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Sacramento District

Engineering Division

Lower San Joaquin River Feasibility Report - Environmental Impact Report / Environmental Impacts Statement

San Joaquin County, California

Hydraulics Addendum

Nov 2017

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QUALITY CONTROL CERTIFICATE

Hydraulic Analysis Section, Engineering Division

PROJECT NAME: SAN JOAQUIN BASIN, INTERIM FEASIBILITY STUDY
PRODUCT: HYDRAULIC DESIGN APPENDIX TO FEASIBILITY STUDY REPORT
Actual Completion Date: 22-Nov-17

PROJECT MANAGER: PATRICK HOWELL

Background: [Include project description, technical products, and review methodology]

District Quality Control was performed on the Nov 2017 report "Lower San Joaquin River Feasibility Report - Environmental Impact Report/Environmental Impacts Statement, San Joaquin County, California, and Hydraulic Design Appendix. Review comments, responses, and back checks on the report are located in the folder with the memorandum. This DQC review is of the final report which included modifications to address, OWPR comments obtained from the draft final report. The draft final report is available in the project folder.

HYDRAULIC LEAD

I have ensured that the above products were prepared in accordance with standard quality control practices. I have also incorporated or resolved all issues identified during District Quality Control (DQC) review.

Hydraulic Lead: Peter Blodgett	Title: Senior Hydraulic Engineer	11/22/2017
Print name	Signature	Date

REVIEWERS

I have reviewed the products noted above and find them to be in accordance with project requirements, standards of the profession, and USACE policies and standards.

DQC Reviewer: Jesse Schlunegger , Chief, Hydraulic Analysis Section	Jesse Schlunegger	11-22-2017
Print name	Signature	Date

RESOURCE PROVIDER

I have reviewed and resolved all critical and technical issues. I agree that all project requirements, standards of the profession, and USACE policies and standards have been met.

Section Chief: Jesse Schlunegger , Chief, Hydraulic Analysis Section	Jesse Schlunegger	11-22-2017
Print name	Signature	Date

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Acronyms and Abbreviations

ACE	Annual Chance of Exceedance
CNRFC	California Nevada River Forecast Center
CVFED	Central Valley Floodplain Evaluation and Delineation
CVFPP	Central Valley Flood Protection Plan
Comp Study	Sacramento and San Joaquin River Basins Comprehensive Study
DWR	Department of Water Resources
EC	Engineer Circular
ETL	Engineer Technical Letter
EM	Engineer Manual
ER	Engineer Regulation
FRM	Flood Risk Management
HEC	Hydrologic Engineering Center
HTOL	Hydraulic Top of Levee
NAD83	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NLDB	National Levee Database
NWS	National Weather Service
PBI	Peterson Brustad Incorporated
RD	Reclamation District
SD	Standard Deviation
SJAFCA	San Joaquin Area Flood Control Agency
ULDC	Urban Levee Design Criteria (State of California)
ULOP	Urban Level of Protection (State of California)
USGS	United States Geological Survey
USACE	United States Army Corps of Engineers
UPRR	Union Pacific Railroad
VE	Value Engineering

1.0 Introduction

1.1 Purpose and Scope

The purpose of this report is to describe the hydraulic analysis conducted in support of the Lower San Joaquin Feasibility Study. This final report is an addendum to the main feasibility study report. This report incorporates comments received during Agency Technical Review (ATR), Independent External Peer Review (IEPR), and Public Review. This report provides a description of the sources of potential flooding and documents the analysis of the final array of alternatives to reduce flood risk. The Section 11.0 of this report describes refinements made to the Recommended Plan to address feasibility study design requirements and support a Class III cost estimate. Analysis of the preliminary and focused array of alternatives is summarized in the main feasibility report. The level of detail of the final array is limited to that necessary to differentiate the final plans.

1.2 Background

The U.S. Army Corps of Engineers, together with the State of California and San Joaquin Area Flood Control Agency (SJAFCA) conducted this feasibility study to select a flood risk management plan that reduces flood risk and provides ancillary ecosystem restoration and recreation benefits within the study area. The goal of the study is to identify a cost effective, technically feasible and locally acceptable project that best reduces flood risk and flood damages and complies with all Federal, State, and local laws and regulations.

1.3 Location

The Lower San Joaquin study area is located within the Stockton metropolitan area of the State of California, approximately 50 miles south of Sacramento. The study area includes approximately 64 square miles of urban and agricultural lands subject to comingled flooding from multiple sources. A map of the San Joaquin River watershed is included as Plate 1. A map of the Sacramento-San Joaquin Delta is provided as Plate 2. A map of the study area topography is included as Plate 3 and a map of economic damage areas is presented in Plate 4.

