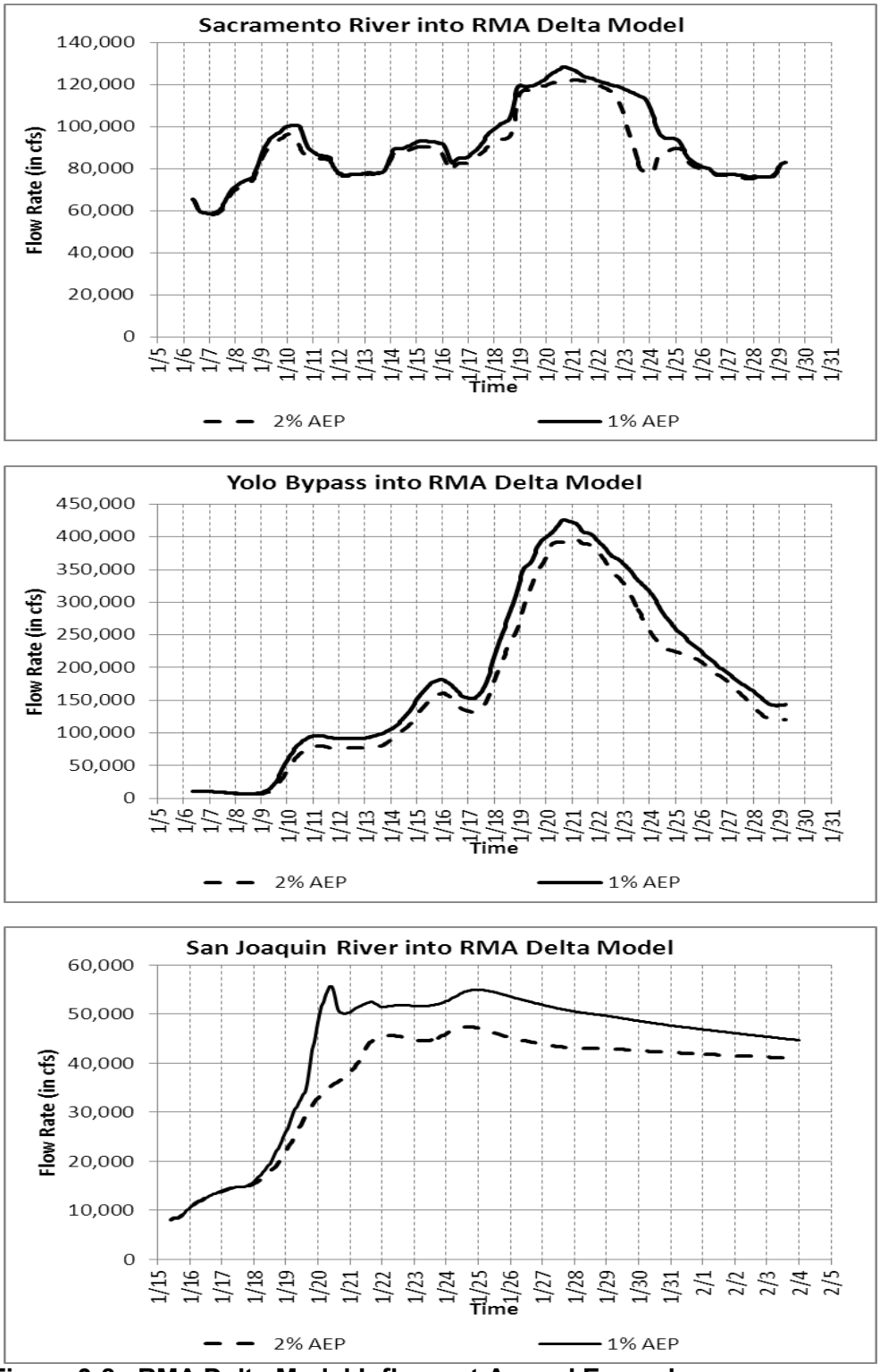


Figure 3-8. RMA Delta Model Inflows at Annual Exceedence Probability of 1 Percent and 2 Percent

The sensitivity analysis yields the following observations:

- Stages at locations in the Delta that are closer to the Sacramento River or Yolo Bypass (e.g., Rio Vista) demonstrated high sensitivity to Sacramento River inflows and very low sensitivity to San Joaquin River inflows.

- Stages at locations in the Delta that are closer to the San Joaquin River (e.g., head of Old River) demonstrated higher sensitivity to San Joaquin River inflows and very low sensitivity to Sacramento River inflows.

- Stages at locations in the Delta closer to Martinez (e.g., Jersey Point), increased with rising inflows during the peak inflow period (between January 19 through 23). However, the peak stage varied within 1 foot under different inflows from the two river basins. Stages at these locations also demonstrated very high sensitivity to tidal stages.

After looking at the results of the historical review and the sensitivity analysis, it was determined that Delta stage analysis would be based on inflows from the two river basins, as well as tributary flows for the same AEP.

3.4.4 Simulation Period

The Sacramento River UNET model simulation period was from January 6 through 29, with peak flows for all flood events occurring between January 18 and 20. The San Joaquin River UNET model simulation period was from January 15 through February 3, with peak flows for all flood events occurring between January 18 and 20.

For the RMA Delta Model, the simulation period for the 1, 0.5, and 0.2 percent AEP events was from January 7 through January 31. The simulation period for the remaining AEP events was from January 7 through February 3 so that river peak stages had passed through the Delta by the end of the simulation period.

Because the Delta simulation period extended beyond the simulation period for the Sacramento River UNET model, Sacramento River and Yolo Bypass inflows to the Delta were extended by repeating the last flow rates of the period beyond the UNET simulation period (i.e., the flow rate for January 29 was repeated for the period of January 30 through January 31 for 1, 0.5, and 0.2 percent AEP events, and through February 3 for the remaining events). Similarly, inflows to the Delta from the San Joaquin River UNET model were extended by repeating the very first flow rate for the period before the UNET simulation period.

3.4.5 Levee Breach Location and Elevation

In the RMA Delta Model, by default, a boundary or shoreline is represented as an "infinite wall" where no overtopping flow is allowed. To simulate levee overtopping, levee failures, and resulting island flooding, network elements representing river channels were connected to Delta island elements with "weir elements." Levee failure was modeled by changing the weir elevation over time on a node-by-node basis to allow complete simulation of a levee failure event. Flow over a levee can transition from free weir flow to submerged weir flow and finally to simple friction loss using a Manning's "n" formula. The RMA model now allows levees to overtop without failure or permits the initiation of levee failure when a threshold water surface elevation is reached.

It is assumed that when river stage is higher than the levee crest of an island, the levee will begin to breach and water will flow into the island until water stage inside and outside the island is in equilibrium. For each Delta island, levee crest elevations were taken approximately every 1,500 feet along the levee from DWR 2008 Light Detection and Ranging (LiDAR) data (URS, 2011). The breach location for each island was selected through the following steps:

- **Step 1** – Use the RMA Delta Model with "infinite walls" " (i.e., no levee overtopping or breaches) to simulate maximum river stage under the 0.5 percent AEP event of the Achieve SPFC Design Flow Capacity Approach.[3] From the Riverine Studies, the SPFC approach resulted in the largest stage increases into the Delta.

- **Step 2** – Calculate overtopping as the difference between the peak river stage and levee crest elevation.

- **Step 3** – Use the location of the maximum overtopping difference from Step 2 as the levee breach location.

The UNET and RMA Delta models overlap at their downstream and upstream ends, respectively. For islands that were simulated in both the RMA Delta Model and the UNET models (see Figure 3-4), levee breach simulation in the RMA Delta Model was based on the same assumptions for levee breach location and elevation as in the UNET models.

[3] Boundary inflow for this event represents the most conservative river flow conditions—that levees upstream do not fail until river stage exceeds the SPFC design flow capacity.

3.5 Assumptions for Achieve SPFC Design Flow Capacity Approach

Changes in modeling assumptions from the No Project condition in the RMA Delta Model for the Achieve SPFC Design Flow Capacity Approach included changing levee breach elevations for SPFC levees to match the SPFC design profile plus three feet of freeboard and using different upstream boundary condition inflows from the Achieve SPFC Design Flow Capacity UNET models of the Sacramento and San Joaquin rivers.

3.6 Assumptions for Protect High Risk Communities Approach

Changes in modeling assumptions from the No Project condition in the RMA Delta Model for the Protect High Risk Communities Approach included changing levee breach elevations for any high risk community levees and using different upstream boundary condition inflows from the Protect High Risk Communities UNET models of the Sacramento and San Joaquin rivers.

3.7 Assumptions for Enhance Flood System Capacity Approach

Changes in modeling assumptions from the No Project condition in the RMA Delta Model for the Enhance Flood System Capacity Approach included Paradise Cut Bypass modifications, transitory storage on Roberts Island, and different upstream boundary condition inflows from the Enhance Flood System Capacity UNET models of the Sacramento and San Joaquin rivers.

3.7.1 Paradise Cut Bypass Modifications

The following improvements to Paradise Cut to increase its capacity to divert water during the high-flow conditions were made in the RMA Delta Model as part of the Enhance Flood System Capacity Approach (see Figure 3-9):

- Removal of about 4 feet of soil from an existing elevated terrace in the reach of Paradise Cut downstream from the weir to the upstream side of the Union Pacific Railroad (UPRR) crossing to lower the tailwater for the Paradise Cut Weir, allowing more flow to be diverted from the San Joaquin River over the weir.

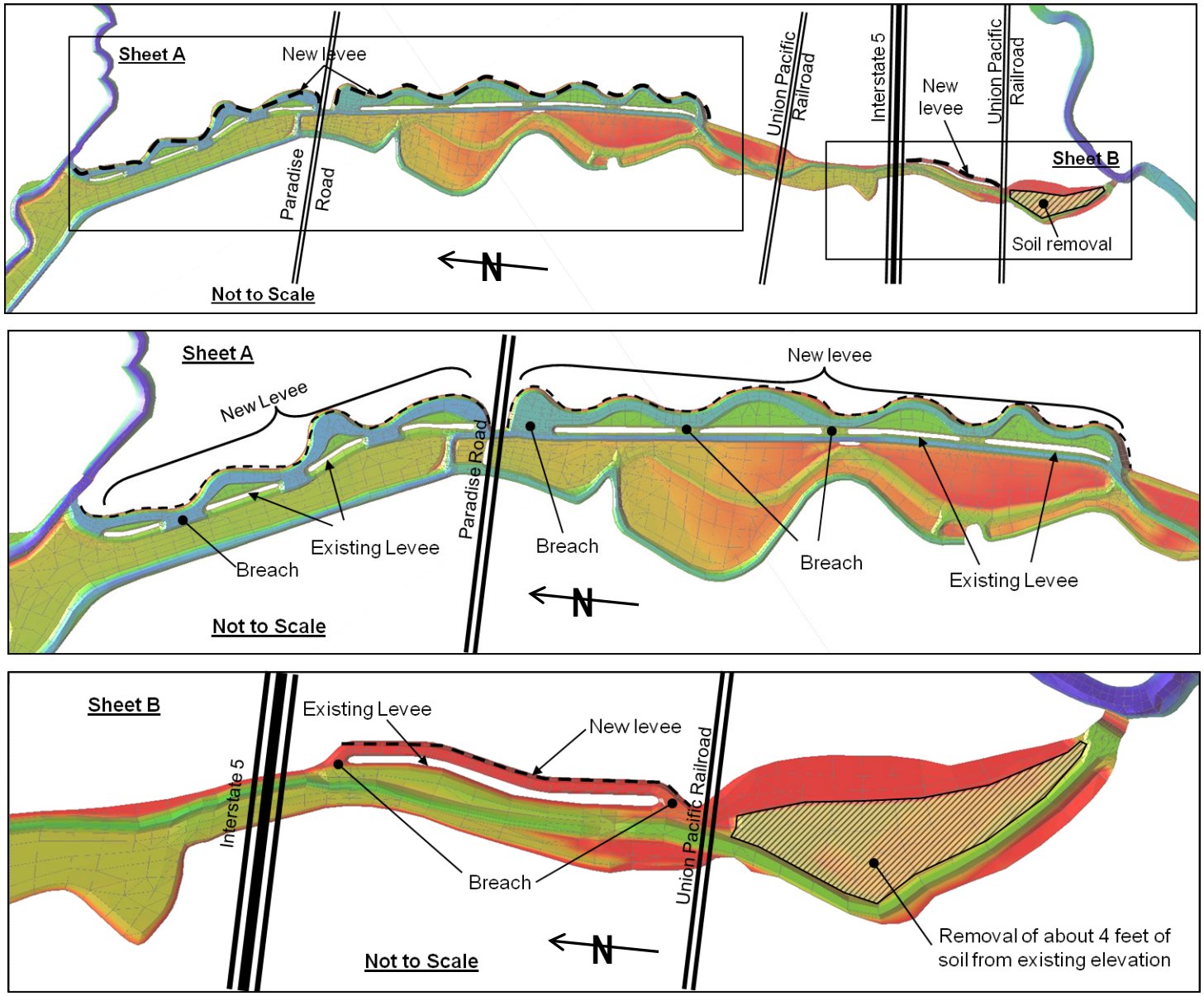


Figure 3-9. Paradise Cut Bypass Configuration

- Construction of a new levee set back about 150 feet from the existing levee on the right bank of Paradise Cut between the UPRR crossing and Interstate 5, with breaches in the existing levee to increase the carrying capacity of Paradise Cut without a corresponding stage increase. The rest of the existing levee would remain as an in-channel island for ecological restoration area.

- Construction of a new levee set back 150 to 900 feet from the existing levee on the right bank of Paradise Cut from downstream of the western Union Pacific Railroad crossing to the Paradise Road crossing. The existing levee would remain but would be breached in several locations, and the area between the existing and new levees would be excavated down to an elevation of 3.0 feet below mean sea level (msl) to form a marsh area for ecological restoration.

- Construction of a new setback levee with levee breaches in the existing levee between Paradise Road and the confluence of Paradise Cut with Old River similar to the section just upstream, except that the area between the existing and new levees would be excavated to an elevation of 5.0 feet below msl to form a marsh area for ecological restoration.

3.7.2 Roberts Island Transitory Storage

Roberts Island transitory storage is to provide about 69,000 acre-feet of storage on 8,800 acres on Upper and Middle Roberts Island for the 1 percent AEP and larger flood events. Floodflows would enter the Roberts Island transitory storage area over a new weir in the levee on the left bank of the San Joaquin River and would be stored until the river subsides to a stage that no longer threatens the metropolitan Stockton area. Stored water would be released back to the San Joaquin River through a new outlet. The following are improvements or additions (see Figure 3-10) for this new transitory storage area:

- Levee repairs along the left bank of San Joaquin River and the right banks of the Middle and Old rivers.

- Construction of a 3,000-foot-long concrete weir (crest height 16.28 feet NGVD29) on the left bank of the San Joaquin River about 2.25 miles downstream from the confluence of the San Joaquin River and Old River.

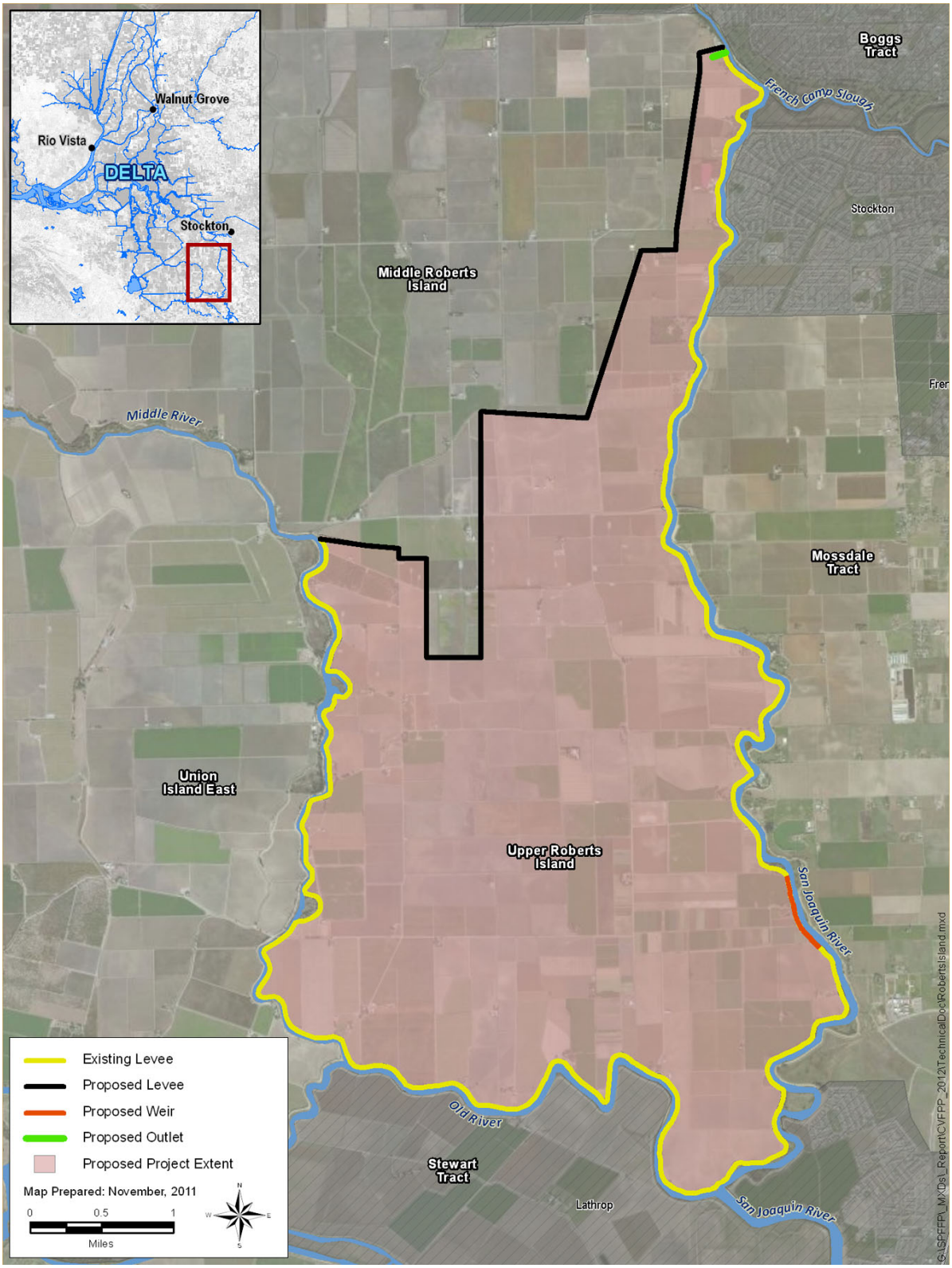


Figure 3-10. Roberts Island Transitory Storage

- Construction of a new 7.4-mile-long levee (crest height about 16.5 feet NGVD29) through the center of Roberts Island to connect the left bank of the San Joaquin River with the right bank of Old River and to separate Middle Roberts Island from Lower Roberts Island. The new levee is necessary because the land surface elevation of Lower Roberts Island is below sea level, and removing any stored water from Lower Robert Island would require pumping, instead of gravity drainage, as is the case with Upper and Middle Robert Island.

- Construction of a gated outlet structure at the northeast corner of Middle Roberts Island, just south of State Highway 4 to accommodate a maximum 2,500 cfs return flow to the San Joaquin River at various stages.

3.8 Assumptions for State Systemwide Investment Approach

Changes in modeling assumptions from the No Project condition for the RMA Delta Model for the State Systemwide Investment Approach included changing levee breach elevations for any high risk community levees, Paradise Cut Bypass modifications (see Section 3.7.1), and using different upstream boundary condition inflows from the State Systemwide Investment Approach UNET models of the Sacramento and San Joaquin rivers.

3.9 Model Limitations

Understanding and applying the results of any model requires an understanding of the limitations of the model. Limitations associated with the RMA Delta Model are as follows:

- Levee breach locations and elevations were predetermined. Once river stage at a predetermined location exceeded the designated elevation, levee overtopping or failure and subsequent island flooding were simulated using 2-D weir elements.

- The RMA Delta Model for the CVFPP should not be used to predict actual levee failures because model inputs are deterministic (i.e., no randomness is involved in the model results, but actual levee failures are a matter of probability).

- River channels were represented by 1-D or 2-D elements to approximate reality and might potentially simplify the representation of the channel at certain locations.

- The RMA Delta Model does not represent flow hydraulics through bridges with the same level of detail as the HEC-RAS program.

- The RMA Delta Model is intended to be used to simulate Delta in-channel water stage and flood depth and flood volume of Delta islands. The water quality module of the RMA Delta Model was not used for the 2012 CVFPP modeling and, thus, flood-associated salinity and particle transport were not evaluated.

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4.0 Results

Figure 4-1 indicates the locations in the Delta at which stage-frequency curves will be plotted to allow comparison of the operations of the flood management system among the No Project condition and the various approaches.

4.1 Stage-Frequency Curves

Figures 4-2 through 4-16 show stage-frequency curves for all of the approaches for the 15 selected output locations in the Delta (Figure 4-1).

Abbreviations are used on the stage-frequency plots to designate the the approaches, as follows:

- SPFC = Achieve SPFC Design Flow Capacity Approach
- PHRC = Protect High Risk Communities Approach
- EFSC = Enhance Flood System Capacity Approach
- SSIA = State Systemwide Investment Approach

4.2 Out-of-System Volumes

Figure 4-1 shows the names and locations of the Delta islands used to tabulate the volume of floodflows leaving the flood management system during a given flood. Tables 4-1 through 4-5 contain the out-of-system volume for the No Project condition and each of the approaches in each of the islands. These out-of-system volumes are instrumental in understanding the function of the flood management system in the Delta. For example, the stage at a given location may be lower for the 100-year flood than for the 50-year flood. If islands upstream from or in the vicinity of this location are reviewed and a significant increase is observed in out-of-system volume between the 50- and the 100-year floods, it can be concluded that a levee breach upstream from the location has reduced the flows to a level less than the 50-year flow.

Another example would be a location where the stage between No Project condition and one of the approaches increases significantly for the same

AEP flood. Again, if upstream out-of-system volume is reduced, it can be concluded that additional flow remains in the river because upstream levees may have been reconstructed or raised and no longer breach as they did in the No Project condition.

4.3 Findings

4.3.1 Achieve SPFC Design Flow Capacity Approach

Restoring all SPFC levees to their design flow capacity for the Achieve SPFC Design Flow Capacity Approach would significantly reduce the number of levee breaks upstream from the Delta and would cause increased stages at all reporting locations in the Delta. The floodwaters that normally would leave the system through levee breaches in the No Project condition would be contained in the river channels and barring other levee breaches would continue downstream to the Delta in this approach. Island inundation from levee breaches would be greater than the No Project condition for AEPs as low as 1 percent. Island inundation would actually decrease for AEPs of 0.5 and 0.2 percent because of increased levee breaks in the downstream areas of the Sacramento and San Joaquin river basins.

4.3.2 Protect High Risk Communities Approach

The Protect High Risk Communities Approach modifies urban levees to pass the 200-year (0.5 percent AEP) flood with 3 feet of freeboard. Since it is only urban levees, and a few small communities, that are modified, stages in the Delta would remain essentially the same as for the No Project condition. Island inundation follows the same pattern and is much the same as the No Project condition except for the 0.5 and 0.2 percent AEP floods where urban areas that sustained a levee breaks in the No Project condition do not break, causing increased flows downstream, which would increase stages and result in increased island inundation in the Delta.

4.3.3 Enhance Flood System Capacity Approach

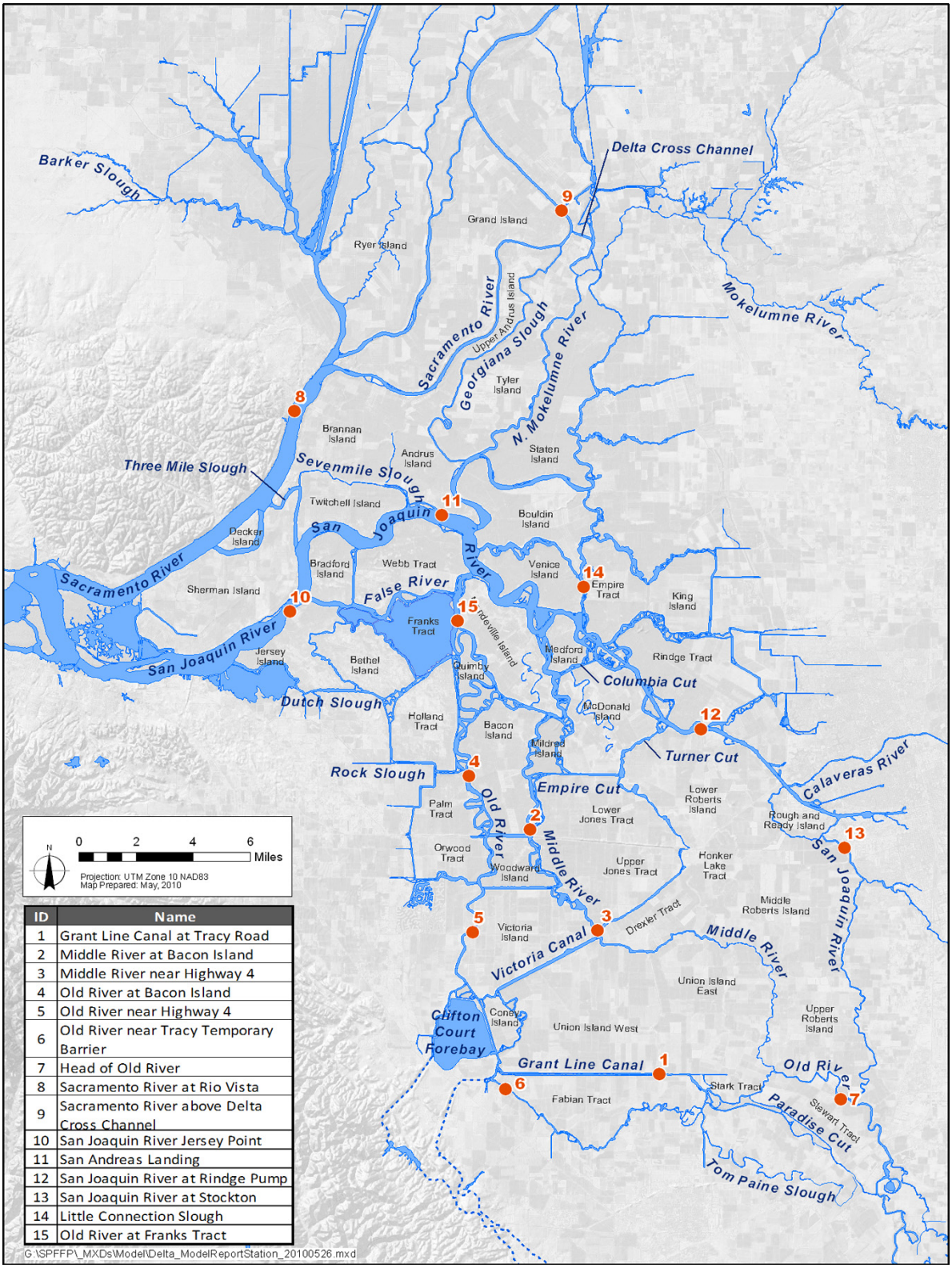
The Enhance Flood System Capacity Approach modifies urban levees to pass the 200-year (0.5 percent AEP) flood with 3 feet of freeboard. In addition, nonurban SPFC levees, including SPFC levees in the Delta, were modified to the 55/57 design profile (the design standard for construction of the State Plan of Flood Control) plus freeboard (3 feet), or the existing TOL elevation as determined by the ULE and NULE projects, whichever was greater. Other key components of the approach are added upstream reservoir storage, widened and new bypasses, levee setbacks, and floodplain storage.