The study area includes portions of communities of Stockton, Lathrop, and Manteca. Based on 2010 census data and floodplain mapping presented herein, approximately 235,000 people reside within the study area 0.2% (1/500) Annual Chance Exceedance (ACE) Floodplain. A map of population density within the study area is provided in Plate 5. The population within hypothetical natural floodplains is tabulated in Table 1. The hypothetical natural floodplain represents the area potentially at risk if a levee was to fail along any of the primary sources of flooding identified in this study.

The majority of land use in the study area is urbanized, comprising approximately 60% of land use. A map of land use types in the study area is presented in Plate 6. The amount of land that is currently developed, protected from development (parks, refuge lands, etc), and potentially developable is provided in Table 2. The primary sources of flooding within the study area are the San Joaquin River Delta, San Joaquin River, Mormon Slough, Calaveras River, and local interior drainage.

Table 1. 2010 Population, Lower San Joaquin Study Area

Economic Evaluation Area	Population within Natural ACE Floodplain						
	50% (1/2)	**10% (1/10)**	**4% (1/25)**	**2% (1/50)**	**1% (1/100)**	**0.5% (1/200)**	**0.2% (1/500)**
NS-02	13600	18700	19400	20400	21400	22800	23000
NS-03	11900	16100	16700	18400	18500	18800	18800
NS-04	0	0	0	26600	32300	35900	38800
CS-01	14300	19000	19900	22000	22600	22900	23100
CS-02	0	0	0	36200	42900	47300	47900
CS-03	0	0	0	24900	28500	31000	38800
RD17	0	0	25800	38200	43600	44600	44600
Total	39800	53800	81900	186600	209800	223300	235000

Table 2. Land Use Types, Lower San Joaquin Feasibility Study Area

Economic Evaluation Area	**Total Area Within 0.2% ACE Floodplain (Acres)**	**Area Protected from Development (Acres)**	**Developed Area (Acres)**	**Undeveloped or Unprotected Area (Acres)**
NS-02	2300	200	1800	300
NS-03	2400	0	1900	500
NS-04	3500	0	3000	400
CS-01	2600	100	2300	300
CS-02	6400	300	5200	900
CS-03	4200	100	3800	400
RD17	19600	200	6600	12800
Total	41200	900	24700	15500
Numbers may not total correctly due to rounding				

1.4 Plan Formulation

The recommended plan described in this report was selected through a risk informed plan formulation process involving multi-disciplinary analysis using an appropriate level of detail for decision making. At each level of screening and analysis the level of detail was improved and the relative uncertainty was assessed. A measure or alternative was carried forward if the level of detail was insufficient to screen it out. Throughout this process the concept of absolute accuracy versus relative accuracy was considered in alternative comparisons. Although it would appear that every plan should be compared to the most accurate assessment of existing conditions, this is not necessary because the relative accuracy between plans is sufficient to select the most optimal plans to move forward. The plan formulation process is summarized below and described in detail in the feasibility report.

The study area was defined based on an initial screening of flood risk management opportunities within the study area. The screening resulted in limiting the flood damage assessment to the economic impact areas shown on Plate 4.

An initial array of alternatives was derived from an evaluation of the without project conditions. The initial array included incremental levee improvements, setback levees and bypass channels.

A focused array of alternatives was derived from the initial array of alternatives. The focused alternatives were evaluated using qualitative and quantitative engineering analyses. Analyses included floodplain hydraulic modeling, cost estimating, and economic benefit estimations. The level of detail was limited to that required to decide which plans to carry forward. Results were evaluated at a combined Value Engineering (VE) study and planning charette attended by the project sponsors and subject matter experts. At the conclusion of the VE study and planning charette, refinements to the focused array of alternatives were identified for further, more detailed analysis. The analysis of the focused array of alternatives included an evaluation of levee raises in select locations. The levee raises were found to produce greater net benefits than without raises. Therefore, the final alternatives included the levee raises. This is discussed in the Feasibility Study Report and Economic Addendum.

Final alternatives were selected from the focused alternatives to be studied in increased detail. The level of detail was increased by included qualitative and quantitative engineering analyses. Analyses included refined cost estimating, economic benefit estimates, and impacts analysis. The level of detail was limited to that required to decide which plan to carry forward as the recommended plan. Additional details describing hydraulic analysis performed for the study are available in internal memorandums on file within the Sacramento District Hydraulic Analysis Section. A summary of the final alternatives described in this report is provided in Table 3. As described in the main report, Alternative 7A was selected as the recommended plan.

Table 3

Comparison of Final Alternative Features

Alternative	Improve Delta Front Levees	Improve North and Central Stockton San Joaquin River Levees	Improve RD17 San Joaquin Levees	Improve Lower Calaveras River Levees	Improve Stockton Diverting Canal Levees	Construct Mormon Slough Bypass	Extend Right Bank Levee of French Camp Slough along Duck Creek	Raise levee height at selected locations
1								
7A	X	X		X			X	X
7B	X	X	X	X				X
8A	X	X		X	X		X	X
8B	X	X	X	X	X			X
9A	X	X		X		X	X	X
9B	X	X	X	X		X		X

1.5 National Flood Insurance Program (NFIP).

NFIP levee accreditation is not a specific USACE planning objective. Estimates of Flood Risk Management (FRM) performance presented in this report are limited to the level of detail needed to support economic analysis and comparison of alternatives during the feasibility study process. Results presented herein may not be sufficiently detailed to support NFIP levee accreditation and do not address all of the guidance requirements in EC 1110-2-6067, USACE Process for the National Flood Insurance Program Levee System Evaluation. In addition, hydrologic and hydraulic results presented in this report may be superseded by results from hydrologic and hydraulic models currently being developed by the State of California and local sponsors. The non federal sponsor is responsible for demonstrating a plan meets the sponsor's NFIP objectives.

The U.S. Department of Homeland Security's FEMA is the federal agency responsible for administering the NFIP. As part of the NFIP, FEMA develops Flood Insurance Rate Maps (FIRMs) to identify areas that may be subject to flooding, for both determining flood insurance rates and flood plain management activities (USACE, 2010). FEMA accredits a levee as providing adequate risk reduction on the FIRM if the levee is certified and an adopted operation and maintenance plan provided by the levee owner are confirmed to be adequate (FEMA, 2012). An area impacted by an accredited levee is still considered within the base floodplain but is shown as a moderate-risk area and is labeled Zone X (shaded) on a FIRM. In this case, the National Flood Insurance Program (NFIP) floodplain management regulations do not have a mandatory flood insurance purchase requirement (FEMA 2012). If the levee is not accredited, the area will be mapped as a high-risk area, known as a Special Flood Hazard Area, or SFHA (FEMA, 2012). In this case, the NFIP floodplain management regulations must be enforced and the federal mandatory purchase of flood insurance applies (FEMA, 2012).

Certification consists of documentation, signed and sealed by a registered Professional Engineer, as defined in Chapter 44 of the Code of Federal Regulations (44 CFR), Section 65.2 (FEMA, 2012). This documentation must state the following:

- The levee meets the requirements of 44 CFR, Section 65.10
- The data is accurate to the best of the certifier's knowledge
- The analyses are performed correctly and in accordance with sound engineering practices

This documentation is provided to FEMA to demonstrate that a registered Professional Engineer certified the levee, and meets the specific criteria and standards to provide risk reduction from at least the one-percent-annual-chance flood (FEMA, 2012).

44 CFR, Section 65.10 provides two options for determining if a levee meets the hydrology and hydraulics requirements for levee certification.

- Freeboard Option. Riverine levees must provide a minimum freeboard of three feet above the water-surface level of the base (1% (1/100) ACE) flood. An additional one foot above the minimum is required within 100 feet in either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required.

- Risk and Uncertainty Option. Exceptions to the minimum riverine freeboard requirement may be approved by FEMA. Appropriate engineering analyses demonstrating adequate protection with a lesser freeboard must be submitted to support a request for such an exception. The material presented must evaluate the uncertainty in the estimated base flood elevation profile and include, but not necessarily be limited to an assessment of statistical confidence limits of the 1% (1/100) ACE discharge; changes in stage-discharge relationships; and the sources, potential, and magnitude of debris, sediment, and ice accumulation. It must be also shown that the levee will remain structurally stable during the base flood when such additional loading considerations are imposed. Under no circumstances will freeboard of less than two feet be accepted.

In the case of USACE certification, EC 1110-2-6067 requires specific assurance levels be met. For assurance less than 90% the levee does not pass the EC 1110-2-6067 NFIP criteria. For assurance between 90 and 95% the levee must have minimum of 3 feet of freeboard to pass the EC 1110-2-6067 NFIP criteria. For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass the EC 1110-2-6067 NFIP criteria.

Both approaches also require minimum geotechnical, geometry, erosion control (including wind-wave action), vegetation, right of way, encroachment, and penetration standards, plus a number of other standards.

Once the levee meets all the requirements of 44 CFR 65.10, FEMA can accredit the levee and show the area behind it as being a moderate-risk area on a Flood Insurance Rate Map (FIRM) (FEMA, 2012). Levee certification does not warrant or guarantee performance, and it is the responsibility of the levee owner to ensure the levee is being maintained and operated properly (FEMA, 2012). Should USACE be requested to provide an NFIP levee system evaluation, USACE will review all components of the entire levee system as outlined in EC 1110-2-6067, not only design and construction issues as noted in the CFR (USACE, 2010).