Even though restoring all urban and SPFC levees as described above should result in additional flow volumes entering the Delta, flow volumes entering the Delta are significantly decreased for the 10, 4, 2, and 1 percent (10-, 25, 50-, 100-year) floods as a result of the added upstream reservoir and floodplain storage. For the 0.5 and 0.2 percent AEP (200- and 500-year) floods the reservoir and floodplain storage is not enough to prevent an increase in flow into the Delta.

Flooding in the Delta is less than for the No Project condition for all AEPs because for two reasons. First, more than a dozen of the islands that flood in the No Project condition have SPFC levees and thus are restored to their design profile for the Enhance Flood System Capacity Approach. Second, for more frequent floods (i.e., 10, 4, 2, and 1 percent AEP), there is flow entering the Delta and therefore lower stages. The combination of these two factors results in less Delta flooding for the Enhance Flood System Capacity Approach.

4.3.4 State Systemwide Investment Approach

The State Systemwide Investment Approach assumes the same improvements to urban levees as the Protect High Risk Communities Approach. In addition, a new bypass and widening of the Yolo and Sutter bypasses are included in the Sacramento River Basin, and Paradise Cut Bypass is widened in the San Joaquin River Basin. Stages in the Delta are similar to or lower than the Protect High Risk Communities Approach, except where changes to the bypasses modify stages. Island inundation is less than the No Project condition for all AEPs except for the 0.2 percent AEP that sustains a less than 1 percent increase in island inundation.



ID	Name
1	Grant Line Canal at Tracy Road
2	Middle River at Bacon Island
3	Middle River near Highway 4
4	Old River at Bacon Island
5	Old River near Highway 4
6	Old River near Tracy Temporary Barrier
7	Head of Old River
8	Sacramento River at Rio Vista
9	Sacramento River above Delta Cross Channel
10	San Joaquin River Jersey Point
11	San Andreas Landing
12	San Joaquin River at Rindge Pump
13	San Joaquin River at Stockton
14	Little Connection Slough
15	Old River at Franks Tract

Figure 4-1. RMA Delta Model Output Locations

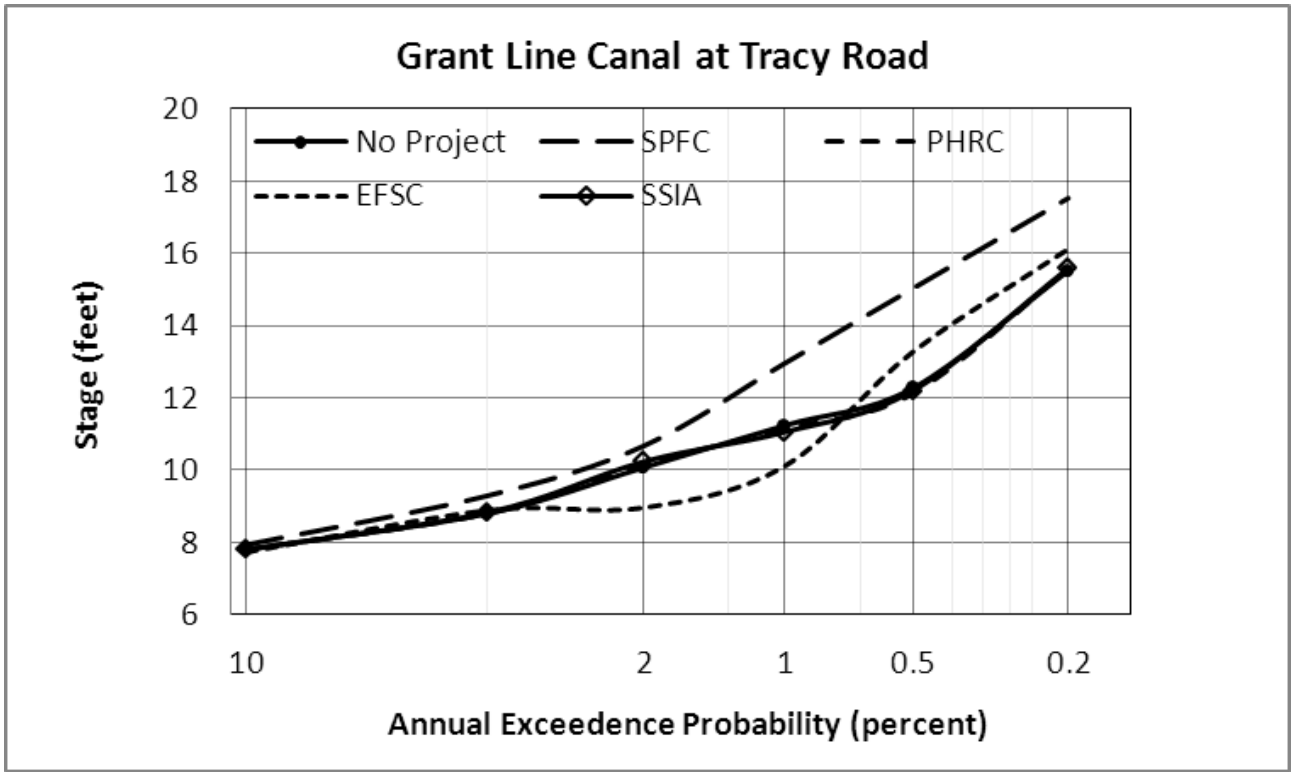


Figure 4-2. Stage-Frequency Curves: Grant Line Canal at Tracy Road [1]

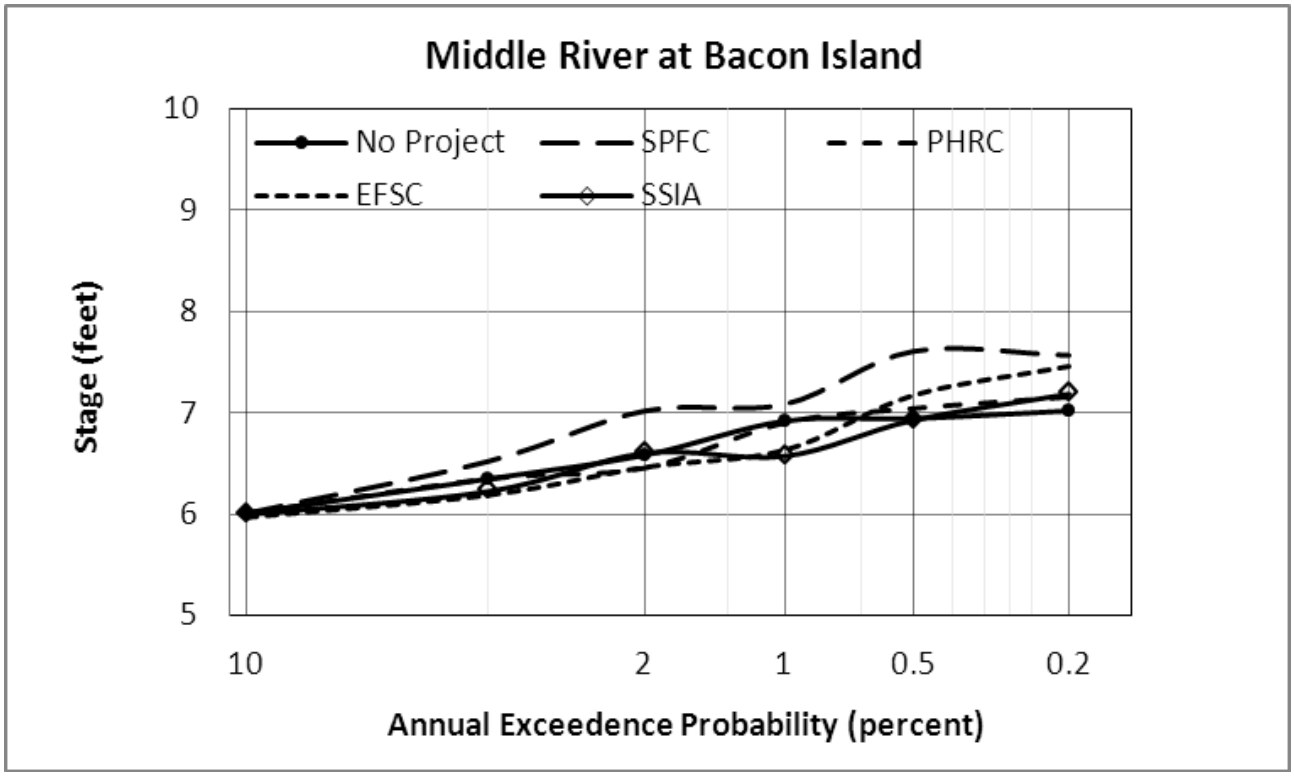


Figure 4-3. Stage-Frequency Curves: Middle River at Bacon Island [2]

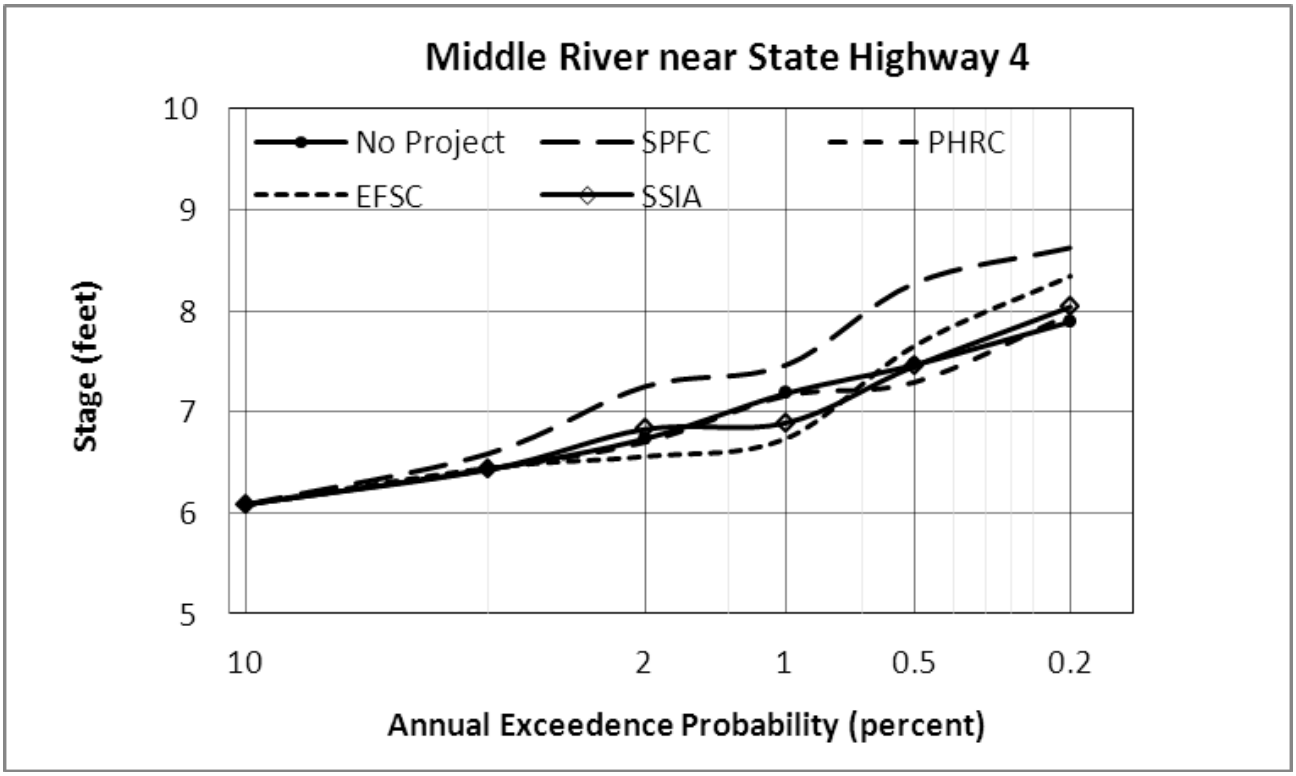


Figure 4-4. Stage-Frequency Curves: Middle River near State Highway 4 [3]

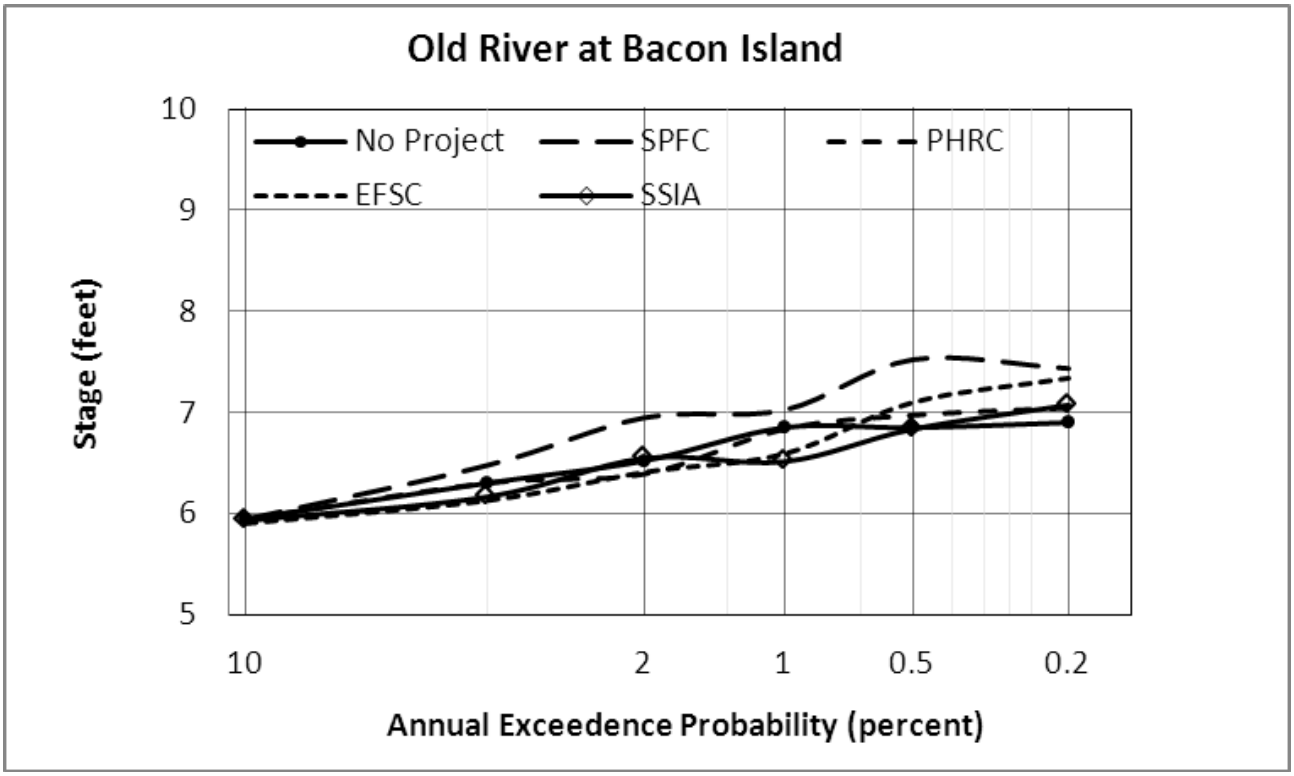


Figure 4-5. Stage-Frequency Curves: Old River at Bacon Island [4]

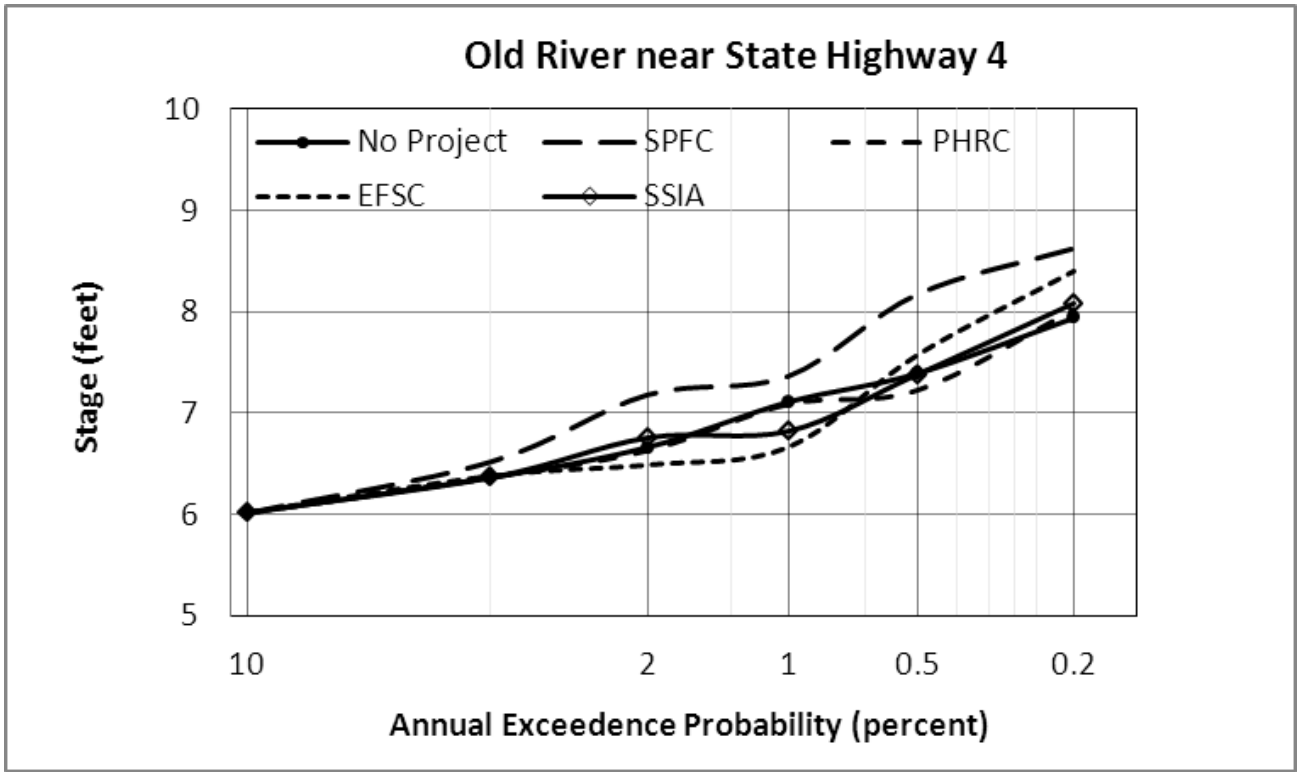


Figure 4-6. Stage-Frequency Curves: Old River near State Highway 4 [5]

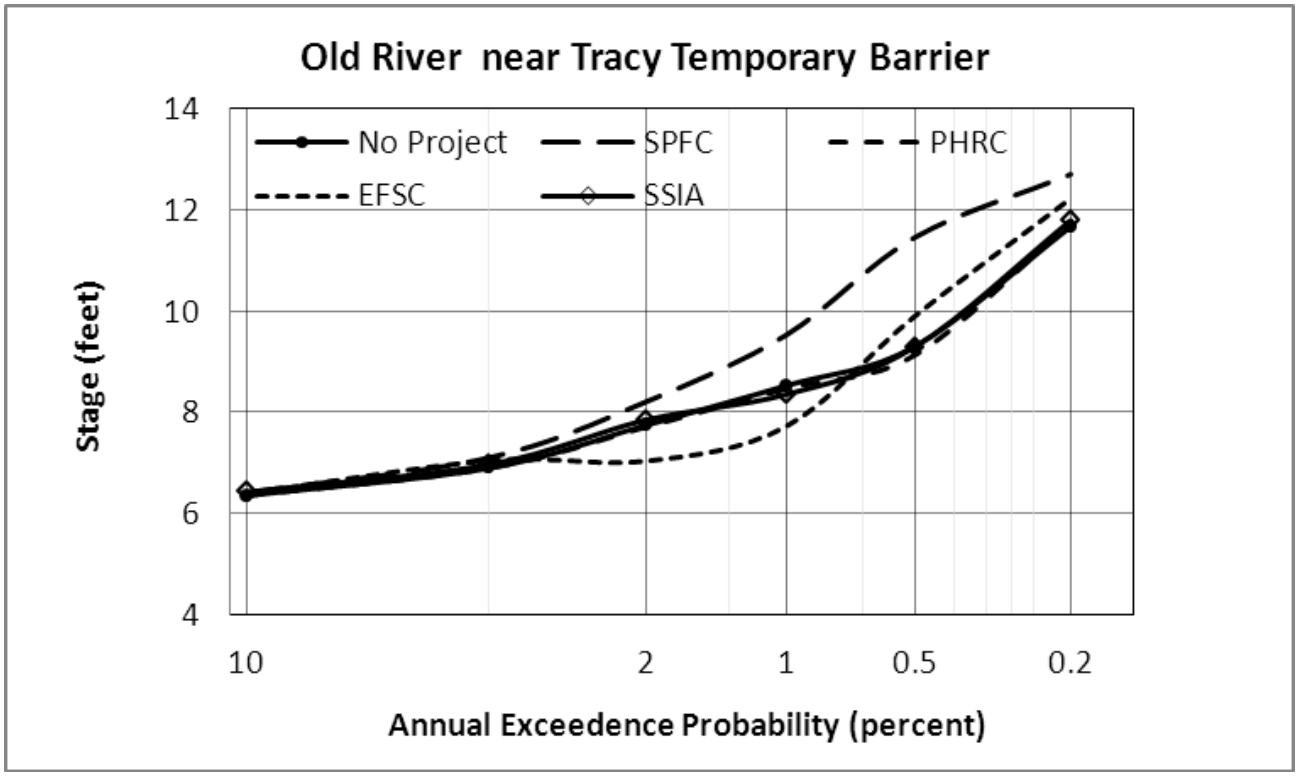


Figure 4-7. Stage-Frequency Curves: Old River near Tracy Temporary Barrier [6]

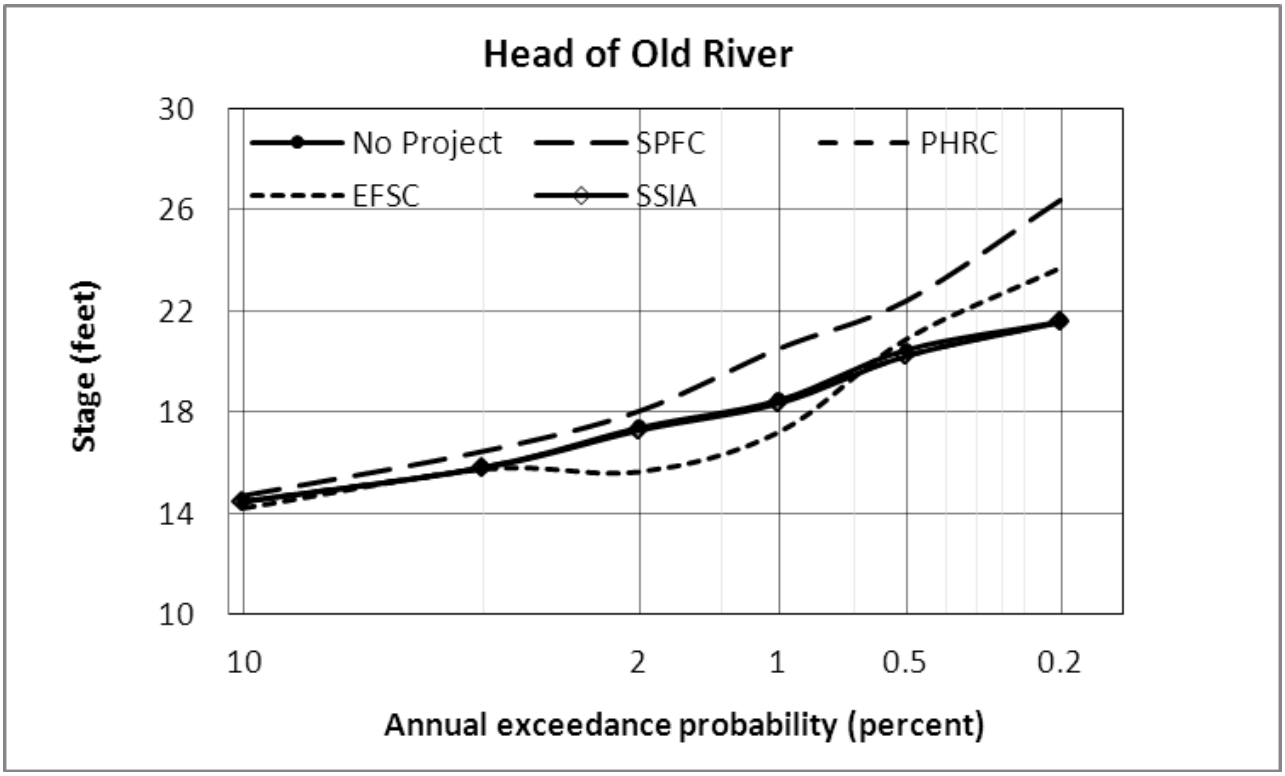


Figure 4-8. Stage-Frequency Curves: Head of Old River [7]

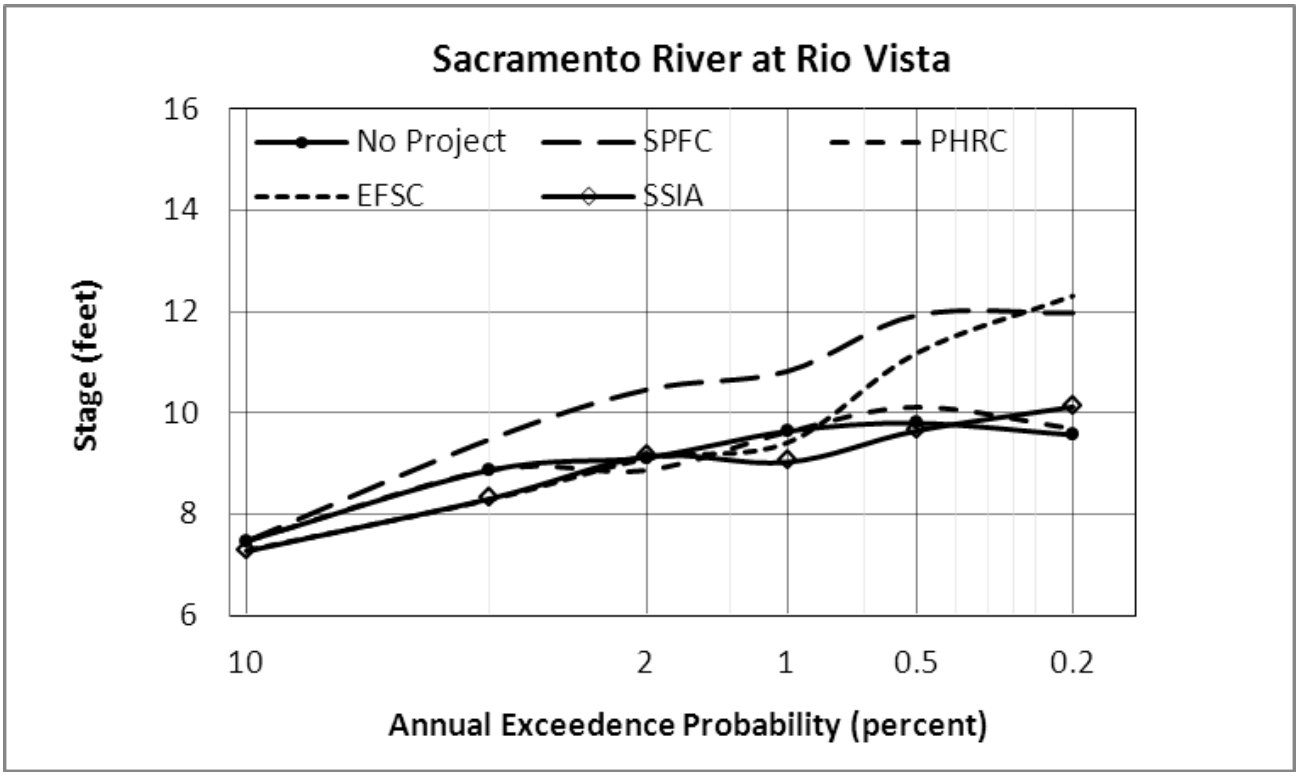


Figure 4-9. Stage-Frequency Curves: Sacramento River at Rio Vista [8]

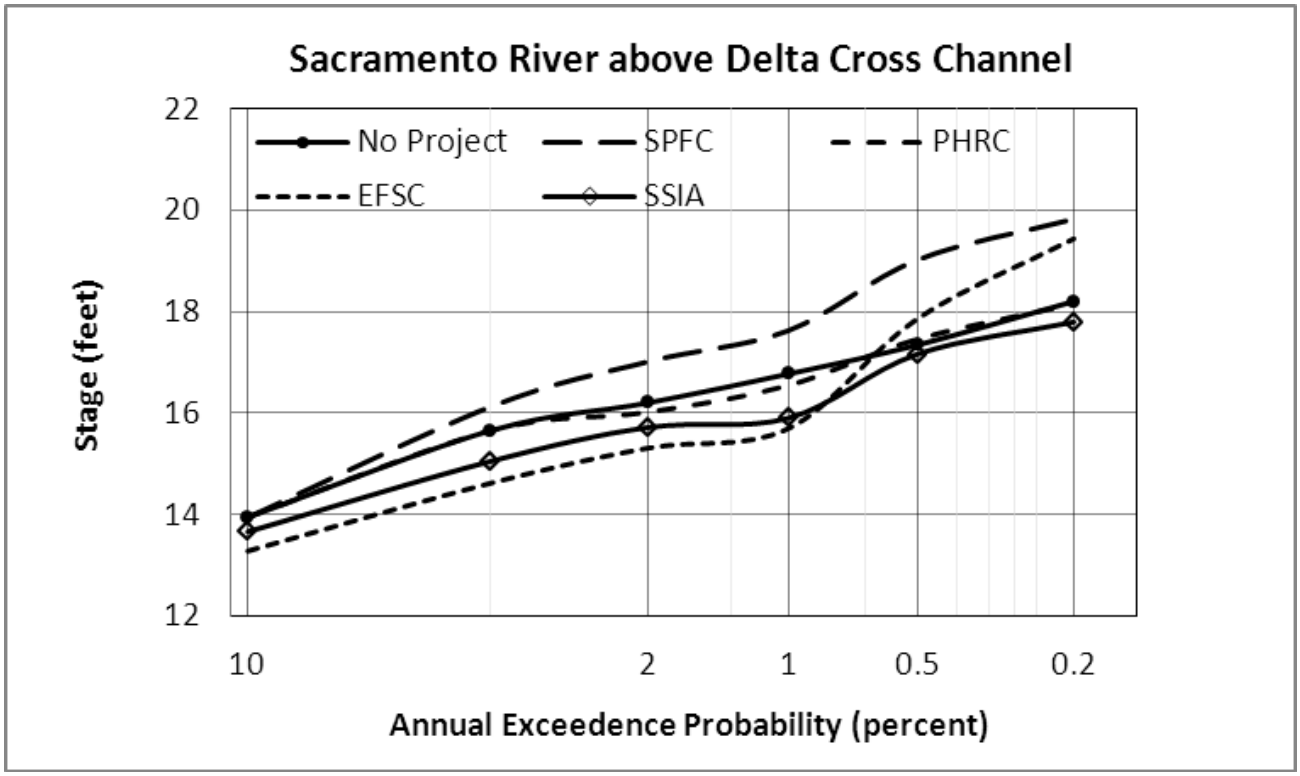


Figure 4-10. Stage-Frequency Curves: Sacramento River Above Delta Cross Channel [9]

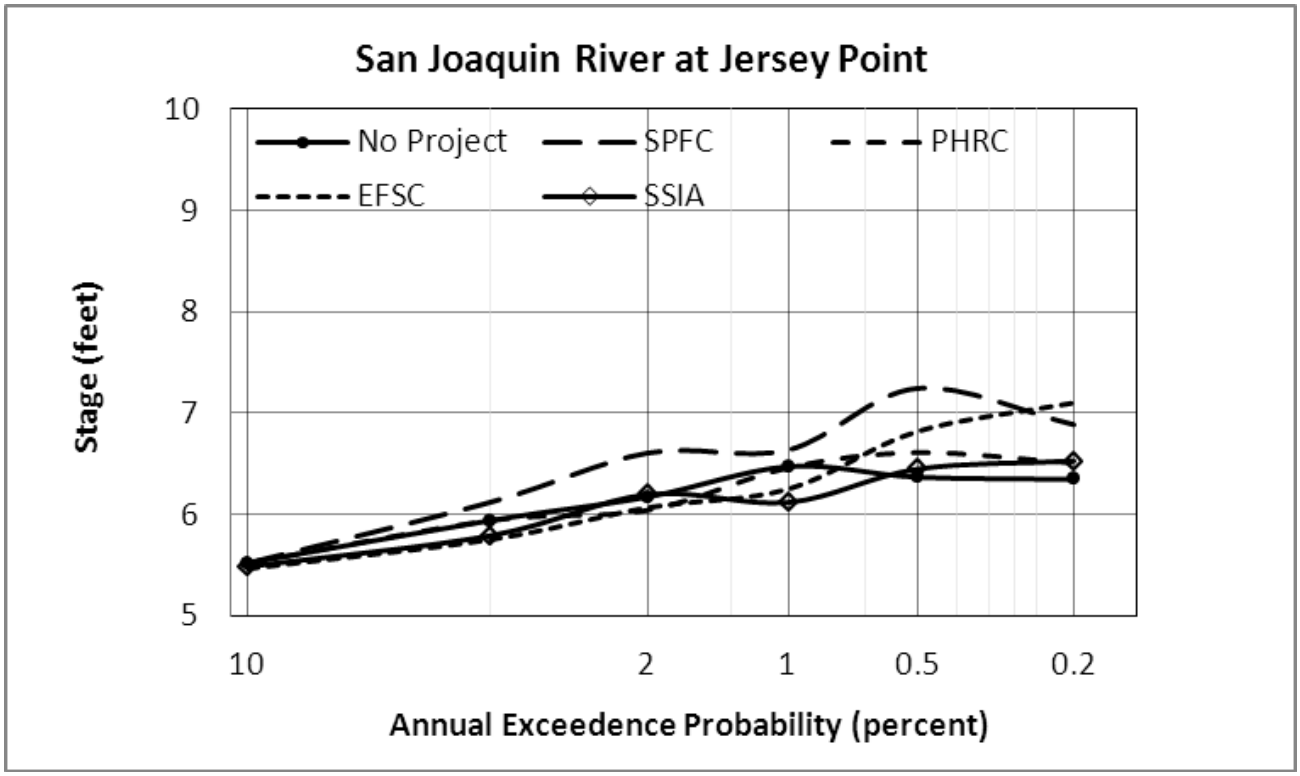


Figure 4-11. Stage-Frequency Curves: San Joaquin River at Jersey Point [10]

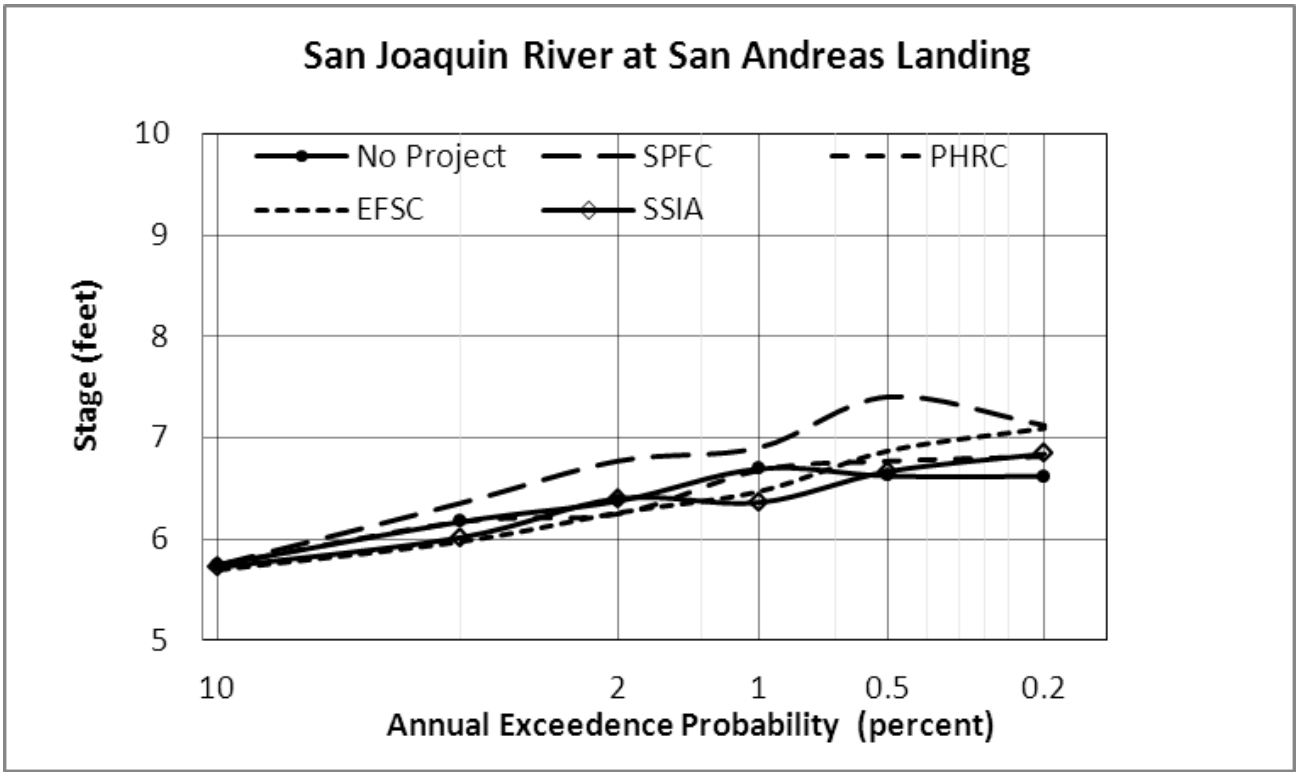


Figure 4-12. Stage-Frequency Curves: San Joaquin River at San Andreas Landing [11]

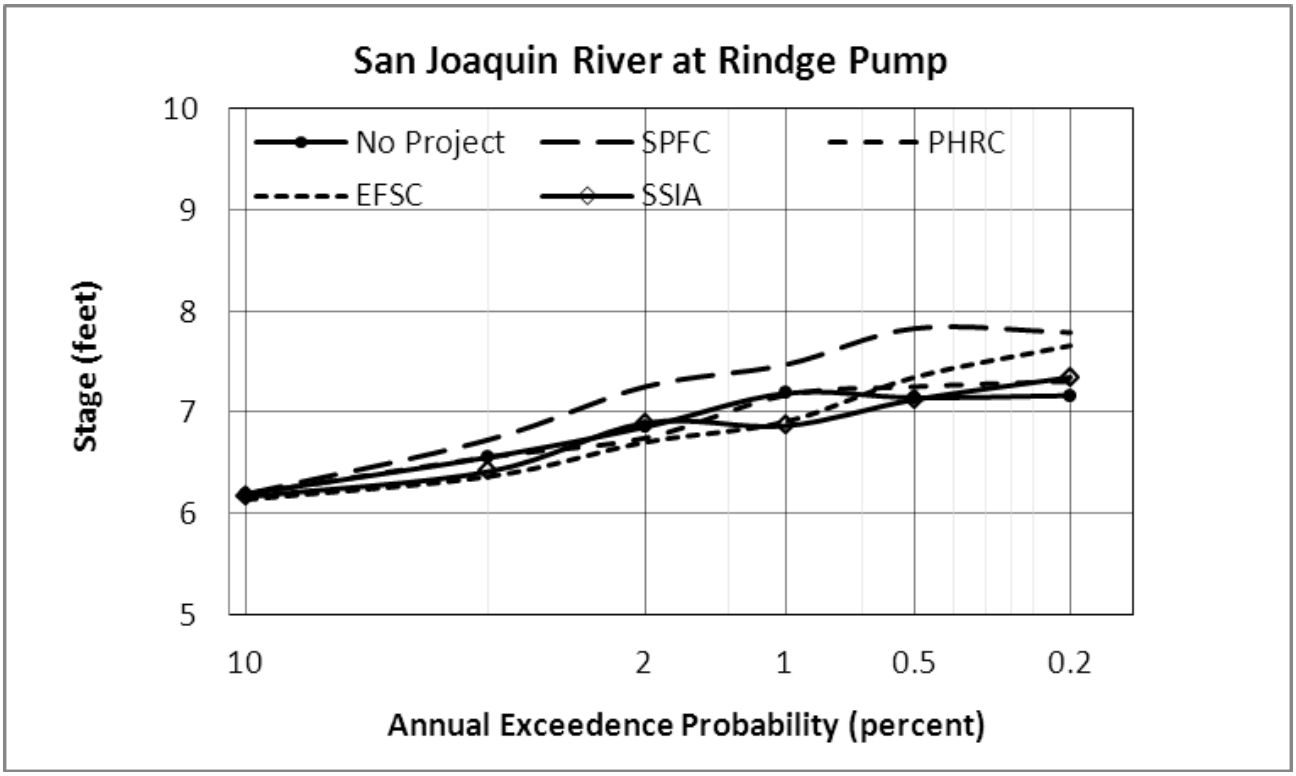


Figure 4-13. Stage-Frequency Curves: San Joaquin River at Rindge Pump [12]

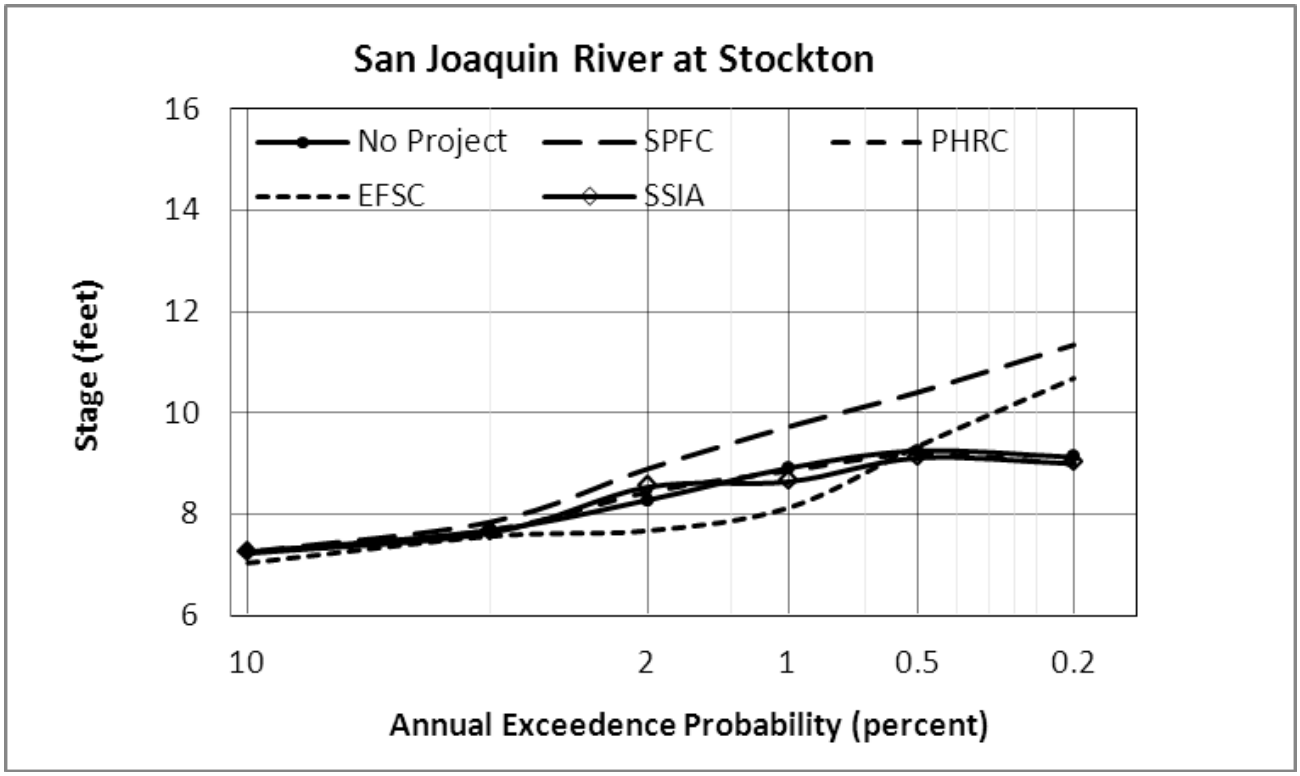


Figure 4-14. Stage-Frequency Curves: San Joaquin River at Stockton [13]

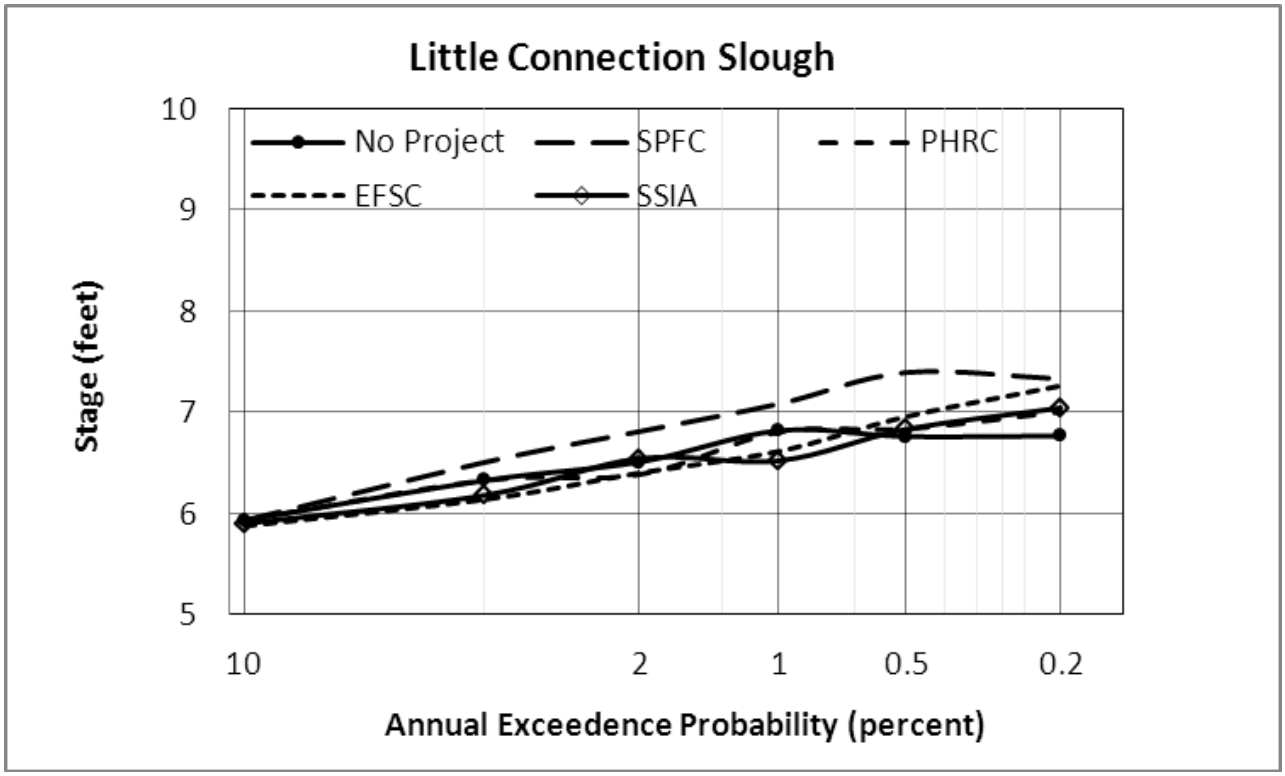


Figure 4-15. Stage-Frequency Curves: Little Connection Slough [14]

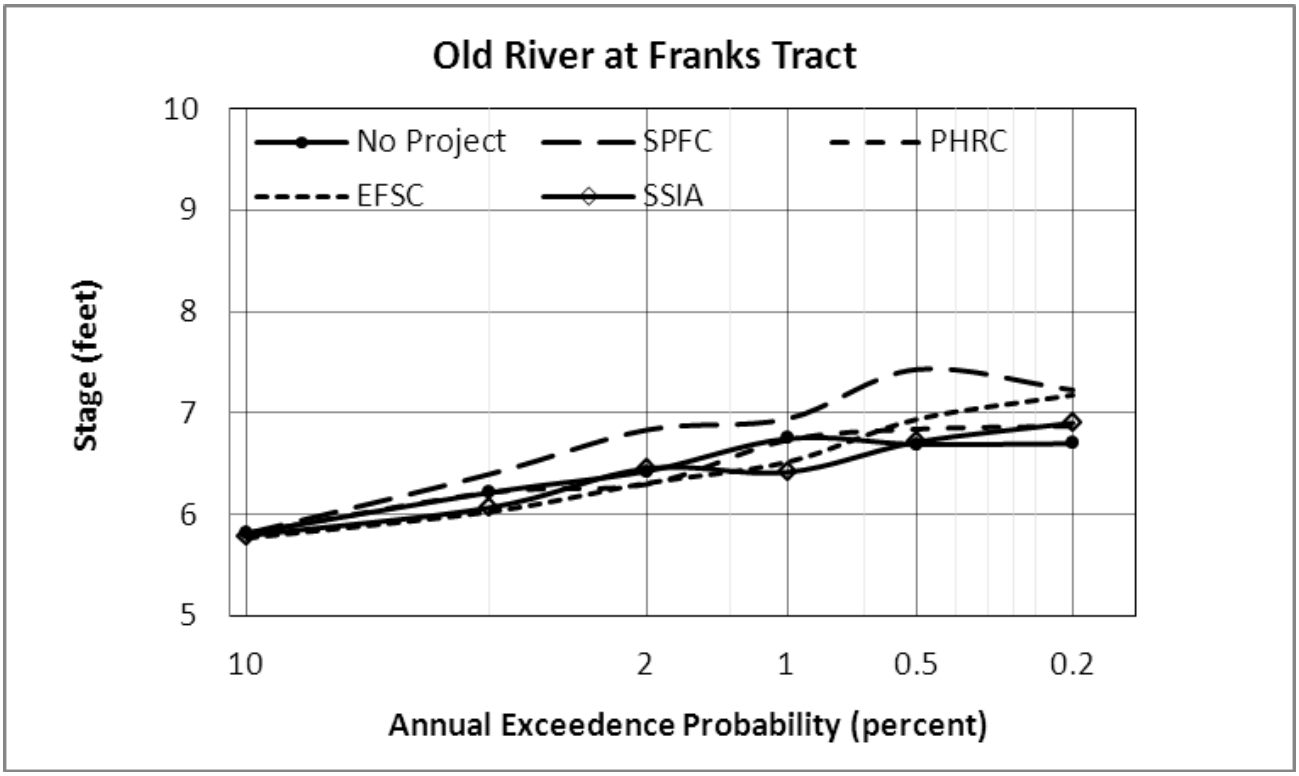


Figure 4-16. Stage-Frequency Curves: Old River at Franks Tract [15]

Table 4-1. Island Inundated (Out-of-System) Volume by Annual Exceedence Probability – No Project Condition

Island Name	Island Inundated Volume due to Flooding (TAF)					
	10% AEP	4% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Merritt Island					82	91
Pierson District (Courtland/RD551)						211
Sutter Island					38	39
Grand Island					310	307
Tyler Island			148	157	168	179
Brannan-Andrus Island						111
Ryer Island						203
Hastings Tract				75	76	72
Lindsey Slough			70	76	77	78
Prospect Island	4	16	16	17	18	18
Little Egbert Tract		45	46	49	51	51
New Hope Tract						
Staten Island						186
Terminous Tract						
Bradford Island				38	38	38
Webb Tract						
Empire Tract				81	80	81
Stewart Tract					22	32
Roberts Island, Drexler Tract, Jones Tract					422	699
Union Island						119
SE Union Island					5	7
Coney Island					15	17
Mandeville Island						
Venice Island						
Medford Island						
Shima Tract						
Veale Tract						
Victoria Island						
Locke						
Total Volume	**4**	**61**	**281**	**492**	**1,401**	**2,539**

Key:
AEP = annual exceedence probability
RD = Reclamation District
SE= Southeast
TAF = thousand acre-feet

Table 4-2. Island Inundated (Out-of-System) Volume by Annual Exceedence Probability – Restore SPFC Design Flow Capacity Approach