Since NFIP accreditation is not a USACE planning objective in the formulation of the National Economic Development (NED) plan, the ability of an NED plan to meet the NFIP criteria is uncertain. An NED plan could appear to meet these criteria during Feasibility. However, an NED plan has no specific authorizing language that requires these criteria are to be met. As a result, it is possible that further analysis during Planning Engineering and Design could determine a NED plan does not meet the NFIP criteria. On the other hand, an NED plan could appear to NOT meet the NFIP criteria during feasibility but could be found to meet those requirements after final design or construction.

1.6 California State Urban Level of Protection.

A local sponsor objective is to meet the California State Urban Level of Protection (ULOP) requirement defined in California Government Code 65007(I). However, this is not a Federal planning objective or requirement. Estimates of Flood Risk Management (FRM) performance presented in this report are limited to the level of detail needed to support economic analysis and comparison of alternatives during the feasibility study process. In addition, hydrologic and hydraulic results presented in this report may be superseded by results from hydrologic and hydraulic models and analysis currently being developed by the State of California and local sponsors. The non federal sponsor is responsible for demonstrating a plan meets the sponsor's ULOP objectives or requirements.

The requirements for a levee to be recognized as contributing to an ULOP are defined in the May 2012 State of California report "Urban Levee Design Criteria" (DWR, 2012). The purpose of the Urban Levee Design Criteria (ULDC) is to provide engineering criteria and guidance for civil engineers to follow in meeting the requirements of California's Government Code Sections 65865.5, 65962, and 66474.5 with respect to findings that levees and floodwalls in the Sacramento-San Joaquin Valley provide protection against a flood that has a 1-in-200 chance of occurring in any given year (Annual Chance of Exceedance (ACE)), and to offer this same guidance to civil engineers working on levees and floodwalls anywhere in California (DWR, 2012).

The ULDC provides two options for determining if a levee meets the urban and urbanizing area levee system design.

- The freeboard option (option 1) requires 3 feet of freeboard above the median 0.5% (1/200) ACE flood event.
- The risk and uncertainty option (option 2) allows for a lesser amount of freeboard if a high level of assurance can be demonstrated. For assurance less than 90% the levee does not pass the ULDC criteria. For assurance between 90 and 95% the levee must have minimum of 3 feet of freeboard to pass the ULDC criteria. For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass the ULDC criteria.

Both ULDC approaches require that modeled water surface profiles assume other levees in the system can overtop, but not fail. Other urban area levees throughout the system are assumed to be at their existing elevation or 0.5% (1/200) plus 3 feet of freeboard, whichever is higher, and non-urban levees are assumed to be at their existing elevation or their authorized design profile, whichever is higher. Both ULDC approaches require that additional freeboard be provided if the wind-wave run-up from a 1.3% ACE wind event would exceed the top of levee for the 0.5% (1/200) ACE event. Both ULDC approaches also require minimum geotechnical, geometry, erosion control, vegetation, right of way, encroachment, and penetration standards, plus a number of other standards.

Since a ULOP finding is not a USACE planning objective in the formulation of the National Economic Development Plan (NED) plan, the ability of an NED plan to meet the ULOP criteria is uncertain. An NED plan could appear to meet these criteria during Feasibility. However, an NED plan has no specific authorizing language that requires these criteria are to be met. As a result, it is possible that further analysis during Planning Engineering and Design could determine an NED plan does not meet the ULOP criteria. On the other hand, an NED plan could appear to NOT meet the ULOP criteria during feasibility but could be found to meet those requirements after final design or construction.

1.7 Evaluation Approach for Final Array

This report describes the hydraulic design and performance analysis of the final alternative array and the recommended plan of the Lower San Joaquin Feasibility Study. Each feature of an alternative was designed following USACE criteria. The performance of each alternative was then evaluated by adjusting inputs in the USACE FDA program to reflect the features of the alternative. The approach of simulating an alternative's performance by changing FDA inputs is described in Section 9 of EM 1110-2-1619, Risk Analysis for Flood Damage Reduction Studies. Inputs to the FDA program were unregulated flow frequency, unregulated flow versus regulated flow, regulated flow versus stage, levee fragility, and stage-damage relationships and their uncertainties. Flow charts describing the hydraulic analysis performed to evaluate the alternatives are provided in Plates 7 and 8 for the San Joaquin and Calaveras Rivers respectively.

a. Final Array Model Scenarios. Hydraulic models were developed to represent two scenarios to support the evaluation of the final array. The results of the following two scenarios were utilized to develop the FDA inputs to the six final alternatives.