Island Name	Island Inundated Volume due to Flooding (TAF)					
	10% AEP	4% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Merritt Island						101
Pierson District (Courtland/RD551)						230
Sutter Island						
Grand Island						
Tyler Island			150	157	166	179
Brannan-Andrus Island					111	162
Ryer Island						
Hastings Tract					92	94
Lindsey Slough						
Prospect Island	4	17	19	20	22	23
Little Egbert Tract		48	53	55	60	61
New Hope Tract						
Staten Island						192
Terminous Tract				162	166	167
Bradford Island			38	39	40	39
Webb Tract			-	125	129	127
Empire Tract			80	82	83	83
Stewart Tract						
Roberts Island, Drexler Tract, Jones Tract						660
Union Island						
SE Union Island						9
Coney Island				15	17	18
Mandeville Island					120	120
Venice Island					70	71
Medford Island					10	21
Shima Tract					17	18
Veale Tract					17	
Victoria Island						133
Locke						3
Total Volume	**4**	**65**	**339**	**655**	**1,120**	**2,511**
Change in Volume from No Project	**0**	**5**	**58**	**163**	**-281**	**-28**

Key:
AEP = annual exceedence probability
RD = Reclamation District
SE= Southeast
TAF = thousand acre-feet

Table 4-3. Island Inundated (Out-of-System) Volume by Annual Exceedence Probability – Protect High Risk Communities Approach

Island Name	Island Inundated Volume due to Flooding (TAF)					
	10% AEP	4% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Merritt Island					85	90
Pierson District (Courtland/RD551)					196	213
Sutter Island					39	39
Grand Island					291	318
Tyler Island			148	157	162	179
Brannan-Andrus Island						119
Ryer Island					193	215
Hastings Tract				75	79	76
Lindsey Slough			72	76	79	78
Prospect Island	4	16	16	17	18	19
Little Egbert Tract		45	46	49	51	51
New Hope Tract						
Staten Island						187
Terminous Tract						
Bradford Island				38	37	39
Webb Tract						
Empire Tract				80	80	82
Stewart Tract					22	32
Roberts Island, Drexler Tract, Jones Tract					421	693
Union Island						119
SE Union Island					5	7
Coney Island						17
Mandeville Island						
Venice Island						
Medford Island						
Shima Tract						
Veale Tract						
Victoria Island						
Locke						
Total Volume	**4**	**61**	**282**	**492**	**1,758**	**2,572**
Change in Volume from No Project	**0**	**0**	**1**	**0**	**357**	**33**

Key:
AEP = annual exceedence probability
RD = Reclamation District
SE= Southeast
TAF = thousand acre-feet

Table 4-4. Island Inundated (Out-of-System) Volume by Annual Exceedence Probability – Enhance Flood System Capacity Approach

Island Name	Island Inundated Volume due to Flooding (TAF)					
	10% AEP	4% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Merritt Island						96
Pierson District (Courtland/RD551)						
Sutter Island						
Grand Island						
Tyler Island			147	154	167	126
Brannan-Andrus Island						130
Ryer Island						
Hastings Tract						94
Lindsey Slough						
Prospect Island	2	15	16	17	21	23
Little Egbert Tract		43	46	48	56	62
New Hope Tract						
Staten Island						191
Terminous Tract						167
Bradford Island					38	39
Webb Tract					125	127
Empire Tract					81	82
Stewart Tract						
Roberts Island, Drexler Tract, Jones Tract	32	71	71	84	114	176
Union Island					87	164
SE Union Island						
Coney Island					15	17
Mandeville Island						
Venice Island						
Medford Island						
Shima Tract						17
Veale Tract						
Victoria Island						
Locke						
Total Volume	**34**	**128**	**280**	**302**	**705**	**1514**
Change in Volume from No Project	**32**	**72**	**70**	**15**	**-382**	**-1045**

Key:
AEP = annual exceedence probability
RD = Reclamation District
SE= Southeast
TAF = thousand acre-feet

Table 4-5. Island Inundated (Out-of-System) Volume by Annual Exceedence Probability – State Systemwide Investment Approach

Island Name	Island Inundated Volume due to Flooding (TAF)					
	10% AEP	4% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Merritt Island					81	87
Pierson District (Courtland/RD551)						146
Sutter Island					38	40
Grand Island						311
Tyler Island			148	154	173	179
Brannan-Andrus Island						
Ryer Island						228
Hastings Tract					74	83
Lindsey Slough				71	76	82
Prospect Island	2	15	16	16	17	19
Little Egbert Tract		42	46	47	49	53
New Hope Tract						
Staten Island						193
Terminous Tract						152
Bradford Island					38	39
Webb Tract						
Empire Tract					81	82
Stewart Tract					21	30
Roberts Island, Drexler Tract, Jones Tract					419	693
Union Island						120
SE Union Island					5	7
Coney Island					15	17
Mandeville Island						
Venice Island						
Medford Island						
Shima Tract						
Veale Tract						
Victoria Island						
Locke				`		
Total Volume	**2**	**57**	**210**	**288**	**1,087**	**2,559**
Change in Volume from No Project	**-2**	**-4**	**-72**	**-204**	**-314**	**20**

Key:
AEP = annual exceedence probability
RD = Reclamation District
SE= Southeast
TAF = thousand acre-feet

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6.0 Acronyms and Abbreviations

1-D .....one-dimensional

2-D .....two-dimensional

AEP.....annual exceedence probability

AF .....acre-foot

Bay Area .....San Francisco Bay Area

Board .....Central Valley Flood Protection Board

Comprehensive Study .....San Joaquin River Basins Comprehensive Study

CVFPP .....Central Valley Flood Protection Plan

CVP.....Central Valley Project

Delta.....Sacramento-San Joaquin Delta

DICU .....Delta Island Consumptive Use

DSM2 .....Delta Simulation Model II

DWR .....California Department of Water Resources

HEC-RAS .....Hydrologic Engineering Centers River Analysis System

LiDAR.....Light Detection and Ranging

LSJR .....Lower San Joaquin River

MAF .....million acre-feet

MBK .....MBK Engineers

msl .....mean sea level

RMA .....Resources Management Associates, Inc.

S-F .....stage-frequency

SPFC .....State Plan of Flood Control

SSIA.....State Systemwide Investment Approach

SWP.....State Water Project

TAF .....thousand acre-feet

TOL .....top of levee

UNET .....Unsteady flow Through a Full NETwork of open channels computer model

UPRR.....Union Pacific Railroad

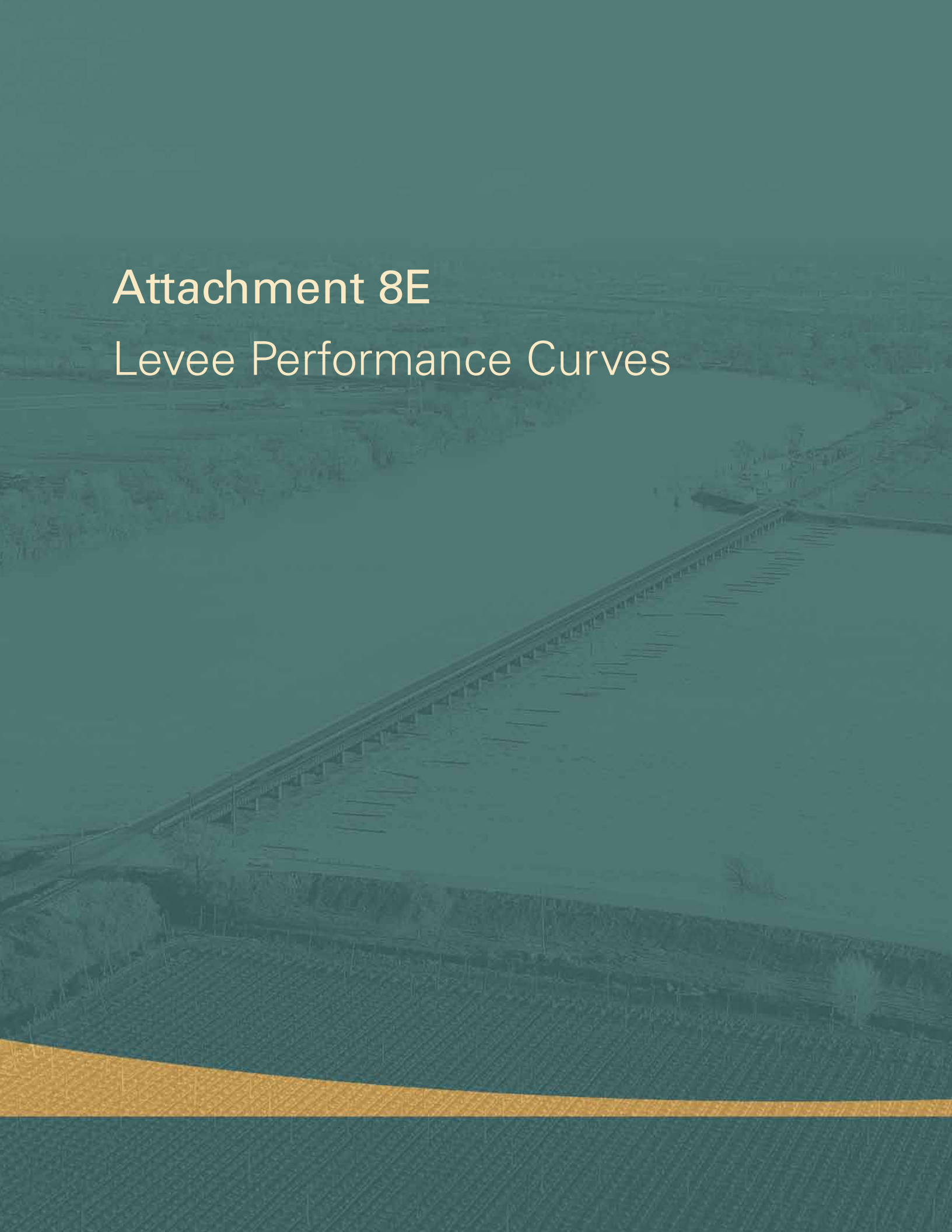
USACE.....U.S. Army Corps of Engineers

USGS.....U.S. Geological Survey

STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

Attachment 8E

Levee Performance Curves



CENTRAL VALLEY
FLOOD MANAGEMENT
PLANNING PROGRAM

FloodSAFE
CALIFORNIA

Public Draft

2012 Central Valley Flood Protection Plan

Attachment 8E: Levee Performance Curves

January 2012

STATE OF CALIFORNIA
THE NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

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1.0 Introduction

This section provides the purpose of this attachment, background information (including planning areas, goals, and approaches), an overview levee performance curves, and report organization.

1.1 Purpose of this Attachment

The hydraulic and economic analysis of the State Plan of Flood Control (SPFC) facilities for the Central Valley Flood Protection Plan (CVFPP) is based on analysis methodologies and computer models developed for the 2002 Sacramento-San Joaquin River Basins Comprehensive Study (Comprehensive Study) conducted by the U.S. Army Corps of Engineers (USACE) and the California Department of Water Resources (DWR) between 1998 and 2002 (2002). In that study, levee performance curves[1] were used to describe the ability of a given levee segment to withstand specified water surface elevations without breaching.

To reflect the most current levee conditions, new levee performance curves were developed using the recently generated data and preliminary evaluations from DWR's Urban Levee Evaluation (ULE) and Non-Urban Levee Evaluation (NULE) levee segments/reaches in the Sacramento River (North) and San Joaquin River (South) basins, in lieu of the Comprehensive Study levee performance curves. The new levee performance curves were based on geotechnical data and evaluations performed through summer 2011.

This attachment first describes the expert consultation process that resulted in equations and techniques for using ULE and NULE data/preliminary evaluations to develop levee performance curves. Next, the methodology used to develop the levee performance curves is described and applied using the ULE and NULE data/preliminary evaluations.

1.2 Background

As authorized by Senate Bill 5, also known as the Central Valley Flood Protection Act of 2008, DWR has prepared a sustainable, integrated flood management plan called the CVFPP, for adoption by the Central Valley

[1] The term levee performance curves and fragility curves are synonymous.

Flood Protection Board (Board). The 2012 CVFPP provides a systemwide approach to protecting lands currently protected from flooding by existing facilities of the SPFC, and will be updated every 5 years.

As part of development of the CVFPP, a series of technical analyses were conducted to evaluate hydrologic, hydraulic, geotechnical, economic, ecosystem, and related conditions within the flood management system and to support formulation of system improvements. These analyses were conducted in the Sacramento River Basin, San Joaquin River Basin, and Sacramento-San Joaquin Delta (Delta).

The DWR Levee Evaluations Program includes the ULE Project, covering State-federal Project (project) and appurtenant non-project levees in highly populated areas, and the NULE Project, which covers the remaining project and appurtenant non-project levees. The ULE Project includes approximately 470 miles of project and non-project levees protecting populations of 10,000 people or more, and the NULE Project includes 1,620 miles of project and non-project levees protecting populations of fewer than 10,000 people. Non-project levees are considered appurtenant and are included in the DWR Levee Evaluations Program when these levees protect part of an overflow basin partially protected by project levees or may impact the performance of project levees.

1.3 CVFPP Planning Areas

For planning and analysis purposes, and consistent with legislative direction, two geographical planning areas were important for CVFPP development (Figure 1-1):

- **SPFC Planning Area** – This area is defined by the lands currently receiving flood protection from facilities of the SPFC (see *State Plan of Flood Control Descriptive Document* (DWR, 2010)). The State of California's (State) flood management responsibility is limited to this area.

- **Systemwide Planning Area** – This area includes the lands that are subject to flooding under the current facilities and operation of the Sacramento-San Joaquin River Flood Management System (California Water Code Section 9611). The SPFC Planning Area is completely contained within the Systemwide Planning Area, which includes the Sacramento River Basin, San Joaquin River Basin, and Delta regions.

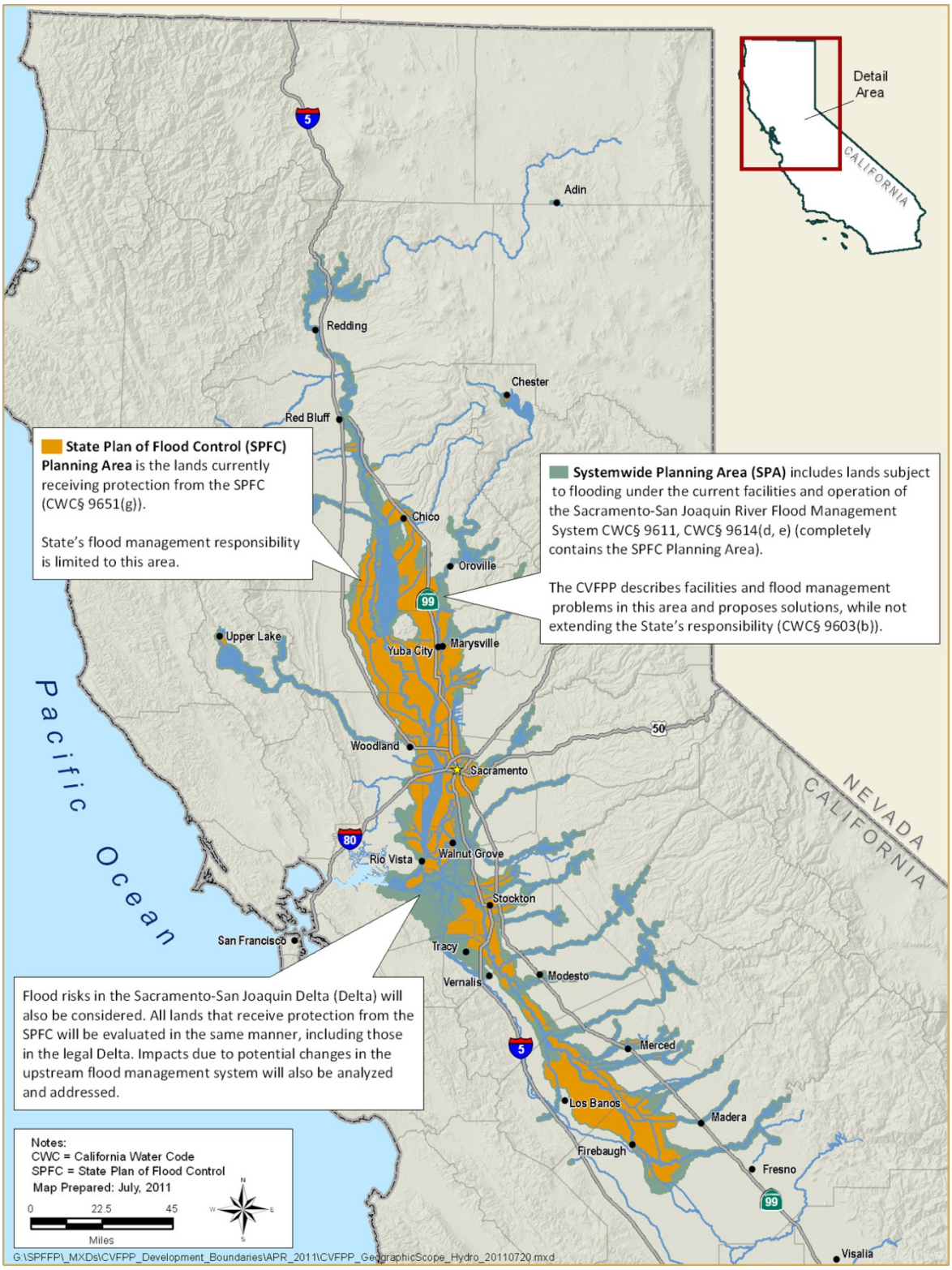


Figure 1-1. Central Valley Flood Protection Plan Planning Areas

Planning and development for the CVFPP occurs differently in these planning areas. The CVFPP focused on SPFC facilities; therefore, evaluations and analyses were conducted at a greater level of detail within the SPFC Planning Area than in the Systemwide Planning Area.

The newly developed levee performance curves are used to support geotechnical levee reliability for hydraulic models of the Sacramento and San Joaquin rivers and major tributaries contained within the SPFC Planning Area. Levee performance curves located within the Stockton Area are discussed in Attachment 8C: Riverine Channel Evaluations.

1.4 2012 CVFPP Planning Goals

To help direct CVFPP development to meet legislative requirements and address identified flood-management-related problems and opportunities, a primary and four supporting goals were developed:

- **Primary Goal** – Improve Flood Risk Management
- **Supporting Goals**:
  - Improve Operations and Maintenance
  - Promote Ecosystem Functions
  - Improve Institutional Support
  - Promote Multi-Benefit Projects

The work described in this attachment is related to the primary goal of improving flood risk management. The levee performance curves help to understand and model the way SPFC levees react to floodwaters and what improvements to the levees may be required to provide desired levels of protection.

1.5 2012 CVFPP Planning Approaches

In addition to **No Project**, three fundamentally different approaches to flood management were initially compared to explore potential improvements in the Central Valley. These approaches are not alternatives; rather, they bracket a range of potential actions and help explore tradeoffs in costs, benefits, and other factors important in decision making. The approaches are as follows:

- **Achieve SPFC Design Flow Capacity** – Address capacity inadequacies and other adverse conditions associated with existing SPFC facilities, without making major changes to the footprint or operation of those facilities.

- **Protect High Risk Communities** – Focus on protecting life safety for populations at highest risk, including urban areas and small communities.

- **Enhance Flood System Capacity** – Seek various opportunities to achieve multiple benefits through enhancing flood system storage and conveyance capacity.

Comparing these approaches helped identify the advantages and disadvantages of different combinations of management actions, and demonstrated opportunities to address the CVFPP goals to different degrees.

Based on this evaluation, a **State Systemwide Investment Approach** was developed that encompasses aspects of each of the approaches to balance achievement of the goals from a systemwide perspective, and includes integrated conservation elements. Figure 1-2 illustrates this plan formulation process.

The levee performance curves developed from the ULE and NULE data describe the No Project condition of the levees. Each of the approaches described above would require modifications or improvements to levees with levee performance curves that result in levels of protection lower than desired for a given location in the particular approach.

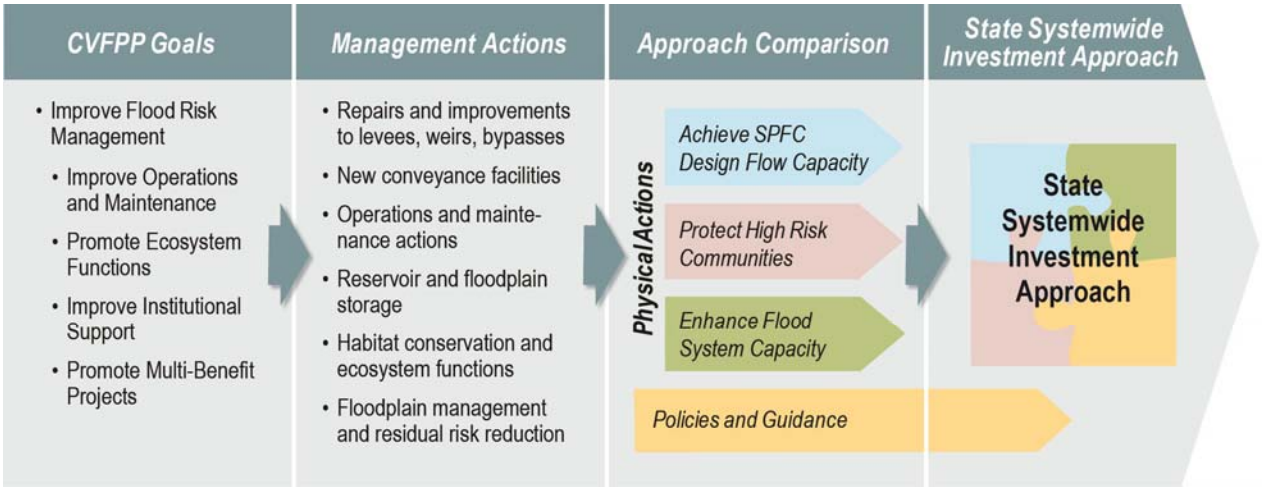


Figure 1-2. Formulation Process for State Systemwide Investment Approach

1.6 Levee Performance Curves

Levee performance curves developed for the CVFPP provide relationships between river water surface elevation (stage) and the probability that the levee segment will fail when exposed to that water surface elevation without human intervention (floodfighting). In this application, "failure" is defined as a levee breach in which water from the waterside of the levee is allowed to flow in an uncontrolled manner to the landside of the levee, potentially resulting in loss of life, personal injury, property damage, and economic loss. The approach used to develop levee performance curves herein generally follows a process similar to that described in the USACE Manual ETL 1102-2-556 (USACE, 1999).

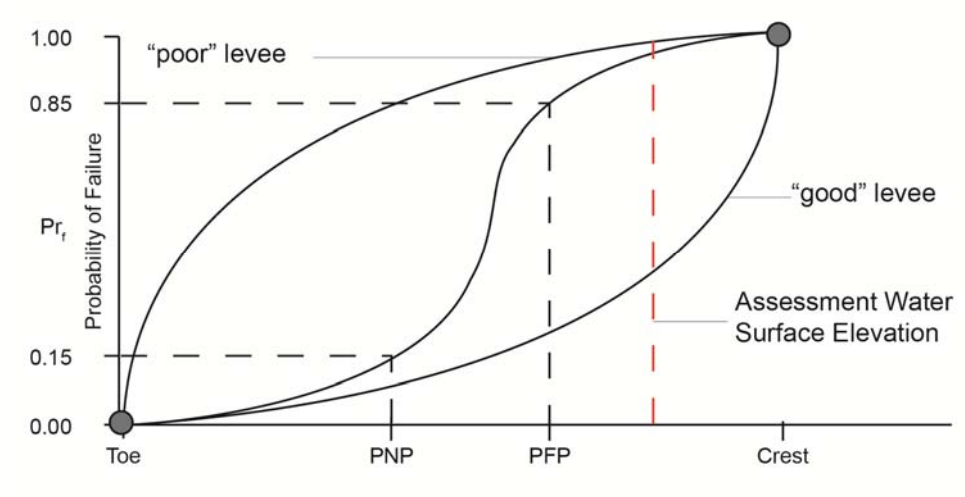
Figure 1-3 provides three example levee performance curves. The probability of failure is plotted on the vertical (dependent) axis and water surface elevation is plotted on the horizontal (independent) axis. Probability of failure is shown on the vertical or dependent axis because probability of failure is a function the channel stage. The range of water surface elevations of interest begin at the landside toe of a levee, below which the probability of failure is assumed to be zero, to the levee crest, where the probability of failure is assumed to be 100 percent because of overtopping.

The three example levee performance curves shown in Figure 1-3 represent the performance of a "poor" levee, a "good" levee, and a "generic" levee. The performance for the good levee shows a low probability of failure until higher water levels are reached and is concave upward, while the performance curve for the poor levee shows a high probability of failure even at low water surface elevations, and is concave downward (convex). The performance curve for the generic levee includes elements of both the poor and the good levees and follows a characteristic levee performance curve "s" shape.

An assessment water surface elevation is also shown in Figure 1-3. The ULE Project has performed geotechnical evaluations for a number of water surface elevations (e.g., 100- and 200-year flood levels). For the NULE Project, geotechnical assessments have been completed that consider likely levee performance at only a single design or assessment water surface elevation (typically the 1955/1957 water surface profile) (Kleinfelder, 2010 and URS, 2010b). For areas that require further study, additional NULE Project work may include geotechnical evaluations as for those areas as needed.

Figure 1-3 also shows the probable non-failure point (PNP) and probable failure point (PFP), representing 15 percent and 85 percent probabilities of

failure, respectively. Previous studies developing levee performance curves for Central Valley levees (e.g., Comprehensive Study, USACE, 2002) have made use of these terms, but they are not used in developing the levee performance curves for the CVFPP. The PFP, or 85 percent probability of failure, however, is used to set the levee failure elevation for use in the hydraulic models.



Key:
Crest = levee crest
PFP = probable failure point
PNP = probable nonfailure point
Toe = levee toe

Figure 1-3. Conceptual Levee Performance Curve Examples

1.7 Report Organization

Organization of this document is as follows:

- Section 1 introduces and describes the purpose of this attachment. It also provides an overview of levee performance curves.
- Section 2 provides a summary of results and findings.
- Section 3 describes the methodology used to develop the levee performance curves.
- Section 4 describes the results in more detail.
- Section 5 contains references for the sources cited in this document.
- Section 6 lists acronyms and abbreviations used in this document.

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2.0 Levee Performance Curve Locations

A total of 307 new levee performance curves were developed using ULE and NULE data and methodologies described in Section 3. Levee performance curves were grouped according their location within the SPFC Planning Area. The SPFC Planning Area is separated into different impact areas. An impact area is a unique, contiguous floodplain located along a stream or waterway. Most impact areas in the SPFC Planning Area are protected by levees. At least one levee performance curve was developed for each of the impact areas, where levees are present, in the SPFC Planning Area. Many of the impact areas have more than one levee. Tables 2-1 and 2-2 show the number of levee performance curves developed for each impact area and the methodology used (ULE/NULE), and Figures 2-1 and 2-2 map the locations of the levee performance curves for the Sacramento River Basin and San Joaquin River Basin, respectively.