(1) Scenario A. This scenario reflects existing levee footprints, levee height, and other hydraulic features. Hydraulic model geometry and flows were based on simulating this existing condition.

(2) Scenario B. This scenario reflects an extension of the RD17 tieback levee at Wetherbee Lake and Walthall Slough to higher ground. For floods events larger than 0.5% (1/200) ACE this results in a stage reduction in the San Joaquin River below Old River. Flood flows that would otherwise outflank the tieback levee and reenter the San Joaquin River at French Camp Slough are diverted to Paradise Cut and Old River. No modifications to the inflow hydrology were necessary because urban areas are significantly upstream and would likely have no impact on flows in the study reach.

b. Project Reach Segments. The study area was divided into project reach segments described in Plates 9A through 9D. The segments were defined based on similar hydrologic, hydraulic, design, and geotechnical characteristics. The engineering design and costs were developed for each of the project reach segments and combined to estimate the costs of each alternative. The estimated cost of each alternative is provided in the feasibility study report.

c. Economic Impact Areas. Economic impact areas were defined based on the concept of "separable area". Separable areas or elements are defined as the subdivision of a study area's flood risk based on hydrologic and hydraulic characteristics with identifiable and distinct economic benefits. A "separable element" is defined in 33 United States Code (U.S.C.) Section 2213(f) as a portion of the project that (1) is physically separable from other portions of the project; and (2)(a) achieves hydrologic effects, or (b) produces physical or economic benefits, which are separately identifiable from those produced by other portions of the project.

Within the Lower San Joaquin study area, the floodplain has a relatively low gradient and topographic relief and the separable areas are not clearly defined by basic topographic features alone. The physical separation was estimated by analyzing the hydrologic characteristics. In general, there are eight separable hydrologic areas. The separation is evident in levee breach simulations conducted for the study and described below. The delta region defines many of the separable areas. The stage within the delta region is affected by coincident ocean tides and inflows from the Sacramento and San Joaquin River system. The physical separation between portions of the Lower San Joaquin study area is described below.

(1) North Stockton 01 (NS-01). This area was screened from the final study area early in the plan formulation process. This area is subject to flooding if a breach were to occur in the levees along the upstream reaches of Bear Creek or Mosher slough and the downstream delta reaches. The eastern limit of the NS-01 area defines the limit of delta flood sources.

(2) North Stockton 02 (NS-02). This area is subject to flooding if a breach were to occur in the levees along the upstream reaches of Mosher Slough, Calaveras River, and downstream delta reaches including Fourteenmile Slough. The eastern limit of the NS-02 area defines the limit of delta flood sources.

(3) North Stockton 03 (NS-03). This area is subject to flooding if a breach were to occur in the levees along the upstream Calaveras River, and downstream delta reaches including Fourteenmile Slough. The eastern limit of the NS-03 area defines the limit of delta flood sources.

(4) North Stockton 04 (NS-04). This area is subject to flooding if a breach were to occur in the levees along the upstream Calaveras River. The area is not subject to flooding from downstream delta reaches.

(5) Central Stockton 01 (CS-01). This area is subject to flooding if a breach were to occur in the levees along Calaveras River, Stockton Diverting Canal, delta reaches, French Camp Slough, and San Joaquin River.

(6) Central Stockton 02 (CS-02). This area is subject to flooding if a breach were to occur in the levees along Stockton Diverting Canal, French Camp Slough, and San Joaquin River.

(7) Central Stockton 03 (CS-03). This area is subject to flooding if a breach were to occur in the levees along Stockton Diverting Canal and Calaveras River. The area is not subject to flooding from the San Joaquin River or delta reaches. The western limit of the area defines the limit of delta flood sources.

(8) Reclamation District 17 (RD17). This area is subject to flooding if a breach were to occur in the San Joaquin River levee or the RD17 tieback levee at Weatherbee Lake and Walthall Slough.

1.8 Evaluation Approach for Recommended Plan

Further analysis was conducted to refine and evaluate the recommended plan. These analysis and results are provided in Chapter 11.

1.9 Datum

As required by ER 1110-2-8160 all elevation data provided herein are referenced to the NAVD88 vertical datum. All horizontal data provided herein are referenced to the North American Horizontal Datum of 1983 (NAD83) Horizontal datum. All horizontal coordinates are projected to the California State Plane Zone III coordinate system.

Historical elevation data were converted to NAVD88 from their original legacy reference datum. The method of conversion followed the requirements in ER 1110-2-8160 and the uncertainty in the conversion was accounted for in the study results. In some cases, the original data used for this study was based on NAVD88 and required no conversion.

The following generalized conversion is provided to compare NAVD88 elevations provided in this study to previous studies presented in the legacy NGVD29 datum. Expressed as an equation, Elevation (NGVD29) = Elevation (NAVD88) minus 2.3 to 2.4 feet. The conversion between NAVD88 and NGVD29 ranges from 2.3 to 2.4 feet in the study area.

2.0 STUDY AREA

2.1 Overview

The study area is situated within the Sacramento-San Joaquin Delta watershed. A map of the watershed is included as Plate 1. The contributing drainage area to the Sacramento-San Joaquin Delta encompasses approximately 40,000 square miles. The main contributors of the drainage area are the Sacramento River (25,200 square miles), San Joaquin River (13,500 square miles), and the Mokelumne River (1,200 square miles). Runoff within the study area is highly influenced by upstream reservoir regulation.

2.2 Topography

A topographic map of the study area is presented in Plate 3. The study area has a general slope from east to west. Elevations within the study area range from 50 ft NAVD88 in the east to -20 ft NAVD88 in the west. The general slope of the study area is interrupted by roadway and railway embankments and levees. These features significantly influence the direction of shallow floodwaters within the floodplain.

2.3 Principle Sources of Flooding

The study area is susceptible to comingled flooding from six principle sources including the Sacramento-San Joaquin Delta, San Joaquin River, Calaveras River and Mormon Slough system, Bear Creek, French Camp Slough system, and Mosher Slough. Interior drainage is not considered a principle source of flooding. The following describes the flood sources within the study area.

a. Sacramento and San Joaquin Delta. The Sacramento and San Joaquin Delta covers more than 1,000 square miles of Central California. A map of the delta is provided as Plate 2. The delta is located at the confluence of the Sacramento and San Joaquin Rivers at the head of Suisun Bay, the most easterly extending arm of the San Francisco Bay system. In general, the Delta extends from about Sacramento on the north, to Stockton on the south, and near Pittsburg on the west. This region, which is very flat, has been reclaimed from a natural tidal area by hundreds of miles of levees along natural and manmade waterways that divide it into about 100 tracts locally know as "islands".

Before the islands were reclaimed, much of the Delta was covered by water from the daily tide cycle. During times of high runoff from the Sacramento and San Joaquin Basins, much of the Delta would be flooded. Reclamation of the many of the Delta islands has subjected the peat soils to oxidation. As a result, the interior of most islands have subsided well below sea level. Elevations within the islands now range from just above mean sea level to 10 feet below mean sea level.

Maximum stages within the Delta result from runoff from storms of different origins which do not have the same annual exceedance frequency at all locations, and from tides of varying magnitudes which seldom reach their maximum stages concurrently with the peak flows. In some years the annual maximum stage at all locations occurs during the same storm event. However, in other years, the peak stages in the northern part of the Delta occur during a different time period than those in the southern part of the Delta and vice versa. The differences are caused by the geographical distribution of the contributing drainage basin, antecedent conditions such as snowpack and soil moisture, and the fluctuation of the storm tracks over California. If the flood runoff is from the Sacramento River basin, the stages will be higher in the northern part of the Delta. If the main flood runoff is from the San Joaquin River, then the stages will be higher in the southern part of the Delta.

The Delta Front reaches of the study area is susceptible to flooding from Fourteenmile Slough and Ten Mile Slough. These delta sloughs have relatively small tributary areas. However, the levees along these sloughs provide flood risk reduction from the large volume of water in the Sacramento San Joaquin Delta. If a breach were to occur in a delta front levee, the floodwaters would likely equalize with the high stage of the delta due to the enormous volume of water.

b. San Joaquin River. The San Joaquin River is the principle stream in the southern half of the Central Valley of California. The San Joaquin is a perennial stream sustained through the summer by melting snow and releases from reservoirs. Its main headwater tributaries, the south and middle forks, rise in glacial lakes in the southern Sierra Nevada. They join at about elevation 3600 feet NAVD88 to form the main stem, which flows west-southwesterly to the valley floor, thence northwesterly down the main trough of the valley to the study area and its terminus at Suisun Bay. Upstream from the study area, the river is joined by several major tributaries flowing from the higher elevations of the Sierra Nevada Mountain Range. There are also a number of minor low elevation tributaries that flow from the east and west and have little effect on flood flows and stages.