Table 2-1. Levee Performance Curve Summary for Sacramento River Basin

Impact Area	Name	Number of Levee Performance Curves	Methodology
SAC01	Woodson Bridge East	0	N/A
SAC02	Woodson Bridge West	0	N/A
SAC03	Hamilton City	1	NULE[1]
SAC04	Capay	1	NULE[1]
SAC05	Butte Basin	10	NULE
SAC06	Butte City	1	NULE
SAC07	Colusa Basin North	5	NULE Anomalous[2]
SAC08	Colusa	1	NULE
SAC09	Colusa Basin South	4	NULE Anomalous[2]
SAC10	Grimes	1	NULE
SAC11	RD 1500 West	2	NULE
SAC12	Sycamore Slough	1	NULE[1]
SAC13	Knight's Landing	1	NULE[1]
SAC14	Ridge Cut (North)	3	NULE
SAC15	Ridge Cut (South)	3	NULE
SAC16	RD 2035	6	ULE
SAC17	East of Davis	3	ULE

Table 2-1 Levee Performance Curve Summary for Sacramento River Basin (contd.)

Impact Area	Name	Number of Levee Performance Curves	Methodology
SAC18	Upper Honcut	1	NULE[1]
SAC20	Gridley	1	ULE
SAC21	Sutter Buttes East	3	NULE[1]
SAC22	Live Oak	1	ULE
SAC23	Lower Honcut	2	NULE[1]
SAC24	Levee District No. 1	6	NULE
SAC25	Yuba City	2	ULE Anomalous Hazard[2]
SAC26	Marysville	3	ULE Anomalous Hazard[2]
SAC27	Linda-Olivehurst	3	NULE Anomalous Hazard[1,2]
SAC28	RD 384	2	ULE
SAC29	Best Slough	8	NULE Anomalous Hazard [2]
SAC30	RD 1001	5	NULE
SAC32	RD 70-1660	5	NULE
SAC33	Meridian	1	NULE
SAC34	RD 1500 East	1	NULE
SAC35	Elkhorn	7	NULE
SAC36	Natomas	4	NULE Anomalous Hazard[2]
SAC37	Rio Linda	1	NULE[1]
SAC38	West Sacramento	1	ULE
SAC39	RD 900	1	ULE
SAC40	Sacramento North	2	ULE
SAC41	RD 302	1	NULE
SAC42	RD 999	3	NULE
SAC43	Clarksburg	1	NULE
SAC44	Stone Lake	3	NULE Anomalous Hazard[2]
SAC45	Hood	1	NULE
SAC46	Merritt Island	2	NULE
SAC47	RD 551	2	NULE
SAC48	Courtland	1	NULE
SAC49	Sutter Island	3	NULE
SAC51	Locke	2	NULE
SAC52	Walnut Grove	1	NULE
SAC53	Tyler Island	1	NULE
SAC54	Andrus Island	8	NULE Anomalous Hazard[2]

Table 2-1 Levee Performance Curve Summary for Sacramento River Basin (contd.)

Impact Area	Name	Number of Levee Performance Curves	Methodology
SAC55	Ryer Island	5	NULE Anomalous Hazard[2]
SAC56	Prospect Island	N/A	TOL
SAC57	Twitchell Island	1	NULE
SAC58	Sherman Island	8	NULE Anomalous Hazard[2]
SAC59	Moore	6	NULE Anomalous Hazard[2]
SAC60	Cache Slough	1	NULE
SAC61	Hastings	1	NULE
SAC62	Lindsey Slough	3	NULE
SAC63	Sacramento South	1	ULE

Notes:
[1] Additional evaluations were required; initial ULE/NULE Project evaluations did not evaluate/assess levees in this impact area.
[2] A short anomalous section within the impact area had a lower levee performance curve than surrounding levees. Used data that resulted in the most conservative (highest hazard) levee performance curve.
Key:
N/A = not applicable – no levee
No. = number
NULE = Non-Urban Levee Evaluations
RD = Reclamation District
TOL = top of levee
ULE = Urban Levee Evaluations

Table 2-2. Levee Performance Curve Summary for San Joaquin River Basin

Impact Area	Name	Number of Levee Performance Curves	Methodology
SJ01	Fresno	0	N/A
SJ02	Fresno Slough East	3	NULE
SJ03	Fresno Slough West	1	NULE[1]
SJ04	Mendota	1	NULE[1]
SJ05	Chowchilla Bypass	2	NULE
SJ06	Lone Willow Slough	6	NULE
SJ07	Mendota North	1	NULE
SJ08	Firebaugh	1	NULE[1]
SJ09	Salt Slough	3	NULE
SJ10	Dos Palos	Used SJ09[3]	NULE
SJ11	Fresno River	5	NULE
SJ12	Berenda Slough	6	NULE
SJ13	Ash Slough	2	NULE
SJ14	Sandy Mush	6	NULE

Table 2-2. Levee Performance Curve Summary for San Joaquin River Basin (contd.)

Impact Area	Name	Number of Levee Performance Curves	Methodology
SJ15	Turner Island	3	NULE
SJ16	Bear Creek	2	NULE
SJ17	Deep Slough	4	NULE
SJ18	West Bear Creek	2	NULE
SJ19	Fremont Ford	1	NULE
SJ20	Merced River	0	N/A
SJ21	Merced River North	3	NULE Anomalous Hazard[2]
SJ22	Orestimba	2	NULE Anomalous Hazard[2]
SJ23	Tuolumne South	2	NULE
SJ24	Tuolumne River	1	NULE
SJ25	Modesto	0	N/A
SJ26	3 Amigos	3	NULE
SJ27	Stanislaus South	2	NULE
SJ28	Stanislaus North	4	NULE
SJ29	Banta Carbona	4	NULE
SJ30	Paradise Cut	1	NULE
SJ31	Stewart Tract	6	NULE Anomalous Hazard[2]
SJ32	East Lathrop	1	NULE[1]
SJ33	Lathrop/ Sharpe	1	NULE[1]
SJ34	French Camp	1	NULE[1]
SJ35	Moss Tract	1	NULE[1]
SJ36	Roberts Island	5	NULE
SJ37	Rough and Ready Island	1	ULE
SJ38	Drexler Tract	2	NULE[1]
SJ39	Union Island	1	NULE
SJ40	Southeast Union Island	3	NULE
SJ41	Fabian Tract	1	NULE
SJ42	RD 1007	1	NULE[1]
SJ43	Grayson	1	NULE

Notes:
[1] Additional evaluations were required; initial ULE/NULE Project evaluations did not evaluate/assess levees in this impact area.
[2] A short anomalous section within the impact area had a lower levee performance curve than surrounding levees. Used data that resulted in the most conservative (highest hazard) levee performance curve.
[3] SJ10 is part of SJ09; therefore, SJ10 used the same levee performance curve as SJ09.

Key:
N/A = not applicable – no levee
NULE = Non-Urban Levee Evaluation
RD = Reclamation District
ULE = Urban Levee Evaluations

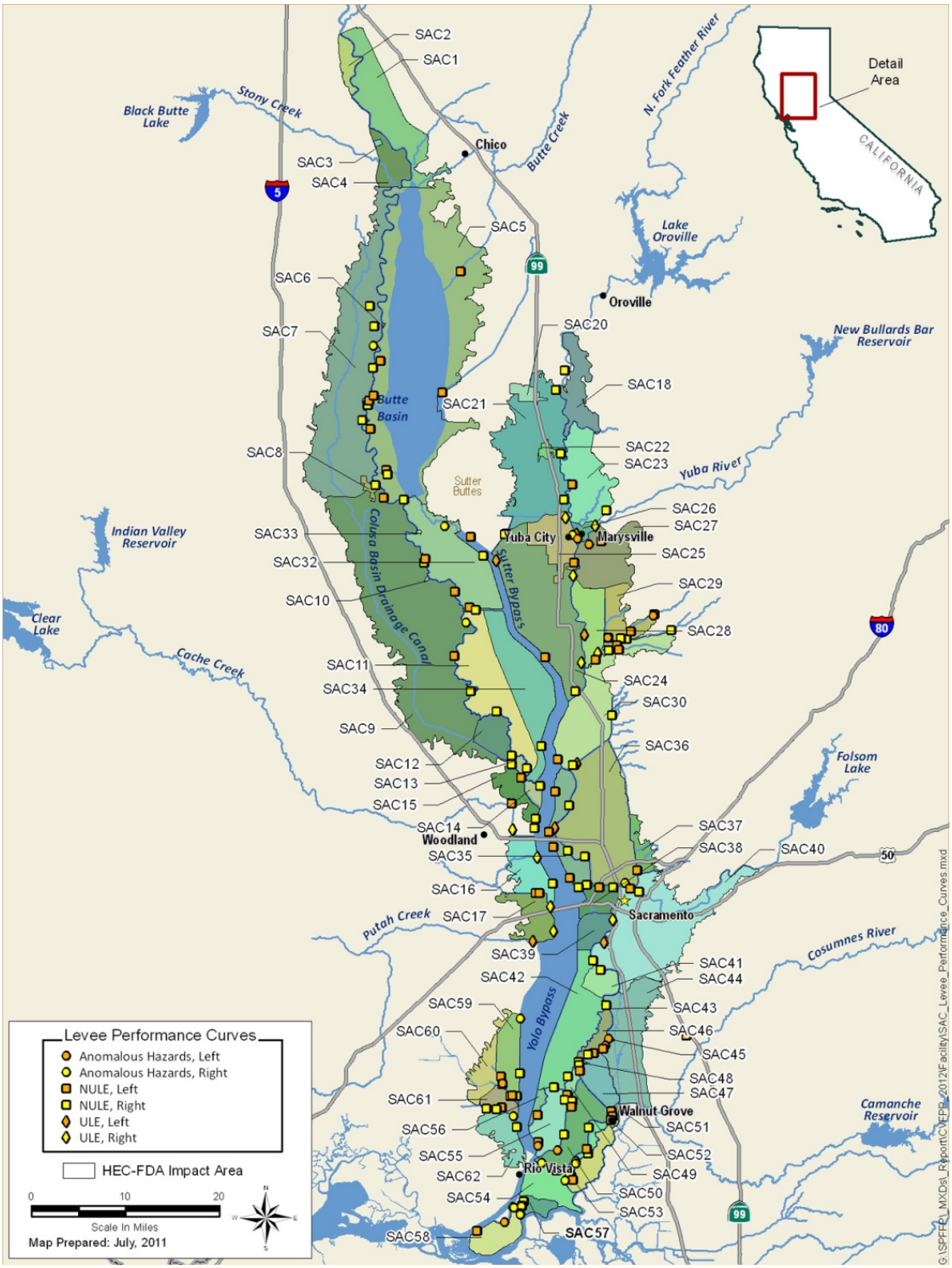


Figure 2-1. Levee Performance Curve Locations, Sacramento River Basin

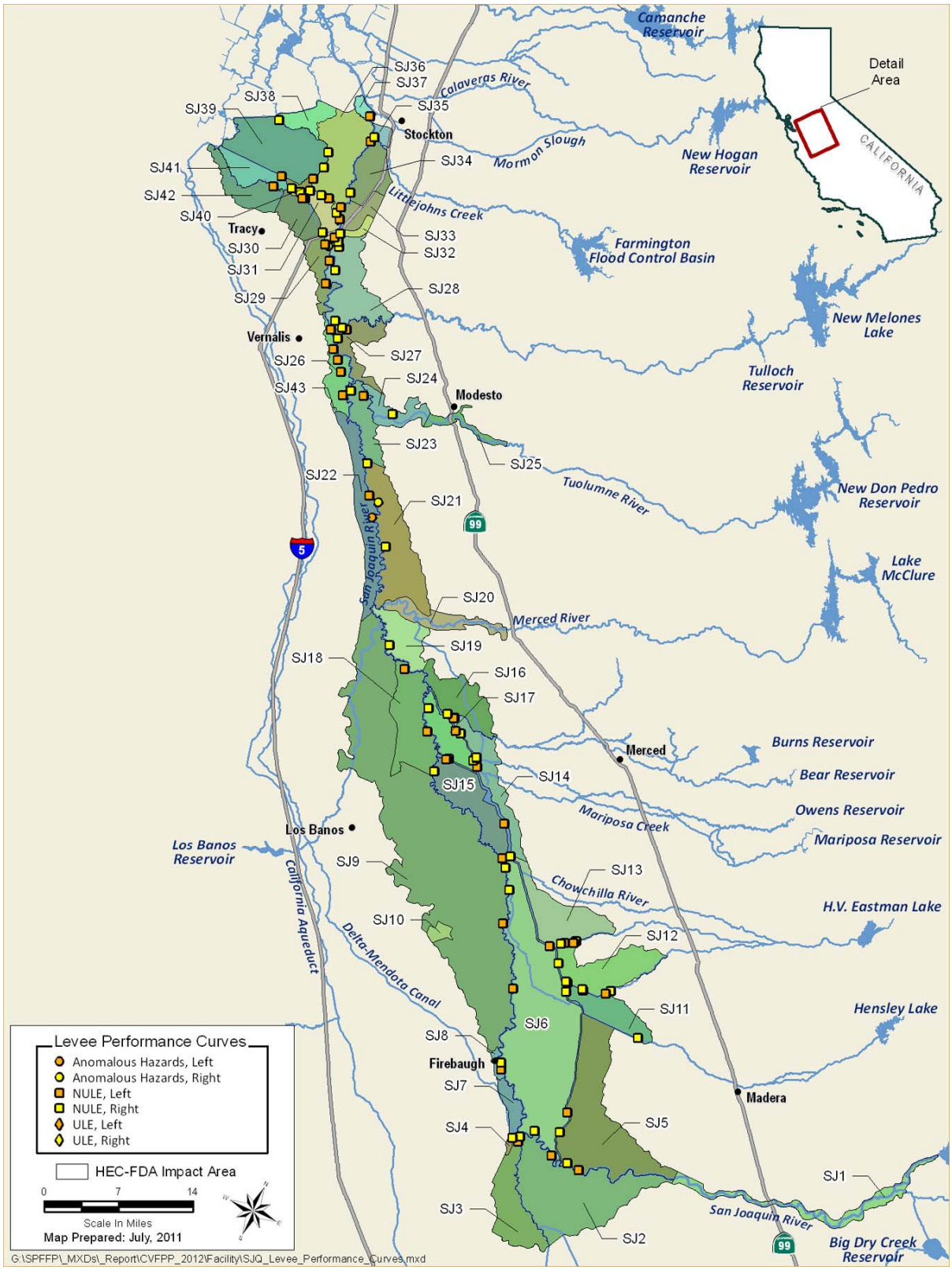


Figure 2-2. Levee Performance Curve Locations, San Joaquin River Basin

3.0 Methodology

This section describes the methodology used to develop levee performance curves, a description of the sources of the data, and a detailed description of the process for developing levee performance curves for both ULE and NULE Project segments.

Note that the detailed description of the process for developing levee performance curves is first described for NULE Project segments. This is because some of the data developed for the NULE levee performance curves were used in the ULE assessment, as additional ULE work has yet to be completed.

3.1 Developing Levee Performance Curve Methodology Overview

The methodology used to develop levee performance curves included review of the data, formulation of a levee expert panel, and a sensitivity analysis.

3.1.1 Data Review

To begin the task of developing levee performance curves, two levee evaluation teams, one in the north study area and one in the south study area, were formed. These teams reviewed the data collected and conclusions drawn during preparation of the NULE Geotechnical Assessment Report (GAR), and the hazard maps developed to support the ULE study areas in the *Flood Control System Status Report* (FCSSR) (DWR, 2011) (URS, 2010a). Each team compiled and summarized key performance events relevant to preparation of levee performance curves, such as information related to historical levee failures and estimates of the water surface elevation during these events, using readily available records.

Based on review and compilation of this information, a standard set of levee performance curves was developed for application to ULE and NULE levee segments. The approach used to develop levee performance curves generally follows a process similar to that described in *USACE Manual Engineering Technical Letter (ETL) 1102-2-556* (USACE, 1999).

3.1.2 Levee Expert Panel

A levee Expert Panel was formed to provide technical expertise, advice, and review (Table 3-1). This panel met multiple times, from fall 2010 through spring 2011, during the development of the levee performance curve methodology. The comments and suggestions of the levee Expert Panel were incorporated in the development of two separate levee performance curve tools (Excel workbooks), one for ULE levees and one for NULE levees. These tools incorporated and made use of data generated during earlier ULE and NULE work, and provided the user options for generating levee performance curves.

Table 3-1. Levee Expert Panel

Name	Organization
David Ford (facilitator)	David Ford Consulting Engineers
Ray Costa	Consultant to DWR
Mike Inamine	DWR
Steve Verigen	GEI
Les Harder	HDR
Scott Anderson	Kleinfelder
Pat Dell	Neil O. Anderson and Associates
Ram Kulkarni	URS Corporation
Michael Ramsbotham	USACE
Ed Ketchum	USACE

Key:
DWR = California Department of Water Resources
USACE = U.S. Army Corps of Engineers

3.1.3 Sensitivity Analysis

A sensitivity analysis was performed to evaluate the effect on estimated damage from varying parameters in the levee performance curve tool. Additionally, preliminary hydraulic modeling was conducted using a complete set of preliminary levee performance curves to evaluate (1) how these draft levee performance curves worked in the context of the existing hydraulic model, and (2) the number of levee failures predicted using the model and preliminary levee performance curves. These results were used to assess how well the results from the models approximated general historical flood conditions. Refinement of the preliminary ULE and NULE levee performance curve tools followed the sensitivity analysis and preliminary hydraulic modeling.

3.2 Sources of Levee Performance Data

The ULE Project has subdivided levees into reaches that are typically on the order of thousands of feet long. The NULE Project has assessed individual levee segments, which are generally two to five miles long, but were as long as 25 miles at some locations. Results of the ULE and NULE projects are summarized in the FCSSR (DWR, 2011).

3.2.1 Urban Levee Evaluations Project

The ULE Project evaluated 470 miles of levees. Based on an initial phase of field explorations, laboratory testing, and subsequent geotechnical analysis, levees in each urban study area were subdivided into reaches, typically 1,000 feet to 3,000 feet long. For the ULE study areas, the ULE teams reviewed data and analysis results from the ULE Technical Review Memoranda; Phase 1 Geotechnical Data Reports; Phase 1 Geotechnical Evaluation Reports; and where already prepared, Supplemental Geotechnical Data Reports. Each team compiled and summarized key performance events relevant to preparation of levee performance curves, such as information related to historical levee failures and estimates of the water surface elevation during these events, using readily available records.

3.2.2 Non-Urban Levee Evaluations Project

During the geotechnical assessment for the NULE Project, existing data were assessed to assign hazard categories to 1,620 mile of levees and results were provided in the GARs (URS, 2010b; Kleinfelder, 2010), as follows:

- **Low (A)** – When water reaches the assessment water surface elevation, there is a low likelihood of either levee failure or the need to floodfight to prevent levee failure.

- **Moderate (B)** – When water reaches the assessment water surface elevation, there is a moderate likelihood of either levee failure or the need to floodfight to prevent levee failure.

- **High (C)** – When water reaches the assessment water surface elevation, there is a high likelihood of either levee failure or the need to floodfight to prevent levee failure.

- **Lacking Sufficient Data (LD)** – Currently lacking sufficient data regarding past performance or hazard indicators.

Floodfight refers to actions taken to prevent geotechnical levee failure, not actions to prevent overtopping.

The LD category indicates that the available data do not resolve potential discrepancies between the expected performance of the levee and actual past performance, or that existing data are contradictory or ambiguous. The category does not necessarily indicate that insufficient data were available to assess the levee segment. Where assessment data were not available, the levee segment was not assessed.

The categorization was done for each of four failure modes: under-seepage, stability, through-seepage, and erosion.

3.3 Levee Performance Curves for NULE Levee Segments

During the geotechnical assessment for the NULE Project, existing data were collected and used to categorize the levees. As described above, each levee segment was categorized as Low (A), Moderate (B), High (C), or Lacking Sufficient Data (LD), and the levee was cumulatively categorized as a whole. The categorization was done for each of four failure modes: under-seepage, stability, through-seepage, and erosion. It is important to note that the categorization was performed for only one water level, the assessment water surface elevation, which, where available, was the 1955/1957 water surface. All NULE categorizations and results are, therefore, for the single assessment water surface elevation. To produce levee performance curves for each NULE segment, levee performance curves were developed for each failure mode. These independent failure mode levee performance curves were then mathematically combined to produce the cumulative or overall levee performance curve for the segment or reach. Thus, two levees with similar failure mode categorizations and similar topographic profiles had very similar levee performance curves.

Topographic information necessary for levee performance curve development included levee crest elevation, levee toe elevation, and assessment water surface elevation. Topographic data used for developing levee performance curves were based on two sources: levee center line survey data obtained from the California Levee Database (CLD), and project-specific light detection and ranging (LiDAR) surveys.

Few additional data were used to generate the NULE levee performance curves; however, abundant data on past performance and past floodwater levels collected during the geotechnical assessment were used to calibrate and review the parameters selected in developing the levee performance curves.

To make use of the failure mode categorizations assigned in NULE to each segment, it was necessary to assign a probability of failure at the assessment water surface elevation for the Low (A), Moderate (B), and High (C) categories. These probability of failure values were not explicitly included in the NULE GAR (URS, 2010b), and part of the efforts expended in this task involved discussions and sensitivity analyses to constrain the values used for each category. Based on review of the sensitivity analysis and input from the levee Expert Panel, the values for each category, at the assessment water surface elevation, were Low (A), 0.5 percent; Moderate (B), 2 percent; and High (C), 16 percent. These points, which define the levee performance curve at the assessment water surface elevation, are called the "pin points." Figure 3-1 shows an example of three schematic levee performance curves for each hazard category (Low (A), Moderate (B) and High (C) curves) for a single failure mode. The pin points are where each curve intersects the assessment water surface elevation and represents the probability of failure at the assessment water surface elevation for each category. It is important to note that the values used here for the pin-point probabilities are for the purposes of this levee performance curve effort; they should not be retroactively imposed on the NULE GAR.

Thus, for NULE levee performance curves, three water surface elevations were used to define the levee performance curves: (1) the levee toe elevation, at which the probability of failure is assumed to be zero, (2) the levee crest elevation, at which overtopping would occur and the probability of failure is set to 100 percent, and (3) the pin-point at the assessment water surface elevation (Figure 3-1). The NULE levee performance curve Excel tool fitted a simple curve through these three points for each failure mode using the assigned probability of failure at the assessment water surface elevation. Below the assessment water surface elevation, the curve was fitted using a "concavity factor" that ranges between zero and 1, with zero yielding a curve of constant slope of no concavity, and 1 yielding a curve that is concave upward and very steep at the assessment water surface elevation. For this analysis, a concavity factor of 0.5 was used for all levee performance curves based on the results of the sensitivity analyses. The levee performance curves are extended above the assessment water surface elevation based on their slope as they approach the assessment water level. Low (A) and Moderate (B) curves extend at constant slope (although the example in Figure 3-1 shows a curving line), and High (C) curves roughly mirror the shape of the curve below the assessment water surface elevation. The same probability values are used for every Low (A), Moderate (B), or High (C) pin-point (e.g., all Moderate (B) levee performance curves were assigned a probability of failure of 2 percent at the assessment water surface elevation, independent of the failure mode, the size of the levee, or other differences in levees). For levee segments categorized LD, pin-point values between those of Low (A) and Moderate (B), or Moderate (B) and

High (C) were used, depending on the nature of the LD categorization (e.g., LD (Low (A) or Moderate(B)) vs. LD (Moderate(B) or High(C))).

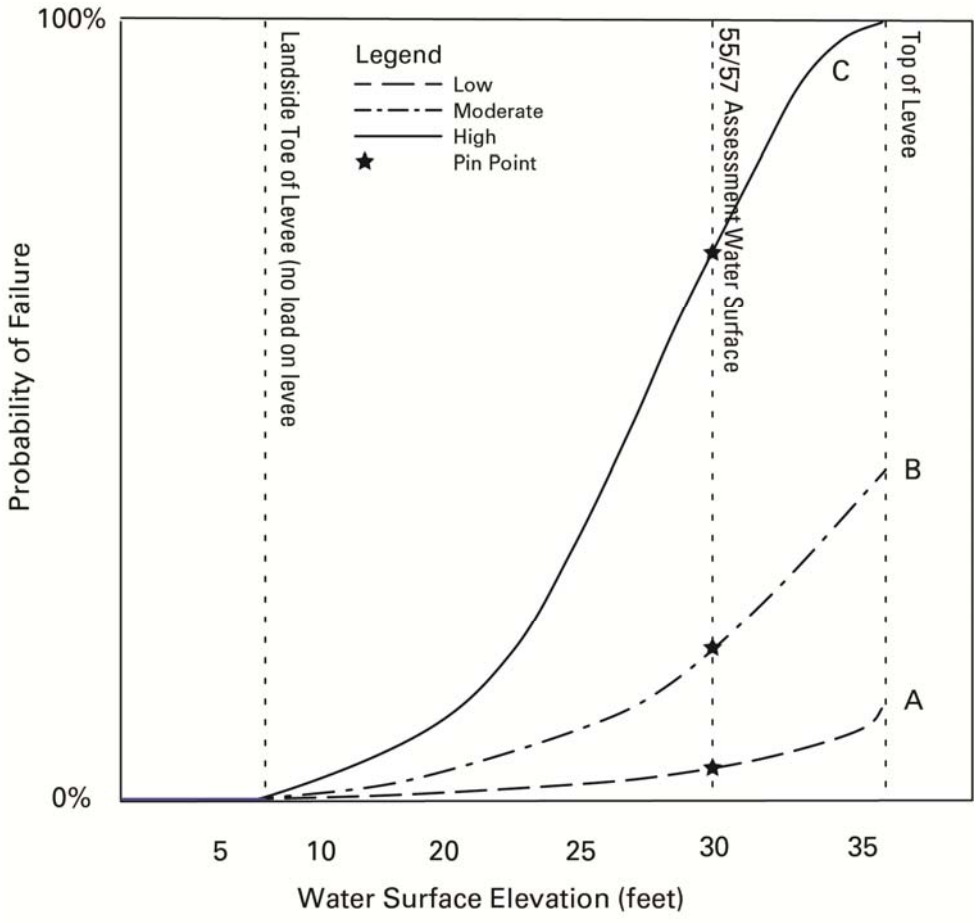


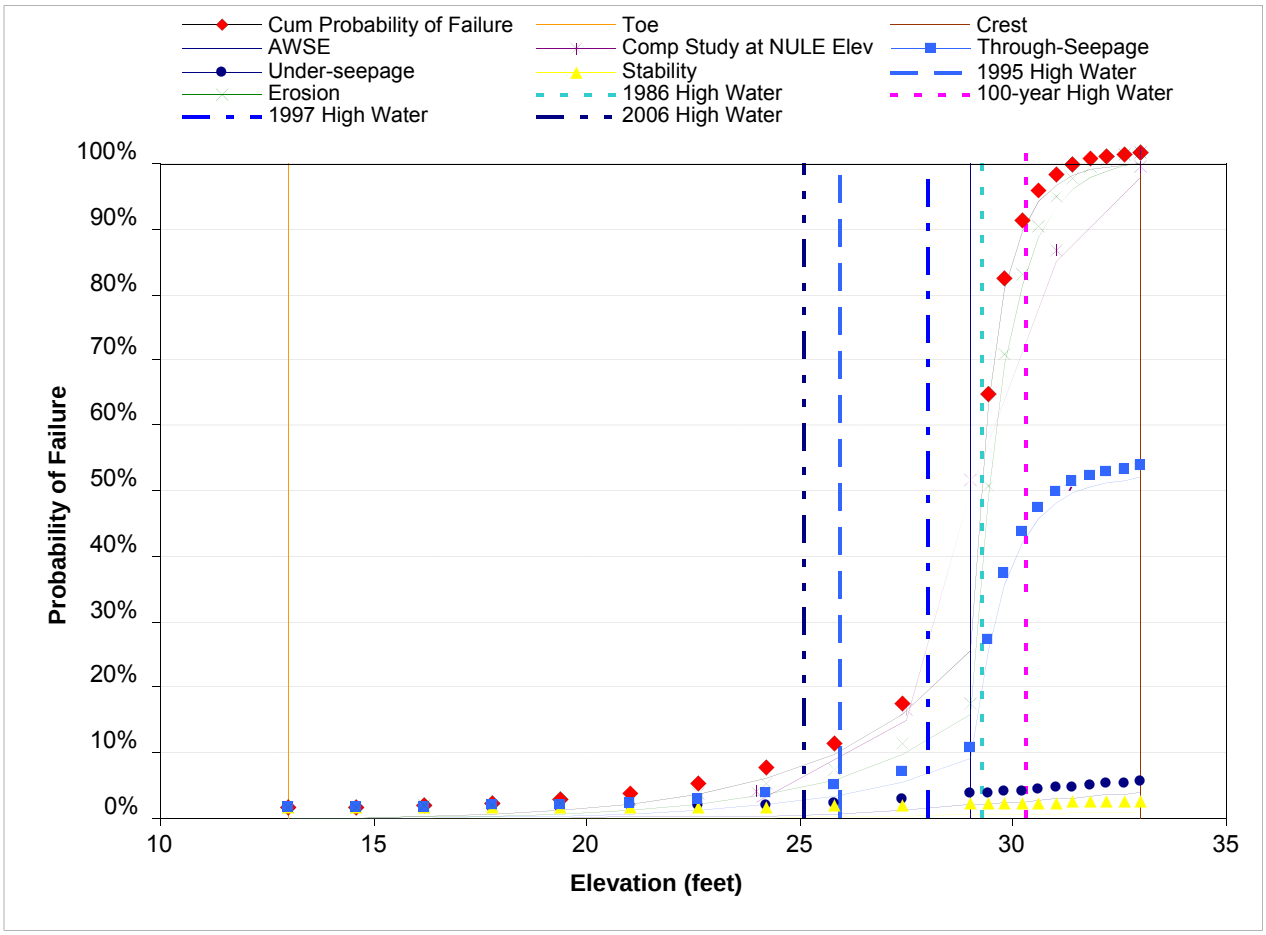
Figure 3-1. Conceptual NULE Levee Performance Curves for Hazard Categories Low (A), Moderate (B), and High (C)

Past flood information (water surface elevation and record of performance) can be used to calibrate or validate levee performance curves for individual segments. Basin-wide compilations of past performance were used as guidance in constraining the chosen pin-point probability values. The four individual failure mode levee performance curves were mathematically combined using the conventional probabilistic summing expression:

$$Cumulative\ Pf = 1 - (1 - Pf(underseepage)) * (1 - Pf(stability)) * (1 - Pf(through\ seepage)) * (1 - Pf(erosion)) \qquad \text{Equation 1}$$

Figure 3-2 shows an example of output generated by the NULE levee performance curve Excel tool. Individual failure modes for this example levee segment were categorized as Moderate (B) for under-seepage; Low (A) for stability; LD (Moderate (B) or High (C)) for through-seepage; and High (C) for erosion in the GAR. The example levee's landside toe is at elevation 13 feet, the crest is at elevation 33 feet, and the assessment water surface was at elevation 29 feet, or 4 feet below the levee crest. The dark blue line with circles shows the levee performance curve for under-seepage, the yellow line is stability, the light blue line with squares is through-seepage, and the green line is erosion. The black line shows the combined or cumulative levee performance curve when the failure mode levee performance curves are summed using the expression above. Also shown are vertical lines depicting the assessment water surface elevation and water surface elevations for a number of historical high-water events. The magenta lines show the levee performance curves used in the Comprehensive Study (USACE, 2002). The solid line shows the Comprehensive Study curve at the levee crest elevation used in the NULE Program, which was estimated based on LiDAR and CLD information. The dashed magenta curve shows the Comprehensive Study curve tied to the elevation used in the Comprehensive Study.

When levee locations were identified where elevations used in the Comprehensive Study hydraulic models were different from the top-of-levee elevations used in the ULE and NULE projects, which are based on more recent and better constrained topographic data, the ULE and NULE elevations were used.



Note: These curves represent a levee segment with the following hazard categories from the GAR: Moderate (B) for under-seepage, Low (A) for stability, LD (Moderate (B) or High (C)) for through-seepage, and High (C) for erosion.
Key:
AWSE = assessment water surface elevation
Cum = cumulative
Elev = elevation
NULE = Non-Urban Levee Evaluations

Figure 3-2. Example NULE Levee Performance Curve

Note that the levee performance curves for the failure modes categorized A (stability) and B (under-seepage) extend above the assessment water surface to the elevation of the levee crest at nearly a constant slope. This means that this example levee is not expected to fail because of either of these failure modes, even when the water surface reaches the levee crest. The failure mode levee performance curves for through-seepage and erosion have the more classic “s”-shaped curves, as does the combined or cumulative levee performance curve. This example levee performance curve shows that there is little probability of the levee failing at low water

levels, and that the cumulative probability of failure at the assessment water surface elevation is about 25 percent.

3.4 Levee Performance Curves for ULE Levee Segments

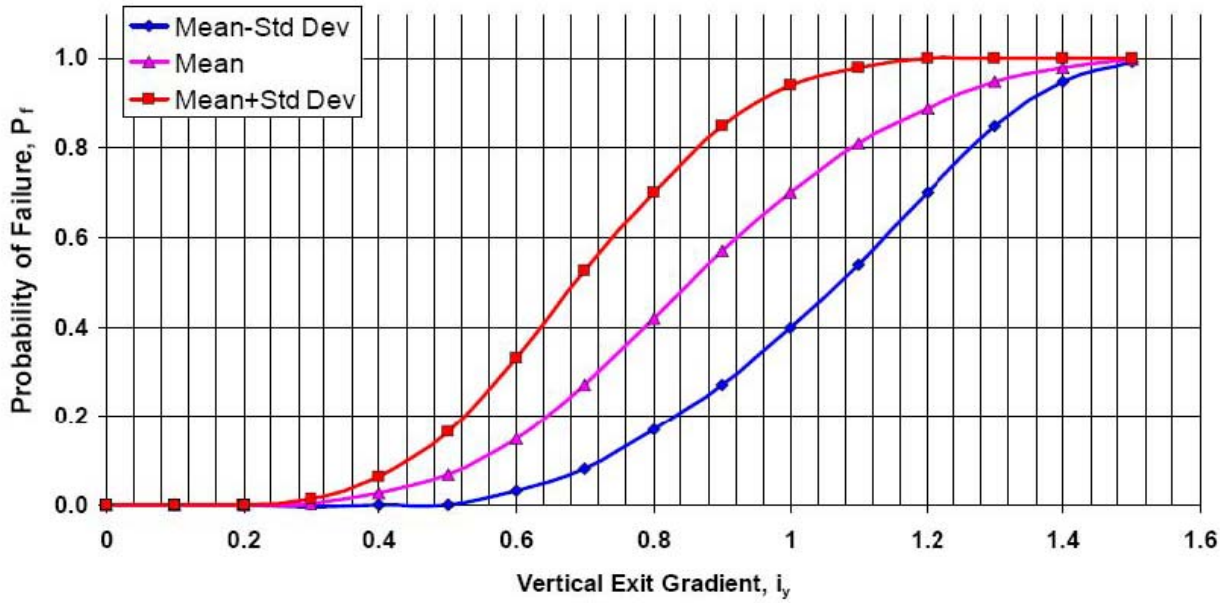
To support the 2012 CVFPP modeling, representative reaches and corresponding cross sections within individual urban study areas were selected for development of levee performance curves. A cumulative ULE levee performance curve for each of these selected cross sections was prepared based on the individual curves for the same four failure modes: assessed in the NULE program (under-seepage, stability, through-seepage, and erosion).

For steady-state under-seepage and steady-state stability, historical data and field and laboratory geotechnical data collected in the initial phase of the ULE Project were used as input to calculate average vertical exit gradients (i) and stability factors of safety (FS) for various flood elevations for each respective cross-section location.

To establish the relationships between i and probability of failure (P_f) and between stability FS and P_f, input from the levee Expert Panel and program-specific information were used to generate classic “s”-shaped curves (see Figure 3-3) (note that Figure 3-3 is a generic example). For this study, the following control points were used to develop the applicable “s” curves:

- Under-seepage i=0.5, P_f =1 percent and i=0.9, P_f =50 percent
- Stability FS=1.4, P_f =1 percent and FS=1.0, P_f =50 percent

Using these relationships for under-seepage and stability, and correlating them to specific results at various river water surface elevations, levee performance curves for under-seepage and stability were then developed using the same concavity factor (0.5) used in development of the NULE levee performance curves. Figures 3-4 and 3-5 show examples of ULE levee performance curves for the under-seepage and stability failure modes.



Source: DWR, 2007

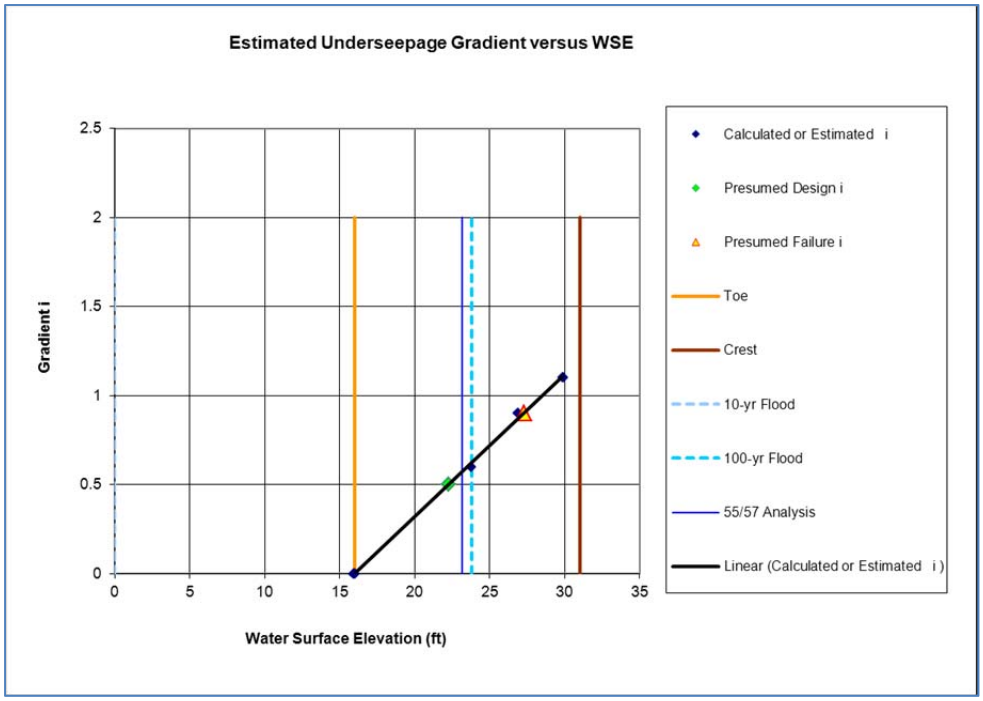
Figure 3-3. Relationship Between Vertical Exit Gradient and Probability of Failure

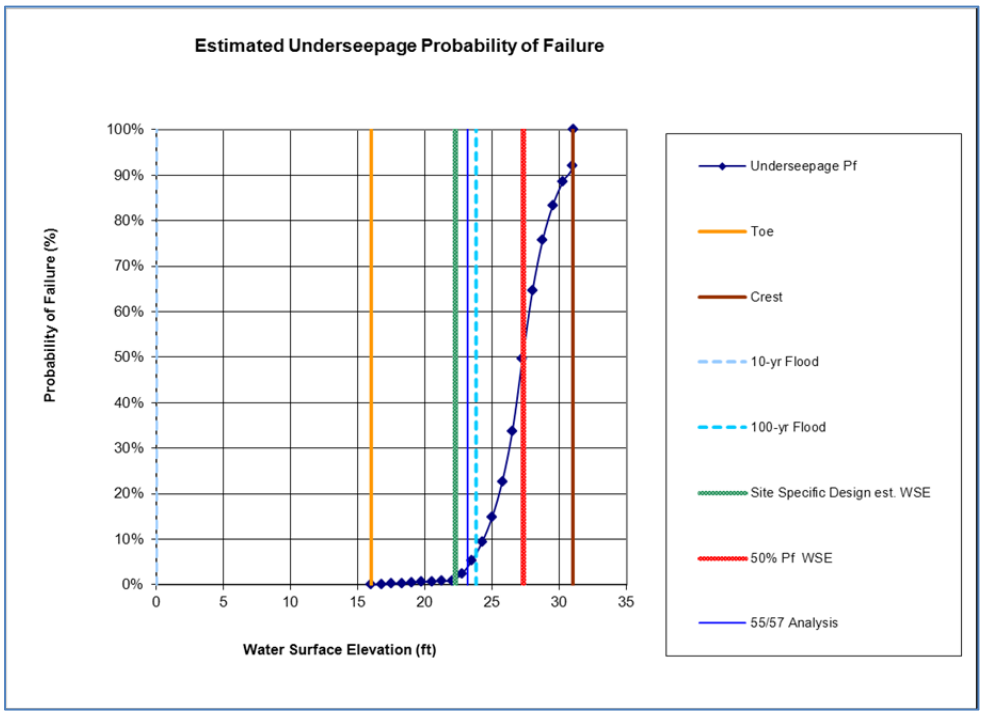
To develop ULE levee performance curves for through seepage, a failure model was developed for landside levee slopes that are composed of erodible materials, typically silts and sands. If these soils are present, then a failure assessment based on the height of seepage "breakout" above the landside toe of the levee was used. The height of seepage breakout above the landside toe was identified from the seepage analyses, which therefore relates the height of seepage breakout to the water surface elevation (flood elevation). The levee performance curve model relates the probability of failure to the height of seepage breakout where erodible materials are present – the higher the breakout, the higher the probability of failure. Figure 3-6 shows the relationship used relating breakout probability of failure versus flood elevation.

For the erosion failure mode, because a formal erosion analysis is not yet available (this work is planned for the final ULE Geotechnical Evaluation Reports), a more qualitative assessment was performed resulting in an erosion A, B, or C classification for each ULE reach for which a levee performance curve was developed. The erosion levee performance curve developed in NULE described in Section 3.3 was then used in the ULE assessment.

The final step was to mathematically combine (using Equation 1) the four failure modes into one cumulative levee performance curve for each selected cross section. Figure 3-7 provides an example cumulative ULE levee performance curve.

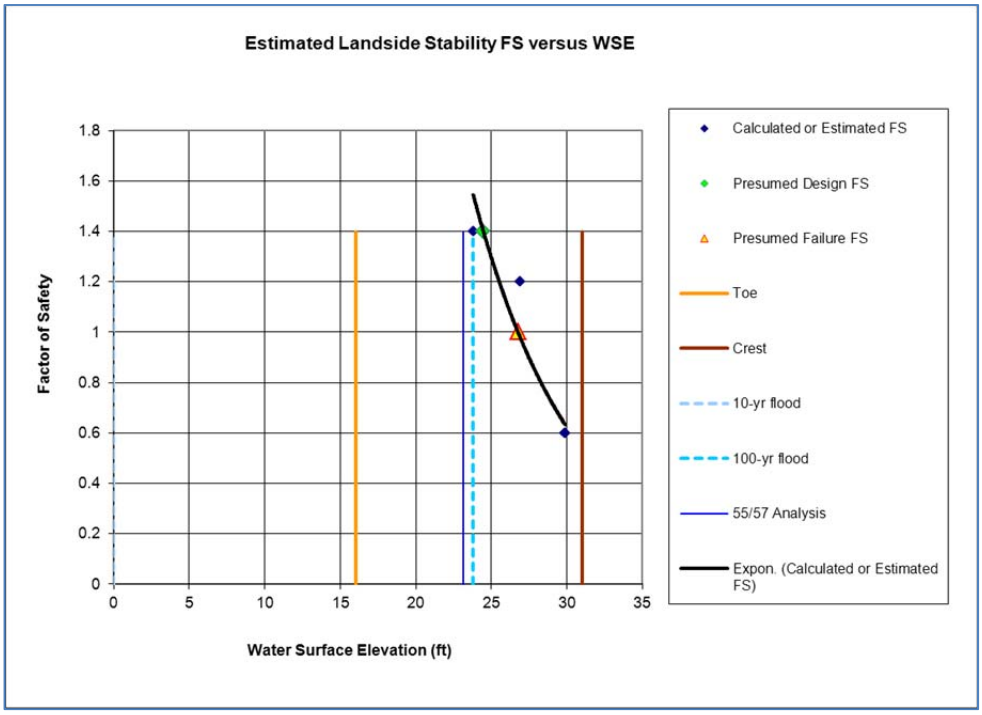
An informal review of ULE levee performance curves was provided by some ULE team Task Managers who were responsible for the ULE study area in question where each cross section is located.

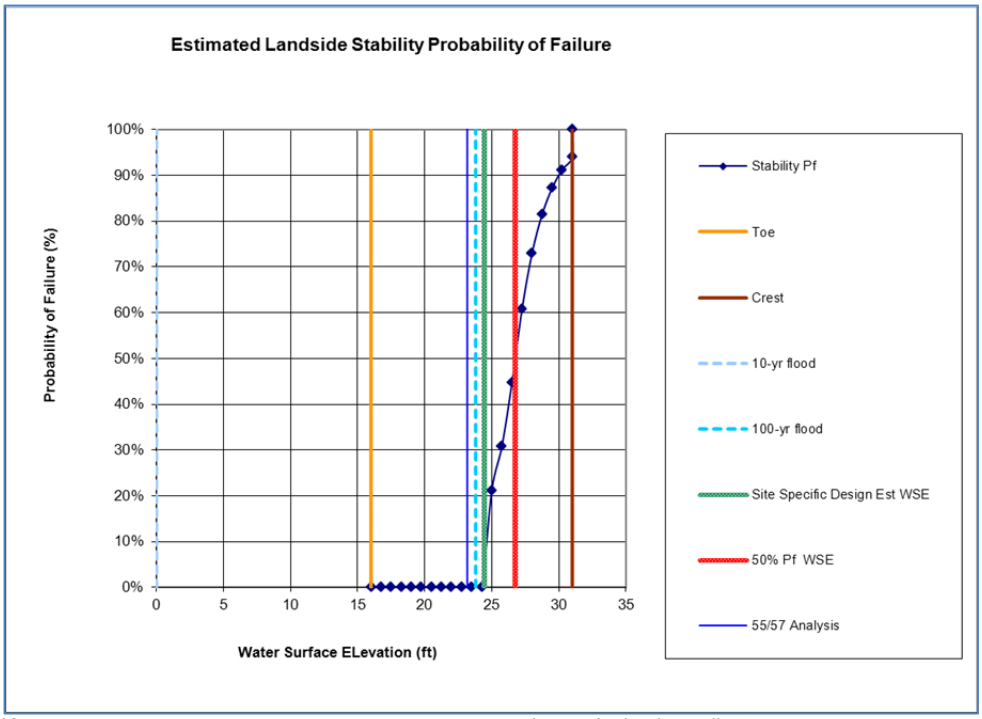




Key:
55/57 = 55/57 assessment water surface elevation
ft = feet
i = vertical exit gradient
P_f = probability of failure
WSE = water surface elevation
yr = year

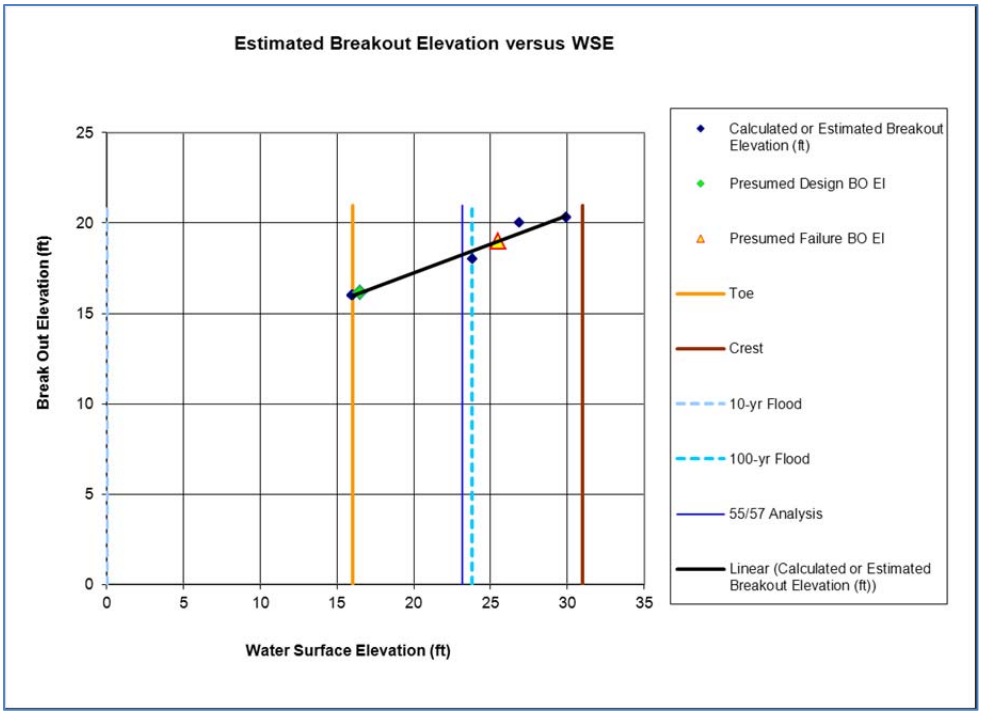
Figure 3-4. Example ULE Under-Seepage Levee Performance Curves

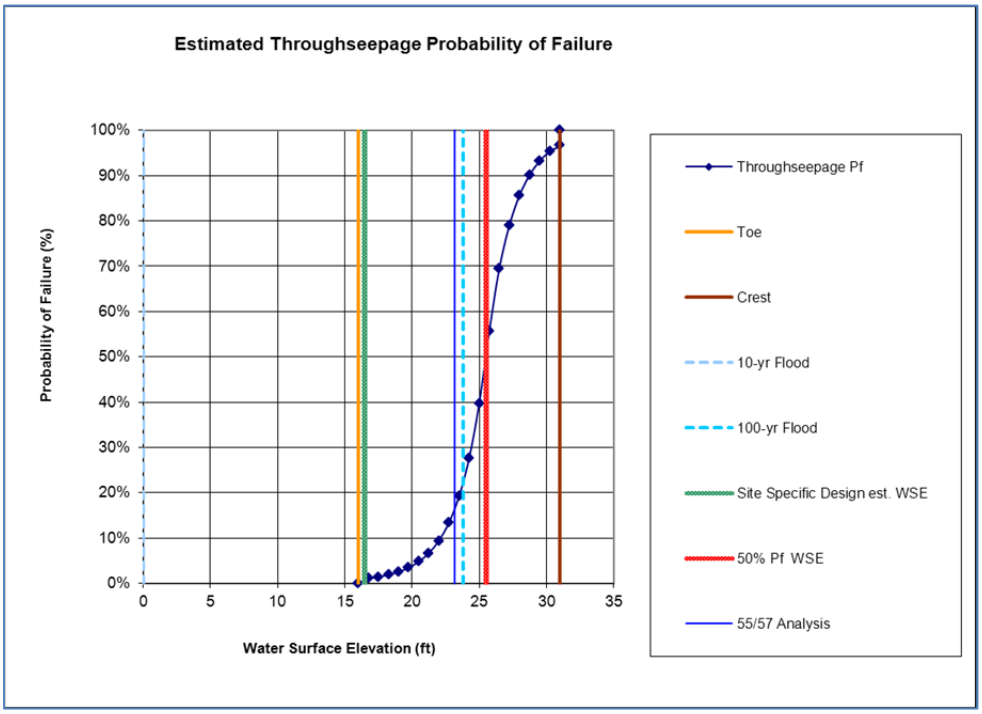




Key:
55/57 = 55/57 assessment water surface elevation
FS = factor of safety
ft = feet
i = vertical exit gradient
P_f = probability of failure
WSE = water surface elevation
yr = year

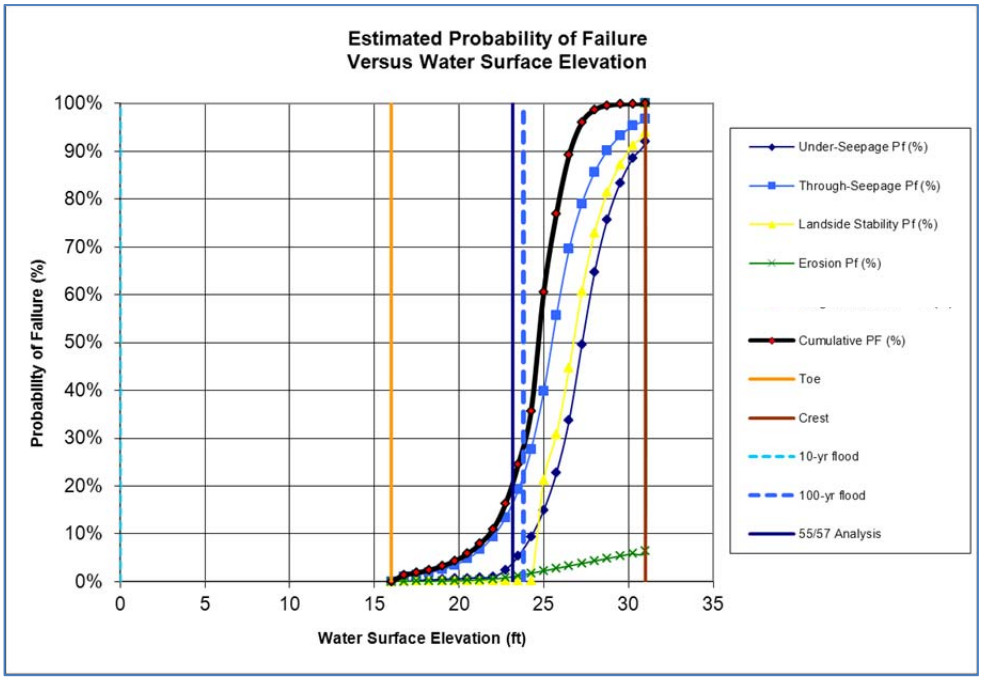
Figure 3-5. Example ULE Stability Levee Performance Curves





Key:
55/57 = 55/57 assessment water surface elevation
BO = breakout
El = failure elevation
ft = feet
i = vertical exit gradient
P_f = probability of failure
WSE = water surface elevation
yr = year

Figure 3-6. Example ULE Through-Seepage Levee Performance Curves



Key:
55/57 = 55/57 assessment water surface elevation
ft = feet
P_f = probability of failure
yr = year

Figure 3-7. Example ULE Levee Performance Curves (with failure mode and combined curves)

3.5 Anomalous Hazards

Levee performance curves for anomalous hazards were also developed. Anomalous hazards were identified in the preliminary ULE analysis and NULE GAR as isolated locations distinct from the overall levee segment for which the following apply:

- Geotechnical conditions are different from the remainder of the segment (reach).
- The current scope of levee assessment approaches used in ULE and NULE Phase 1 do not lend themselves to further detailed analyses of the hazard at these sites (e.g., analyses of structures, penetrations, encroachments).
- In many cases, the anomalous conditions are associated with observations of past poor performance.
- Available information in the area with anomalous conditions suggests that the levee may be susceptible to failing in one of the four failure

modes assessed in NULE Phase 1 (under-seepage, instability, through-seepage, or erosion).