The major tributaries flowing from the Sierra Nevada Mountains to the east are the Stanislaus, Tuolumne, Merced, Chowchilla, and Fresno Rivers. Less significant eastside tributaries comprise Calaveras River, Bear Creek, and French Camp Slough (terminus of Duck and Littlejohns Creeks systems). The principal westside tributaries are Panoche, Los Banos, San Luis, and Orestimba Creeks. Fresno Slough, a distributary of the Kings river that cuts through the valley-floor barrier ridge separating the Tulare Lake Basin from the San Joaquin River Basin proper, could contribute runoff to the San Joaquin River during extreme flood events. Reaches of the San Joaquin River within the study area are described below.

(1) Stanislaus River to Paradise Cut. The confluence of the San Joaquin and Stanislaus Rivers defines the upstream extent of the hydraulic model used for this study. The USGS San Joaquin River near Vernalis stream gage is located at the upstream end of this reach approximately 2 miles downstream of the Stanislaus River. Within this reach the San Joaquin River has a meandering plan form consisting of oxbows and cutoffs. The main channel slope is approximately 0.8 feet per mile and varies in width from 300 to 600 feet. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. The floodway between the levees varies in width from 900 feet to 4000 feet. The distance between the waterside levee toe and channel bank ranges from zero feet to over 2000 feet. Flood stages within this reach are dominated by runoff from the San Joaquin River watershed.

2) Paradise Cut to Old River. Paradise cut defines the upstream extent of this reach. Paradise cut is a distributary from the San Joaquin River and conveys floodwaters west into the Sacramento-San Joaquin Delta. The flow split into paradise cut is managed by Paradise Dam which is a 230 foot long rock weir along the left bank of the San Joaquin River. The flow split is defined by the hydraulic characteristics of the dam and a meander cutoff levee located on the San

Joaquin River downstream of the dam. The meander cutoff levee extends west from the right bank levee and impinges on the San Joaquin River downstream of Paradise Cut.

Within this reach the San Joaquin River transitions to a less sinuous plan form. The main channel slope is approximately 0.6 feet per mile and varies in width from 300 to 600 feet. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. At the upstream end of the reach, the floodway width between the levees varies from 900 feet to 4000 feet and the distance between the waterside levee toe and channel bank ranges from zero feet to over 2000 feet. At the downstream end of the reach, the floodway width narrows to approximately 500 feet. However, there is one oxbow reach where the floodway is approximately 2000 feet wide. Flood stages within this reach are dominated by runoff from the San Joaquin River watershed.

Approximately 1 mile downstream of Paradise cut on the right bank is Wetherbee Lake and the upstream tieback levee of RD17. The Wetherbee Lake levee segment along the San Joaquin River was a feature of the San Joaquin Flood Control Project which cut off Walthall slough from the San Joaquin River to reduce damages to a resort development along the river. The RD17 tieback is located downstream of Walthall Slough and extends east along the right bank of the slough to high ground. The RD17 tieback levee is higher than the upstream right bank levee of the San Joaquin River and diverts any floodwaters on the right overbank back into the San Joaquin River. This situation occurred in the flood of January 1997 and is shown on Plate 10. Flood stages within this channel reach are dominated by runoff from the San Joaquin River. Flood stages in the right overbank are dominated by runoff from the San Joaquin River and Stanislaus River.

(3) Old River to French Camp Slough. Old River defines the upstream extent of this reach. Old River is a distributary from the San Joaquin River and conveys floodwaters west into the Sacramento-San Joaquin Delta. There is no hydraulic structure to manage the flow split. The flow split is defined by the hydraulic characteristics of Old River and the San Joaquin River downstream of the flow split.

Within this reach the San Joaquin River further transitions to a less sinuous plan form. The main channel slope is approximately 0.09 feet per mile and varies in width from 200 to 300 feet. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. From Old River to approximately 4 miles downstream right bank levee is approximately 3 feet taller than the left bank. The floodway width between the levees varies from 300 feet to 400 feet and widens to 1400 feet at a few meander bends. The waterside levee face forms the channel bank along most of this reach. Flood stages within this reach are dominated by runoff from the San Joaquin River.

(4) French Camp Slough to Burns Cutoff. French camp slough defines the upstream extent of this reach. French camp slough is a tributary to the San Joaquin River. The reach characteristics of French Camp slough are described below. Within this reach the main channel slope is approximately 0.09 feet per mile. The main channel varies in width from 200 to 300 feet. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. The floodway width between the levees varies from 300 feet to 400 feet. The waterside

levee face is next to the channel bank along most of this reach. Flood stages within this reach are dominated by runoff from the San Joaquin River. However, influence of ocean tides is evident in flood stage hydrographs.

(5) Burns Cutoff to Deep Water Ship Channel. Burns Cutoff defines the upstream extent of this reach. Burns cutoff is a secondary channel of the San Joaquin River which conveys water on the west side of Rough and Ready Island. Burns cutoff flows back to the San Joaquin River/Stockton Deep Water Ship Channel just downstream of the Calaveras River.