Anomalous hazards are not related to the potential for overtopping (e.g., low spots in a levee crown at a bridge or ramp) as overtopping is not included as a failure mode in ULE and NULE.

As mentioned, additional levee performance curves were developed for anomalous hazard locations identified by the ULE teams and in the NULE GAR. Groups of anomalous hazards and suggested modifications to parent segment category ratings for NULE are listed in Table 3-2. Anomalous conditions for ULE generally followed the methods described in Section 3.4 above.

In some cases, the anomalous hazard rating and parent segment category rating are identical. The anomalous hazard will still impact the hydraulic and damage models by adding an additional potential breakout location within the segment.

Table 3-2. Anomalous Hazard Groups and Suggested Modifications to Parent Segment Category Ratings

Anomalous Hazard Group	Suggested Modifications to Rating
Erosion coincident with constructed features	Increase erosion rating to C
Poor past performance coincident with a penetration (usually through-seepage)	Increase through-seepage rating to C
Large siphon	Increase under-seepage rating to C
Site of past breach that has been repaired and has had either (1) poor performance since repair, or (2) an adjacent landside hole (e.g., scour pool, which shortens flow path)	Increase under-seepage and through-seepage ratings to C
Soft foundations resulting from buried sloughs or the like, with associated indicators of stability problems	Increase stability rating to C
Landside holes (adjacent or near to levee) associated with boils or other poor performance	Increase under-seepage rating to C
Permanent unrepaired breach	Use new topography and assign all failure modes a category of C
Significant encroachment/transition in levee geometry	Increase impacted failure mode to C
Documented geotechnical conditions at specific anomalous hazard locations	Increase other failure mode ratings

Note:
C = When water reaches assessment water surface elevation, there is a high likelihood of either levee failure or the need to floodfight to prevent levee failure.

3.6 Capabilities of HEC-FDA

The risk analysis capabilities provided by the USACE Hydrologic Engineering Center Flood Damage Assessment (HEC-FDA) program are used in the CVFPP, and as described in Attachment 8F: Flood Damage Analysis. Because the levee performance curves are important input data for the HEC-FDA model, this section briefly describes the capabilities and uses of HEC-FDA.

The HEC-FDA program can be used to perform an integrated hydrologic engineering, risk, and economic analysis during formulation and evaluation of flood risk management plans.

The use of risk analysis procedures for formulating and evaluating flood damage reduction measures is described in USACE Engineer Manual 1110-2-1619 (1996) and Engineer Regulation 1105-2-101 (2006). These documents describe how to quantify uncertainty in discharge-exceedence probability, stage-discharge relationships, and stage-damage functions and incorporate uncertainty into economic and engineering performance analyses. The program applies Monte Carlo simulation, a numerical analysis procedure that computes the expected value of damage while explicitly accounting for uncertainty in the basic parameters used to determine flood inundation damage. One of those basic parameters is the levee performance curve.

HEC-FDA assists in formulating and evaluating flood risk management plans using these procedures to calculate damage-stage-uncertainty information at damage reach index locations. Expected annual damage and flood risk are computed in the evaluation portion of the program.

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4.0 Results

This section presents the levee performance curves and describes the limitations in using these curves.

4.1 Summary

This section presents the levee performance curves developed using the techniques described above for use in systemwide SPFC hydraulic and economic damage modeling and for preparing the 2012 CVFPP. Table 4-1 contains levee performance curves for the Sacramento River Basin and Table 4-2 contains the levee performance curves for the San Joaquin River Basin.

The ULE Excel tool for developing levee performance curves should only be used on a reach-by-reach basis. The NULE Excel tool allows the user to modify certain parameters and rapidly generate a new set of levee performance curves for all NULE segments.

Table 4-1. Sacramento River Basin Levee Performance Curves

ID		SA1	SA2	SA3		SA4		SA5		SA6	
Name		Woodson Bridge East	Woodson Bridge West	Hamilton City		Capay		Butte Basin		Butte City	
Toe Elevation[3]		1	1	145.5		135.4		52.6		52.6	
AWSE				151.0		142.1		66.1		66.1	
Crest Elevation[3]				154.0		145.1		69.1		69.1	
Type of Project (ULE or NULE)				NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	2	2	145.47	0	135.38	0	52.59	0	52.59	0
				146.02	0	136.05	0	53.94	0	53.94	0
				146.57	0	136.72	0	55.29	0	55.29	0
				147.12	1	137.38	1	56.64	1	56.64	1
				147.67	1	138.05	1	57.99	1	57.99	1
				148.22	2	138.72	2	59.34	2	59.34	2
				148.77	4	139.39	4	60.69	3	60.69	3
				149.32	6	140.06	6	62.04	6	62.04	6
				149.87	10	140.72	10	63.39	9	63.39	9
				150.42	16	141.39	16	64.74	15	64.74	15
				150.97	25	142.06	25	66.09	24	66.09	24
				151.27	63	142.36	63	66.39	63	66.39	63
				151.57	81	142.66	81	66.69	80	66.69	80
				151.87	90	142.96	90	66.99	89	66.99	89
				152.17	94	143.26	94	67.29	94	67.29	94
				152.47	97	143.56	97	67.59	97	67.59	97
				152.77	98	143.86	98	67.89	98	67.89	98
				153.07	99	144.16	99	68.19	99	68.19	99
				153.37	100	144.46	100	68.49	99	68.49	99
				153.67	100	144.76	100	68.79	100	68.79	100
				153.97	100	145.06	100	69.09	100	69.09	100
				-	-	-	-	-	-	-	-

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA7		SA8		SA9		SA10		SA11	
Name		Colusa Basin North		Colusa		Colusa Basin South		Grimes		RD 1500 West	
Toe Elevation[3]		84.6		58.6		46.6		46.6		36.6	
AWSE		97.6		67.6		57.4		57.4		48.2	
Crest Elevation[3]		101.6		70.6		61.4		61.4		53.7	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	84.61	0	58.58	0	46.60	0	46.60	0	36.63	0
		85.91	0	59.48	0	47.68	0	47.68	0	37.79	0
		87.21	0	60.38	0	48.76	0	48.76	0	38.95	0
		88.51	0	61.28	0	49.84	1	49.84	1	40.11	1
		89.81	0	62.18	0	50.92	2	50.92	2	41.27	1
		91.11	0	63.08	0	52.00	3	52.00	3	42.43	2
		92.41	1	63.98	1	53.08	5	53.08	5	43.59	3
		93.71	1	64.88	1	54.16	8	54.16	8	44.75	4
		95.01	2	65.78	2	55.24	13	55.24	13	45.91	7
		96.31	3	66.68	3	56.32	20	56.32	20	47.07	12
		97.61	6	67.58	4	57.40	32	57.40	32	48.23	20
		98.01	6	67.88	5	57.80	73	57.80	73	48.78	52
		98.41	7	68.18	5	58.20	88	58.20	88	49.33	71
		98.81	8	68.48	6	58.60	94	58.60	94	49.88	83
		99.21	8	68.78	6	59.00	97	59.00	97	50.43	90
		99.61	9	69.08	7	59.40	98	59.40	98	50.98	94
		100.01	10	69.38	7	59.80	99	59.80	99	51.53	97
		100.41	10	69.68	8	60.20	100	60.20	100	52.08	98
		100.81	11	69.98	8	60.60	100	60.60	100	52.63	99
		101.21	12	70.28	9	61.00	100	61.00	100	53.18	100
		101.61	100	70.58	100	61.40	100	61.40	100	53.73	100
		-	-	-	-	-	-	-	-	-	-

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA12		SA13		SA14		SA15		SA16	
Name		Sycamore Slough		Knights Landing		Ridge Cut (North)		Ridge Cut (South)		RD 2035	
Toe Elevation[3]		33.2		30.3		29.6		15.5		13.0	
AWSE		42.2		39.3		38.6		33.5		25.0	
Crest Elevation[3]		47.2		42.3		43.6		39.5		30.0	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Breach (percent)	33.16	0	30.35	0	29.56	0	15.51	0	12.97	0
		34.06	0	31.25	0	30.46	0	17.31	0	14.17	0
		34.96	0	32.15	0	31.36	0	19.11	0	15.37	0
		35.86	0	33.05	0	32.26	0	20.91	1	16.57	1
		36.76	0	33.95	0	33.16	0	22.71	1	17.77	2
		37.66	1	34.85	1	34.06	1	24.51	2	18.97	3
		38.56	1	35.75	1	34.96	1	26.31	3	20.17	5
		39.46	2	36.65	2	35.86	2	28.11	4	21.37	8
		40.36	3	37.55	3	36.76	3	29.91	7	22.57	13
		41.26	4	38.45	4	37.66	4	31.71	12	23.77	20
		42.16	7	39.35	7	38.56	7	33.51	20	24.97	32
		42.66	9	39.64	8	39.06	9	34.11	52	25.42	73
		43.16	10	39.94	9	39.56	10	34.71	71	25.87	88
		43.66	11	40.23	10	40.06	11	35.31	83	26.32	94
		44.16	13	40.52	11	40.56	13	35.91	90	26.77	97
		44.66	14	40.82	11	41.06	14	36.51	94	27.22	98
		45.16	16	41.11	12	41.56	16	37.11	96	27.67	99
		45.66	17	41.41	13	42.06	17	37.71	98	28.12	100
		46.16	19	41.70	14	42.56	19	38.31	99	28.57	100
		46.66	20	41.99	15	43.06	20	38.91	100	29.02	100
		47.16	100	42.29	100	43.56	100	39.51	100	29.47	100
		-	-	-	-	-	-	-	-	-	-

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA17		SA18		SA20		SA21		SA22	
Name		East of Davis		Upper Honcut		Gridley		Sutter Buttes East		Live Oak	
Toe Elevation[3]		26.0		0.00		77.7		77.7		77.7	
AWSE		31.6		0.00		83.2		83.2		83.2	
Crest Elevation[3]		37.2		0.00		88.7		88.7		88.7	
Type of Project (ULE or NULE)		ULE		NULE		ULE		ULE		ULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	25.97	0	96.57	0	77.73	0	77.73	0	77.73	0
		26.53	0	97.00	0	78.28	17	78.28	17	78.28	17
		27.09	0	97.42	0	78.83	24	78.83	24	78.83	24
		27.65	0	97.84	1	79.38	33	79.38	33	79.38	33
		28.22	1	98.26	2	79.93	44	79.93	44	79.93	44
		28.78	1	98.68	3	80.48	57	80.48	57	80.48	57
		29.34	1	99.11	4	81.03	70	81.03	70	81.03	70
		29.90	1	99.53	7	81.58	82	81.58	82	81.58	82
		30.47	1	99.95	12	82.13	90	82.13	90	82.13	90
		31.03	1	100.37	19	82.68	95	82.68	95	82.68	95
		31.59	1	100.79	30	83.23	97	83.23	97	83.23	97
		32.15	2	101.09	74	83.78	99	83.78	99	83.78	99
		32.72	3	101.39	91	84.33	99	84.33	99	84.33	99
		33.28	4	101.69	97	84.88	100	84.88	100	84.88	100
		33.84	6	101.99	99	85.43	100	85.43	100	85.43	100
		34.40	9	102.29	100	85.98	100	85.98	100	85.98	100
		34.97	12	102.59	100	86.53	100	86.53	100	86.53	100
		35.53	16	102.89	100	87.08	100	87.08	100	87.08	100
		36.09	21	103.19	100	87.63	100	87.63	100	87.63	100
		36.65	28	103.49	100	88.18	100	88.18	100	88.18	100
		37.22	35	103.79	100	88.73	100	88.73	100	88.73	100
		37.23	100			88.74	100	88.74	100	88.74	100

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA23		SA24		SA25		SA26		SA27	
Name		Lower Honcut		Levee Dist. #1		Yuba City		Marysville		Linda-Olivehurst	
Toe Elevation[3]		68.7		49.2		63.7		65.7		67.7	
AWSE		81.7		53.7		74.0		77.7		75.2	
Crest Elevation[3]		85.7		56.7		84.2		89.7		82.7	
Type of Project (ULE or NULE)		NULE		NULE		ULE		ULE		ULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	68.73	0	49.16	0	63.72	0	65.73	0	67.73	0
		70.03	0	49.61	0	64.74	0	66.93	1	68.48	0
		71.33	0	50.06	0	65.77	1	68.13	1	69.23	0
		72.63	1	50.51	0	66.79	1	69.33	2	69.98	0
		73.93	2	50.96	0	67.82	1	70.53	2	70.73	0
		75.23	3	51.41	0	68.84	3	71.73	2	71.48	0
		76.53	4	51.86	1	69.87	11	72.93	2	72.23	0
		77.83	7	52.31	1	70.89	22	74.13	2	72.98	0
		79.13	12	52.76	2	71.92	55	75.33	4	73.73	0
		80.43	19	53.21	3	72.94	77	76.53	7	74.48	0
		81.73	30	53.66	4	73.97	91	77.73	12	75.23	0
		82.13	74	53.96	5	74.99	97	78.93	21	75.98	0
		82.53	91	54.26	6	76.02	99	80.13	36	76.73	0
		82.93	97	54.56	7	77.04	100	81.33	60	77.48	1
		83.33	99	54.86	8	78.07	100	82.53	81	78.23	1
		83.73	100	55.16	10	79.09	100	83.73	92	78.98	1
		84.13	100	55.46	11	80.12	100	84.93	97	79.73	1
		84.53	100	55.76	12	81.14	100	86.13	99	80.48	2
		84.93	100	56.06	13	82.17	100	87.33	100	81.23	2
		85.33	100	56.36	14	83.19	100	88.53	100	81.98	2
		85.73	100	56.66	(100)	84.22	100	89.73	100	82.73	3
		-	-	-	-	84.23	100	89.74	100	82.74	100

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA28		SA29		SA30		SA32		SA33	
Name		RD 384		Best Slough		RD 1001		RD 70-1660		Meridian	
Toe Elevation[3]		44.7		70.3		35.7		42.6		42.6	
AWSE		54.7		77.8		55.7		57.2		57.2	
Crest Elevation[3]		64.7		80.8		62.7		61.8		61.8	
Type of Project (UULE or NULE)		ULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	44.70	0	70.29	0	35.69	0	42.60	0	42.60	0
		45.70	0	71.04	0	37.69	0	44.07	0	44.07	0
		46.70	0	71.79	0	39.69	1	45.53	0	45.53	0
		47.70	0	72.54	1	41.69	1	47.00	1	47.00	1
		48.70	0	73.29	2	43.69	2	48.46	2	48.46	2
		49.70	0	74.04	3	45.69	4	49.93	3	49.93	3
		50.70	0	74.79	5	47.69	6	51.39	4	51.39	4
		51.70	1	75.54	7	49.69	11	52.86	7	52.86	7
		52.70	1	76.29	12	51.69	17	54.32	12	54.32	12
		53.70	1	77.04	20	53.69	27	55.79	19	55.79	19
		54.70	1	77.79	31	55.69	42	57.25	30	57.25	30
		55.70	1	78.09	72	56.39	86	57.71	74	57.71	74
		56.70	1	78.39	87	57.09	96	58.17	91	58.17	91
		57.70	1	78.69	94	57.79	99	58.63	97	58.63	97
		58.70	1	78.99	97	58.49	100	59.08	99	59.08	99
		59.70	1	79.29	98	59.19	100	59.54	100	59.54	100
		60.70	2	79.59	99	59.89	100	60.00	100	60.00	100
		61.70	2	79.89	99	60.59	100	60.46	100	60.46	100
		62.70	2	80.19	100	61.29	100	60.91	100	60.91	100
		63.70	3	80.49	100	61.99	100	61.37	100	61.37	100
		64.70	4	80.79	100	62.69	100	61.83	100	61.83	100
		64.71	100	-	-	-	-	-	-	-	-

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA34		SA35		SA36		SA37		SA38	
Name		RD 1500 East		Elkhorn		Natomas		Rio Linda		West Sacramento	
Toe Elevation[3]		18.6		8.5		23.3		19.5		24.5	
AWSE		42.8		28.0		32.9		30.4		32.5	
Crest Elevation[3]		48.1		31.5		38.9		41.4		40.5	
Type of Project (ULE or NULE)		NULE		NULE		NULE		ULE		ULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	18.59	0	8.47	0	23.26	0	19.47	0	24.46	0
		21.01	0	10.42	0	24.23	0	20.56	0	25.26	0
		23.43	0	12.37	0	25.20	0	21.66	0	26.06	0
		25.85	1	14.32	1	26.16	1	22.75	0	26.86	0
		28.27	1	16.27	2	27.13	1	23.85	0	27.66	1
		30.69	2	18.22	3	28.10	2	24.94	0	28.46	1
		33.11	3	20.17	4	29.07	3	26.04	0	29.26	1
		35.53	5	22.12	7	30.04	4	27.13	0	30.06	1
		37.95	8	24.07	12	31.00	7	28.23	0	30.86	1
		40.37	13	26.02	19	31.97	12	29.32	0	31.66	2
		42.79	21	27.97	30	32.94	20	30.42	0	32.46	2
		43.32	52	28.32	74	33.54	52	31.51	0	33.26	2
		43.85	71	28.67	91	34.14	71	32.61	1	34.06	8
		44.38	83	29.02	97	34.75	83	33.70	1	34.86	11
		44.91	90	29.37	99	35.35	90	34.80	1	35.66	15
		45.44	94	29.72	100	35.95	94	35.89	1	36.46	21
		45.97	96	30.07	100	36.55	97	36.99	2	37.26	29
		46.50	98	30.42	100	37.15	98	38.08	2	38.06	41
		47.03	99	30.77	100	37.76	99	39.18	3	38.86	57
		47.56	100	31.12	100	38.36	100	40.27	3	39.66	72
		48.09	100	31.47	100	38.96	100	41.37	3	40.46	81
		-	-	-	-	-	-	41.38	100	40.47	100

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA39		SA40		SA41		SA42		SA43	
Name		RD 900		Sacramento North		RD 302		RD 999		Clarksburg	
Toe Elevation[3]		17.5		25.5		10.5		12.5		11.5	
AWSE		27.5		34.0		26.5		27.5		23.0	
Crest Elevation[3]		37.5		42.5		30.5		31.5		28.5	
Type of Project (ULE or NULE)		ULE		ULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	17.46	0	25.46	0	10.50	0	12.49	0	11.53	0
		18.46	1	26.31	0	12.10	0	13.99	0	12.68	0
		19.46	1	27.16	0	13.70	0	15.49	0	13.83	0
		20.46	2	28.01	0	15.30	1	16.99	1	14.98	0
		21.46	2	28.86	0	16.90	1	18.49	1	16.13	0
		22.46	2	29.71	0	18.50	2	19.99	2	17.28	0
		23.46	3	30.56	1	20.10	4	21.49	3	18.43	1
		24.46	4	31.41	1	21.70	6	22.99	4	19.58	1
		25.46	6	32.26	1	23.30	10	24.49	7	20.73	2
		26.46	10	33.11	1	24.90	16	25.99	12	21.88	3
		27.46	18	33.96	5	26.50	25	27.49	19	23.03	5
		28.46	28	34.81	7	26.90	63	27.89	46	23.58	6
		29.46	43	35.66	10	27.30	81	28.29	60	24.13	7
		30.46	63	36.51	14	27.70	90	28.69	67	24.68	8
		31.46	82	37.36	19	28.10	94	29.09	72	25.23	9
		32.46	93	38.21	27	28.50	97	29.49	74	25.78	9
		33.46	98	39.06	39	28.90	98	29.89	76	26.33	10
		34.46	100	39.91	55	29.30	99	30.29	77	26.88	11
		35.46	100	40.76	70	29.70	99	30.69	77	27.43	12
		36.46	100	41.61	81	30.10	100	31.09	78	27.98	13
		37.46	100	42.46	88	30.50	100	31.49	100	28.53	100
		37.47	100	42.47	100	-	-	-	-	-	-

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA44		SA45		SA46		SA47		SA48	
Name		Stone Lake		Hood		Merritt Island		RD 551		Courtland	
Toe Elevation[3]		15.6		15.6		10.5		5.6		5.6	
AWSE		20.5		20.6		21.1		21.6		21.6	
Crest Elevation[3]		26.4		26.4		23.6		25.6		25.6	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	15.56	0	15.56	0	10.55	0	5.56	0	5.56	0
		16.06	0	16.06	0	11.60	0	7.16	0	7.16	0
		16.56	0	16.56	0	12.65	0	8.76	0	8.76	0
		17.06	1	17.06	1	13.70	1	10.36	1	10.36	1
		17.56	2	17.56	2	14.75	1	11.96	1	11.96	1
		18.05	3	18.05	3	15.80	2	13.56	2	13.56	2
		18.55	5	18.55	5	16.85	4	15.16	4	15.16	4
		19.05	8	19.05	8	17.90	6	16.76	6	16.76	6
		19.55	13	19.55	13	18.95	10	18.36	10	18.36	10
		20.05	20	20.05	20	20.00	16	19.96	16	19.96	16
		20.55	32	20.55	32	21.05	25	21.56	25	21.56	25
		21.13	76	21.13	76	21.30	63	21.96	63	21.96	63
		21.72	91	21.72	91	21.55	81	22.36	81	22.36	81
		22.30	97	22.30	97	21.80	90	22.76	90	22.76	90
		22.88	99	22.88	99	22.05	94	23.16	94	23.16	94
		23.46	100	23.46	100	22.30	97	23.56	97	23.56	97
		24.04	100	24.04	100	22.55	98	23.96	98	23.96	98
		24.62	100	24.62	100	22.80	99	24.36	99	24.36	99
		25.20	100	25.20	100	23.05	99	24.76	99	24.76	99
		25.79	100	25.79	100	23.30	100	25.16	100	25.16	100
		26.37	100	26.37	100	23.55	100	25.56	100	25.56	100
		-	-	-	-	-	-	-	-	-	-

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA49		SA50		SA51		SA52		SA53	
Name		Sutter Island		Grand Island		Locke		Walnut Grove		Tyler Island	
Toe Elevation[3]		4.6		-0.4		7.6		9.6		-2.4	
AWSE		16.1		12.4		15.0		14.5		9.6	
Crest Elevation[3]		22.6		17.8		20.1		22.6		11.6	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	4.58	0	-0.42	0	7.61	0	9.61	0	-2.38	0
		5.73	0	0.87	0	8.35	0	10.10	0	-1.18	0
		6.88	0	2.15	1	9.09	0	10.59	0	0.02	0
		8.03	1	3.44	1	9.83	0	11.08	0	1.22	1
		9.18	2	4.73	2	10.57	0	11.57	0	2.42	2
		10.33	3	6.01	3	11.31	0	12.06	0	3.62	3
		11.48	5	7.30	6	12.05	0	12.55	0	4.82	5
		12.63	8	8.59	9	12.79	0	13.04	0	6.02	8
		13.78	13	9.87	15	13.53	1	13.53	1	7.22	13
		14.93	20	11.16	24	14.27	1	14.02	1	8.42	20
		16.08	32	12.45	37	15.01	2	14.51	2	9.62	32
		16.73	73	12.98	81	15.52	3	15.32	3	9.82	75
		17.38	88	13.52	94	16.03	3	16.13	5	10.02	91
		18.03	94	14.06	98	16.54	4	16.94	6	10.22	97
		18.68	97	14.60	99	17.05	4	17.75	7	10.42	99
		19.33	98	15.13	100	17.56	5	18.56	8	10.62	100
		19.98	99	15.67	100	18.07	5	19.37	9	10.82	100
		20.63	100	16.21	100	18.58	6	20.18	11	11.02	100
		21.28	100	16.75	100	19.09	6	20.99	12	11.22	100
		21.93	100	17.28	100	19.60	7	21.80	13	11.42	100
		22.58	100	17.82	100	20.11	100	22.61	100	11.62	100
		-	-	-	-	-	-	-	-	-	-

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA54		SA55		SA56		SA57		SA58	
Name		Andrus Island		Ryer Island		Prospect Island		Twitchell Island		Sherman Island	
Toe Elevation[3]		-2.4		-1.4		0.00		-1.4		-12.4	
AWSE		11.6		11.8		0.00		9.1		8.6	
Crest Elevation[3]		13.6		20.8		0.00		13.6		10.6	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	-2.39	0	-1.42	0			-1.40	0	-12.43	0
		-0.99	0	-0.10	0			-0.35	0	-10.33	0
		0.41	0	1.22	0			0.70	0	-8.23	0
		1.81	0	2.55	1			1.75	0	-6.13	1
		3.21	0	3.87	2			2.80	0	-4.03	2
		4.61	1	5.20	3			3.85	1	-1.93	3
		6.01	1	6.52	5			4.90	1	0.17	5
		7.41	2	7.84	8			5.95	2	2.27	8
		8.81	3	9.17	13			7.00	3	4.37	13
		10.21	4	10.49	20			8.05	5	6.47	20
		11.61	7	11.82	32			9.10	8	8.57	32
		11.81	7	12.72	75			9.55	9	8.77	75
		12.01	8	13.62	91			10.00	10	8.97	91
		12.21	8	14.52	97			10.45	12	9.17	97
		12.41	9	15.42	99			10.90	13	9.37	99
		12.61	9	16.33	100			11.35	14	9.57	100
		12.81	9	17.23	100			11.80	15	9.77	100
		13.01	10	18.13	100			12.25	16	9.97	100
		13.21	10	19.03	100			12.70	18	10.17	100
		13.41	10	19.94	100			13.15	19	10.37	100
		13.61	100	20.84	100		100	13.60	100	10.57	100
		-	-	-	-			-	-	-	-

Table 4-1. Sacramento River Basin Levee Performance Curves (contd.)

ID		SA59		SA60		SA61		SA62		SA63	
Name		Moore		Cache Slough		Hastings		Lindsey Slough		Sacramento South	
Toe Elevation[3]		2.0		2.5		1.5		3.5		27.5	
AWSE		15.5		15.5		14.3		14.3		33.3	
Crest Elevation[3]		19.5		18.0		16.7		18.5		39.1	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		ULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	2.04	0	2.52	0	1.53	0	3.53	0	27.46	0
		3.39	0	3.82	0	2.81	0	4.60	0	28.04	0
		4.74	0	5.12	0	4.09	0	5.68	1	28.62	0
		6.09	1	6.42	0	5.36	1	6.75	1	29.20	0
		7.44	1	7.72	0	6.64	1	7.83	2	29.78	1
		8.79	2	9.02	0	7.92	2	8.90	4	30.36	1
		10.14	4	10.32	0	9.19	3	9.98	6	30.94	1
		11.49	6	11.62	1	10.47	5	11.06	11	31.52	1
		12.84	10	12.92	1	11.75	8	12.13	17	32.10	2
		14.19	16	14.22	2	13.02	13	13.21	27	32.68	2
		15.54	25	15.52	3	14.30	21	14.28	42	33.26	3
		15.94	63	15.77	3	14.54	52	14.71	86	33.84	4
		16.34	81	16.02	3	14.79	71	15.13	96	34.42	5
		16.74	90	16.27	3	15.03	83	15.55	99	35.00	6
		17.14	94	16.52	4	15.27	90	15.97	100	35.58	7
		17.54	97	16.77	4	15.52	94	16.40	100	36.16	8
		17.94	98	17.02	4	15.76	96	16.82	100	36.74	8
		18.34	99	17.27	4	16.00	98	17.24	100	37.32	9
		18.74	99	17.52	4	16.25	99	17.66	100	37.90	10
		19.14	100	17.77	5	16.49	100	18.09	100	38.48	12
		19.54	100	18.02	100	16.73	100	18.51	100	39.06	13
		-	-	-	-	-	-	-	-	39.07	100

Notes:
[1] No State-federal project levees found within the impact area
[2] Assume overbank flow
[3] Elevations in feet, NGVD29

Key:
- = not applicable
AWSE = Assessment Water Surface Elevation
NGVD29 = National Geodetic vertical Datum of 1929
NULE = Non-Urban Levee Evaluations
RD = Reclamation District
ULE = Urban Levee Evaluations

Table 4-2. San Joaquin River Basin Levee Performance Curves

ID		SJ1	SJ2		SJ3		SJ4		SJ5	
Name		Fresno	Fresno Slough East		Fresno Slough West		Mendota		Chowchilla Bypass	
Toe Elevation[3]		1	159.6		150.9		151.6		157.8	
AWSE			163.8		155.9		153.6		166.8	
Crest Elevation[3]			166.8		158.9		156.6		170.8	
Type of Project (ULE or NULE)			NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	2	159.59	0	150.90	0	151.61	0	157.82	0
			160.01	0	151.40	0	151.81	0	158.72	0
			160.43	0	151.90	0	152.01	0	159.62	0
			160.85	1	152.40	0	152.21	0	160.52	1
			161.27	2	152.90	1	152.41	0	161.42	2
			161.69	3	153.40	1	152.61	0	162.32	3
			162.11	5	153.90	3	152.81	0	163.22	4
			162.53	7	154.40	4	153.01	0	164.12	7
			162.95	12	154.90	7	153.21	1	165.02	12
			163.37	20	155.40	11	153.41	1	165.92	19
			163.79	31	155.90	18	153.61	2	166.82	30
			164.09	75	156.20	46	153.91	3	167.22	74
			164.39	91	156.50	60	154.21	4	167.62	91
			164.69	97	156.80	67	154.51	5	168.02	97
			164.99	99	157.10	72	154.81	7	168.42	99
			165.29	100	157.40	74	155.11	8	168.82	100
			165.59	100	157.70	76	155.41	9	169.22	100
			165.89	100	158.00	77	155.71	10	169.62	100
			166.19	100	158.30	78	156.01	11	170.02	100
			166.49	100	158.60	79	156.31	12	170.42	100
			166.79	100	158.90	100	156.61	100	170.82	100

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		SJ6		SJ7	SJ8		SJ9		SJ10	
Name		Lone Willow Slough		Mendota North	Firebaugh		Salt Slough		Dos Palos	
Toe Elevation[3]		152.2		1	141.2		114.8		114.8	
AWSE		157.4			143.1		117.5		117.5	
Crest Elevation[3]		160.4			146.1		120.5		120.5	
Type of Project (ULE or NULE)		NULE			NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	152.22	0	2	141.17	0	114.83	0	114.83	0
		152.74	0		141.37	0	115.10	0	115.10	0
		153.26	0		141.56	0	115.37	0	115.37	0
		153.78	0		141.76	0	115.64	0	115.64	0
		154.30	1		141.95	0	115.91	0	115.91	0
		154.82	1		142.14	0	116.18	0	116.18	0
		155.34	2		142.34	0	116.45	0	116.45	0
		155.86	3		142.53	1	116.72	1	116.72	1
		156.38	4		142.73	1	116.99	1	116.99	1
		156.90	7		142.92	2	117.26	2	117.26	2
		157.42	12		143.11	3	117.53	3	117.53	3
		157.72	28		143.41	4	117.83	4	117.83	4
		158.02	39		143.71	6	118.13	5	118.13	5
		158.32	45		144.01	8	118.43	6	118.43	6
		158.62	49		144.31	9	118.73	7	118.73	7
		158.92	52		144.61	11	119.03	9	119.03	9
		159.22	54		144.91	12	119.33	10	119.33	10
		159.52	55		145.21	14	119.63	11	119.63	11
		159.82	56		145.51	15	119.93	12	119.93	12
		160.12	57		145.81	17	120.23	13	120.23	13
		160.42	100		146.11	100	120.53	100	120.53	100

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		SJ11		SJ12		SJ13		SJ14		SJ15	
Name		Fresno River		Berenda Slough		Ash Slough		Sandy Mush		Turner Island	
Toe Elevation[3]		184.1		148.2		139.1		98.6		96.5	
AWSE		189.2		150.9		142.6		105.8		105.7	
Crest Elevation[3]		192.2		153.9		145.6		109.8		109.7	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	184.14	0	148.17	0	139.06	0	98.61	0	96.49	0
		184.65	0	148.44	0	139.41	0	99.33	0	97.41	0
		185.16	0	148.71	0	139.76	0	100.05	0	98.33	0
		185.67	0	148.98	0	140.11	0	100.77	1	99.25	1
		186.18	1	149.25	0	140.46	0	101.49	2	100.17	2
		186.69	1	149.52	0	140.81	0	102.21	3	101.09	3
		187.20	3	149.79	0	141.16	1	102.93	4	102.01	4
		187.71	4	150.06	0	141.51	1	103.65	7	102.93	7
		188.22	7	150.33	1	141.86	2	104.37	12	103.85	12
		188.73	11	150.60	1	142.21	3	105.09	19	104.77	19
		189.24	19	150.87	2	142.56	5	105.81	30	105.69	30
		189.54	51	151.17	3	142.86	7	106.21	74	106.09	74
		189.84	71	151.47	4	143.16	8	106.61	91	106.49	91
		190.14	82	151.77	5	143.46	10	107.01	97	106.89	97
		190.44	89	152.07	5	143.76	11	107.41	99	107.29	99
		190.74	94	152.37	6	144.06	13	107.81	100	107.69	100
		191.04	96	152.67	7	144.36	14	108.21	100	108.09	100
		191.34	98	152.97	8	144.66	16	108.61	100	108.49	100
		191.64	99	153.27	9	144.96	17	109.01	100	108.89	100
		191.94	100	153.57	10	145.26	19	109.41	100	109.29	100
		192.24	100	153.87	100	145.56	100	109.81	100	109.69	100

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		SJ16		SJ17		SJ18		SJ19		SJ20
Name		Bear Creek		Deep Slough		West Bear Creek		Fremont Ford		Merced River
Toe Elevation[3]		84.4		84.2		81.3		64.0		1
AWSE		89.1		89.9		85.8		70.9		
Crest Elevation[3]		92.1		92.9		88.8		73.5		
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	84.39	0	84.20	0	81.27	0	63.97	0	2
		84.86	0	84.77	0	81.72	0	64.66	0	
		85.33	0	85.34	0	82.17	0	65.35	0	
		85.80	0	85.91	0	82.62	0	66.04	0	
		86.27	0	86.48	1	83.07	1	66.73	0	
		86.74	0	87.05	1	83.52	1	67.42	0	
		87.21	0	87.62	3	83.97	3	68.11	1	
		87.68	1	88.19	4	84.42	4	68.80	1	
		88.15	1	88.76	7	84.87	7	69.49	2	
		88.62	2	89.33	11	85.32	11	70.18	3	
		89.09	3	89.90	19	85.77	19	70.87	4	
		89.39	4	90.20	51	86.07	51	71.13	5	
		89.69	5	90.50	71	86.37	71	71.39	5	
		89.99	6	90.80	82	86.67	82	71.65	6	
		90.29	7	91.10	89	86.97	90	71.91	7	
		90.59	8	91.40	94	87.27	94	72.17	7	
		90.89	9	91.70	96	87.57	96	72.43	8	
		91.19	9	92.00	98	87.87	98	72.69	8	
		91.49	10	92.30	99	88.17	99	72.95	9	
		91.79	11	92.60	100	88.47	100	73.21	10	
		92.09	100	92.90	100	88.77	100	73.47	100	

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		SJ21		SJ22		SJ23		SJ24		SJ25
Name		Merced River North		Orestimba		Tuolumne South		Tuolumne River		Modesto
Toe Elevation[3]		42.0		48.6		33.0		40.1		1
AWSE		52.6		57.0		38.6		47.0		
Crest Elevation[3]		54.9		57.0		38.6		50.4		
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	41.98	0	48.59	0	33.01	0	40.09	0	2
		43.04	0	49.43	0	33.57	0	40.79	0	
		44.10	0	50.27	0	34.13	0	41.48	1	
		45.16	1	51.11	1	34.69	1	42.17	1	
		46.22	2	51.95	2	35.25	2	42.86	2	
		47.28	3	52.79	3	35.81	3	43.56	4	
		48.34	4	53.63	4	36.37	4	44.25	6	
		49.40	7	54.47	7	36.93	7	44.94	10	
		50.46	12	55.31	12	37.49	12	45.64	17	
		51.52	19	56.15	19	38.05	19	46.33	27	
		52.58	30	56.99	30	38.61	31	47.02	41	
		52.81	74	57.29	74	38.61	75	47.36	87	
		53.04	91	57.59	91	38.61	91	47.70	97	
		53.27	97	57.89	97	38.61	97	48.05	99	
		53.50	99	58.19	99	38.61	99	48.39	100	
		53.73	100	58.49	100	38.61	100	48.73	100	
		53.96	100	58.79	100	38.61	100	49.07	100	
		54.19	100	59.09	100	38.61	100	49.41	100	
		54.42	100	59.39	100	38.61	100	49.75	100	
		54.65	100	59.69	100	38.61	100	50.09	100	
		54.88	100	59.99	100	38.61	100	50.43	100	

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		SJ26		SJ27		SJ28		SJ29		SJ30	
Name		3 Amigos		Stanislaus South		Stanislaus North		Banta Carbona		Paradise Cut	
Toe Elevation[3]		28.4		23.5		27.9		19.5		0.6	
AWSE		38.7		36.6		35.5		28.4		14.7	
Crest Elevation[3]		41.7		40.0		38.5		32.1		22.4	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	28.41	0	23.52	0	27.93	0	19.49	0	0.58	0
		29.44	0	24.83	0	28.69	0	20.39	0	1.99	0
		30.47	0	26.14	1	29.45	1	21.28	0	3.40	1
		31.50	1	27.45	1	30.21	1	22.17	1	4.81	1
		32.53	2	28.76	2	30.97	2	23.07	2	6.22	2
		33.56	3	30.07	4	31.73	4	23.96	3	7.63	4
		34.59	5	31.38	6	32.49	6	24.86	5	9.04	6
		35.62	7	32.69	10	33.25	10	25.75	7	10.45	10
		36.65	12	34.00	17	34.01	17	26.64	12	11.86	17
		37.68	20	35.31	27	34.77	27	27.54	20	13.27	27
		38.71	31	36.62	41	35.53	41	28.43	31	14.68	41
		39.01	75	36.96	87	35.83	87	28.79	75	15.45	87
		39.31	91	37.30	97	36.13	97	29.16	91	16.22	97
		39.61	97	37.64	99	36.43	99	29.52	97	16.99	99
		39.91	99	37.98	100	36.73	100	29.88	99	17.76	100
		40.21	100	38.32	100	37.03	100	30.25	100	18.53	100
		40.51	100	38.66	100	37.33	100	30.61	100	19.30	100
		40.81	100	39.00	100	37.63	100	30.97	100	20.07	100
		41.11	100	39.34	100	37.93	100	31.34	100	20.84	100
		41.41	100	39.68	100	38.23	100	31.70	100	21.61	100
		41.71	100	40.02	100	38.53	100	32.06	100	22.38	100

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		SJ31		SJ32		SJ33		SJ34		SJ35	
Name		Stewart Tract		East Lathrop		Lathrop/ Sharpe		French Camp		Moss Tract	
Toe Elevation[3]		13.7		16.6		12.7		10.7		4.4	
AWSE		23.4		22.8		18.2		17.0		11.7	
Crest Elevation[3]		28.8		30.9		29.0		26.0		19.4	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	13.75	0	16.65	0	12.67	0	10.69	0	4.41	0
		14.72	0	17.27	0	13.22	0	11.32	0	5.14	0
		15.69	0	17.89	0	13.77	0	11.95	0	5.87	0
		16.66	1	18.51	0	14.32	0	12.58	0	6.60	0
		17.63	2	19.13	0	14.87	0	13.21	0	7.33	1
		18.60	3	19.75	0	15.42	0	13.84	0	8.06	1
		19.57	5	20.37	1	15.97	1	14.47	1	8.79	3
		20.54	7	20.99	1	16.52	1	15.10	1	9.52	4
		21.51	12	21.61	2	17.07	2	15.73	2	10.25	7
		22.48	20	22.23	3	17.62	3	16.36	3	10.98	11
		23.45	31	22.85	5	18.17	5	16.99	5	11.71	19
		23.99	75	23.66	7	19.25	9	17.89	8	12.48	51
		24.53	91	24.47	10	20.33	12	18.79	10	13.25	71
		25.07	97	25.28	12	21.41	16	19.69	13	14.02	83
		25.61	99	26.09	15	22.49	19	20.59	15	14.79	90
		26.15	100	26.90	17	23.57	22	21.49	18	15.56	94
		26.69	100	27.71	19	24.65	26	22.39	20	16.33	97
		27.23	100	28.52	21	25.73	29	23.29	23	17.10	98
		27.77	100	29.33	24	26.81	32	24.19	25	17.87	99
		28.31	100	30.14	26	27.89	35	25.09	27	18.64	100
		28.85	100	30.95	100	28.97	100	25.99	100	19.41	100

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		SJ36		SJ37		SJ38		SJ39		SJ40	
Name		Roberts Island		Rough and Ready Island		Drexler Tract		Union Island		SE Union Island	
Toe Elevation[3]		4.6		2.7		-2.8		8.6		5.3	
AWSE		17.0		8.6		7.7		13.5		13.4	
Crest Elevation[3]		26.1		13.9		8.4		23.4		19.3	
Type of Project (ULE or NULE)		NULE		NULE		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	4.58	0	2.75	0	-2.85	0	8.61	0	5.28	0
		5.82	0	3.34	0	-1.80	0	9.10	0	6.09	0
		7.06	0	3.93	0	-0.74	0	9.59	0	6.90	1
		8.30	1	4.52	0	0.31	0	10.08	1	7.71	1
		9.54	1	5.11	0	1.37	1	10.57	2	8.52	2
		10.78	2	5.70	0	2.43	1	11.06	3	9.33	4
		12.02	3	6.29	0	3.48	2	11.55	5	10.14	6
		13.26	4	6.88	0	4.54	3	12.04	7	10.95	10
		14.50	7	7.47	1	5.59	5	12.53	12	11.76	17
		15.74	12	8.06	1	6.65	8	13.02	20	12.57	27
		16.98	20	8.65	2	7.71	13	13.51	31	13.38	41
		17.89	52	9.18	3	7.78	29	14.50	75	13.97	87
		18.80	71	9.71	3	7.85	39	15.49	91	14.56	97
		19.71	83	10.24	4	7.92	45	16.48	97	15.15	99
		20.62	90	10.77	5	7.99	48	17.47	99	15.74	100
		21.53	94	11.30	5	8.06	50	18.46	100	16.33	100
		22.44	97	11.83	6	8.13	52	19.45	100	16.92	100
		23.35	98	12.36	7	8.20	53	20.44	100	17.51	100
		24.26	99	12.89	7	8.27	53	21.43	100	18.10	100
		25.17	100	13.42	8	8.34	54	22.42	100	18.69	100
		26.08	100	13.95	100	8.41	100	23.41	100	19.28	100

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		SJ41		SJ42		SJ43	
Name		Fabian Tract		RD 1007		Grayson	
Toe Elevation[3]		5.5		6.3		31.6	
AWSE		10.4		10.4		42.4	
Crest Elevation[3]		21.3		19.3		46.2	
Type of Project (ULE or NULE)		NULE		NULE		NULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	5.49	0	6.27	0	31.60	0
		5.98	0	6.68	0	32.68	0
		6.47	0	7.09	0	33.76	0
		6.95	0	7.50	0	34.84	0
		7.44	0	7.91	0	35.92	0
		7.93	0	8.32	0	37.00	1
		8.42	1	8.73	0	38.08	1
		8.91	1	9.14	1	39.16	1
		9.40	2	9.55	1	40.24	2
		9.89	3	9.96	2	41.32	4
		10.38	4	10.37	3	42.40	6
		11.47	8	11.26	6	42.78	7
		12.57	11	12.15	9	43.16	8
		13.67	15	13.04	12	43.54	9
		14.76	18	13.93	15	43.92	10
		15.86	21	14.82	17	44.30	11
		16.95	24	15.71	20	44.68	11
		18.05	28	16.60	22	45.06	12
		19.15	31	17.49	25	45.44	13
		20.24	33	18.38	27	45.82	14
		21.34	100	19.27	100	46.20	100

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		STK6		STK7		STK8	
Name		Stockton Diverting Canal		Calaveras River North		Bear Creek South	
Toe Elevation[3]		31.2		8.6		19.8	
AWSE		33.8		9.9		25.7	
Crest Elevation[3]		36.8		17.6		30.0	
Type of Project (ULE or NULE)		NULE		ULE		ULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	31.16	0	8.58	0	19.85	0
		31.42	0	9.03	1	20.36	0
		31.68	0	9.48	1	20.87	0
		31.94	0	9.93	1	21.38	0
		32.20	0	10.38	1	21.89	1
		32.46	0	10.83	1	22.40	1
		32.72	0	11.28	2	22.91	1
		32.98	0	11.73	2	23.42	1
		33.24	1	12.18	2	23.93	1
		33.50	1	12.63	2	24.44	1
		33.76	2	13.08	14	24.95	4
		34.06	3	13.53	17	25.46	8
		34.36	4	13.98	21	25.97	13
		34.66	5	14.43	26	26.48	19
		34.96	6	14.88	32	26.99	26
		35.26	6	15.33	40	27.50	35
		35.56	7	15.78	49	28.01	46
		35.86	8	16.23	59	28.52	58
		36.16	9	16.68	68	29.03	67
		36.46	10	17.13	76	29.54	75
		36.76	100	17.58	81	30.05	81
				17.59	100	30.06	100

Table 4-2. San Joaquin River Basin Levee Performance Curves (contd.)

ID		STK9		STK10	
Name		Bear Creek North		Central Stockton	
Toe Elevation[3]		16.4		2.1	
AWSE		19.8		9.6	
Crest Elevation[3]		24.0		15.6	
Type of Project (ULE or NULE)		ULE		ULE	
Water Surface Elevation (NGVD29) (feet)	Associated Probability of Failure (breach) (percent)	16.39	0	2.08	0
		16.77	0	2.75	1
		17.15	0	3.43	1
		17.53	0	4.10	2
		17.91	0	4.78	2
		18.29	1	5.45	2
		18.67	1	6.13	3
		19.05	1	6.80	4
		19.43	1	7.48	5
		19.81	1	8.15	6
		20.19	1	8.83	8
		20.57	2	9.50	11
		20.95	2	10.18	15
		21.33	2	10.85	22
		21.71	2	11.53	30
		22.09	4	12.20	41
		22.47	6	12.88	55
		22.85	8	13.55	69
		23.23	11	14.23	78
		23.61	14	14.90	85
		23.99	18	15.58	90
		24.00	100	15.59	100

Notes:
[1] No State-federal project levees found within the impact area
[2] Assume overbank flow
[3] Elevations in feet, NGVD29

Key:
- = not applicable
AWSE = Assessment Water Surface Elevation
NGVD29 = National Geodetic vertical Datum of 1929
NULE = Non-Urban Levee Evaluations
RD = Reclamation District
ULE = Urban Levee Evaluations

4.2 Limitations

This assessment has been performed in accordance with the standard of care commonly used as the state-of-practice in the civil engineering profession. Standard of care is defined as the ordinary diligence exercised by fellow practitioners in this geographic area performing the same services under similar circumstances during the same time period. The levee performance curves are intended to be used in current hydraulic and economic damage modeling being performed by DWR for the CVFPP; these curves should not be taken out of this context in forecasting local levee performance issues.

The current version of the NULE levee performance curve Excel tool has produced curves for more than 200 NULE levee segments using cross-section-specific geometry, GAR categories, and a few curve-fitting parameters. Because geometries of levees vary widely, some curves may look distorted when compared to the expected curve shapes presented in Figure 3-1. This distortion is present to greater or lesser degrees for levees with only one or two high (C) or lacking sufficient data (LD) ratings and is further exacerbated for levees that are either very short (particularly if they have more than 3 feet of freeboard) or very tall (particularly if they have less than 3 feet of freeboard). The tool provides a set of curves with consistent properties relative to each other that are appropriate for the intended use in systemwide models and that are sufficient for initial hydraulic and damage modeling. The impact of these distortions (if any) can be addressed once results of initial damage model runs become available.

As mentioned above, levee performance curves presented in this attachment are intended for use with systemwide hydraulic and economic damage modeling performed for the CVFPP. Actual hydraulic and economic damage modeling results depend on a number of factors beyond the geotechnical levee performance curves (such as hydrologic and hydraulic uncertainty), and although the levee performance curves may seem reasonable, they may, when combined with other factors and used in the modeling, produce unexpected results; therefore, care must be taken in their use.

In the methodology described in Section 3, individual failure mode levee performance curves were combined to yield a cumulative or combined levee performance curve. This approach assumed that the failure modes are independent, and that the different failure processes operate independently. This assumption is likely not true in all cases and has been offset to some extent by reducing the probability of failure for individual failure modes.

In developing NULE levee performance curves, for simplicity, the geometry and location of the under-seepage cross section that was assessed in the GARs (URS, 2010a; Kleinfelder, 2010) was used for each curve. For some NULE segments, the GARs used different cross sections for different failure modes. In developing levee performance curves, geometry and location from the GAR under-seepage cross section was used as input for hydraulic and flood damage models.

As noted previously, levee crest elevations used in the Comprehensive Study (USACE, 2002) are sometimes different from those identified in the ULE and NULE projects. The ULE and NULE projects relied on recent LiDAR and CLD topographic data to estimate topographic parameters. The Comprehensive Study relied on older, since superseded, topographic information and, in most instances, the ULE and NULE levee crest elevations were used.

In developing NULE levee performance curves, results from the draft North and South GARs (URS, 2010b; Kleinfelder, 2010) were used without modification. There are ongoing efforts to finalize these GARs, and some of the data used in development of the levee performance curves may change. Similarly, ULE data that were current through the FCSSR, and some data used to develop the levee performance curves, may change as the ULE Project proceeds.

DWR makes no warranty that actual encountered site and subsurface conditions will exactly conform to the conditions described herein, nor that the interpretations and recommendations in this attachment will be sufficient for construction-planning aspects of any future work to reconstruct or remediate levees. The design engineer or contractor should perform a sufficient number of independent explorations and tests that the engineer or contractor believes are necessary to verify subsurface conditions rather than relying solely on the information presented in this attachment or other referenced documentation.

DWR does not attest to the accuracy, completeness, or reliability of maps, data sources, and geotechnical borings and other subsurface data produced by others that were presented in the GARs and used to develop levee performance curves described in this attachment. DWR has not performed independent validation or verification of data reported by others.

Data presented in this attachment are time-sensitive in that they apply only to locations and conditions that were identified at the time this attachment was prepared. Data should not be applied to any other projects in or near the area of this study nor should they be applied at a future time without

appropriate verification, at which point the entity verifying the data takes on the responsibility for the data and any liability for its use.

The levee performance curve information and results contained in this attachment is for the use and benefit of DWR. Use by any other party is at their own discretion and risk.

Information in this attachment should not to be used as a basis for design, construction, remedial action, or major project-specific capital spending decisions.

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6.0 Acronyms and Abbreviations

AWSE ..... Assessment Water Surface Elevation

Board ..... Central Valley Flood Protection Board

CLD ..... California Levee Database

Comprehensive Study Sacramento and San Joaquin River Basins Comprehensive Study

CVFPP ..... Central Valley Flood Protection Plan

Delta ..... Sacramento-San Joaquin Delta

DWR ..... California Department of Water Resources

ETL ..... Engineer Technical Letter

FCSSR ..... Flood Control System Status Report

FS ..... factor of safety

GAR ..... Geotechnical Assessment Reports

HEC-FDA ..... Hydrologic Engineering Center Flood Damage Assessment model

i ..... vertical exit gradient

LD ..... lacking sufficient data

LiDAR ..... light detection and ranging

NULE ..... Non-Urban Levee Evaluation

Pf ..... probability of failure

PFP ..... probable failure point

PNP ..... probable non-failure point

SPFC ..... State Plan of Flood Control

ULE ..... Urban Levee Evaluation

USACE ..... U.S. Army Corps of Engineers

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