Within this reach the San Joaquin River main channel slope is approximately 0.05 feet per mile and is approximately 300 feet wide. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. The right bank levee height tapers to high ground at the downstream end of the reach where it meets the San Joaquin Deep Water Ship Channel. The floodway width between the levees varies from 300 feet to 400 feet. The waterside levee face is next to the channel bank along most of this reach. Flood stages within this reach are dominated by runoff from the San Joaquin River. However, influence of ocean tides is evident in flood stage hydrographs.

(6) Deep Water Ship Channel to Calaveras River. The Stockton Deep water ship channel turning basin defines the upstream extent of this reach. Within this reach the San Joaquin River is maintained as a navigation channel through periodic dredging to a minimum draft of 35 feet below Mean Low Low Water (MLLW). Within this reach the channel slope is approximately 0.05 feet per mile is approximately 600 feet wide and is contained by high ground on either side. Smith canal is located along the right bank of this reach approximately one mile downstream of the turning basin. The Calaveras, a tributary to the San Joaquin River is near the downstream end of this reach. Flood stages within this reach are dominated by runoff from the Sacramento and San Joaquin Rivers in combination with ocean tides. Inflows from the Calaveras River and Smith Canal have a negligible influence on the stage in this reach because flood flows are not coincident with the San Joaquin River. In addition the San Joaquin River has a relatively large cross sectional area due to the channel dredging.

c. Calaveras River and Mormon Slough. The Calaveras River is a tributary of the San Joaquin River. Elevations in the Calaveras River drainage vary from about 6,000 feet in the highest headwater areas to about 30 feet in the lower part of the study area. A map of the watershed is provided in Plate 11. In the study area, the Calaveras River is distributary in nature. The stream divides into the north and south branches at Bellota, where a diversion structure was constructed as part of the Federal Mormon Slough Project. The average slope of Mormon Channel is 6.2 feet per mile from Bellota to the Diverting Canal. The average slope of the Diverting Canal from Mormon Slough to the Calaveras River is 3.6 feet per mile. The average slope of the Calaveras River downstream of the diverting canal to the Stockton Deep Water Ship Channel is 2 feet per mile. The average slope of the Mormon Slough from Belota to the Diverting Canal is The northern branch Calaveras River, flows westerly across the valley floor to join the San Joaquin River just west of Stockton. Very little flow enters this branch except during the summer when diversions are made for irrigation and ground-water replenishment. The southern branch, Mormon Slough, carries most of the flow. Its course extends in a general southwesterly direction from Bellota to the Stockton Diverting Canal diversion dam. The structure diverts all flood flows to the diverting canal which discharges into the Calaveras River. The Mormon Slough reach below the diverting dam is referred to locally as Old Mormon Slough. The source of flow in Old Mormon Slough is the local tributary area downstream of the diversion structure.

d. Bear Creek. Bear Creek is a tributary to Disappointment Slough of the San Joaquin Delta. Bear Creek is located near the city of Stockton. A map of the watershed is provided as Plate 12. At its confluence with Disappointment Slough, Bear Creek has a drainage area of approximately 115 square miles. The watershed drains the western slopes of the Sierra Nevada foothills and has a maximum elevation of 1,000 feet NAVD88. The watershed is significantly below the average snowline elevation. The average slope of Bear Creek is approximately 4.8 feet per mile from Highway 88 to Interstate 5. Based on preliminary hydrologic and hydraulic model analysis, Bear Creek was not found to be a source of flood risk to the economic impact areas defined within the study area boundary. Therefore, the results of the detailed hydraulic analysis for Bear Creek are not provided in this report.

e. Duck Creek. Duck Creek is a small tributary of the French Camp Slough, south of the City of Stockton, lying between the Calaveras River-Mormon Slough system and Littlejohn Creek. It has a total drainage area of 54 square miles. A map of the watershed is included in Plate 13. Reduction of flood flow in the stream is accomplished by the Farmington Reservoir Project, which prevents overflow of Littlejohn Creek floodwater into Duck Creek, and the Duck Creek Diversion which diverts floodwater from upper Duck Creek into the improved channel of Littlejohn Creek. Approximately half of the Duck Creek drainage area lies above the Duck Creek Diversion Dam. The upstream area, about 28 square miles in extent, lies below 500 feet in elevation and is a typical foothill area, with an overall streambed slope of about 20 feet per mile. Downstream of the diversion structure the gently sloping flat valley floor is a poorly defined tributary drainage area. Within the study area Duck Creek is under backwater from the San Joaquin River and has a very slight channel slope. This creek has no effect on major flood flows in the San Joaquin River.

f. French Camp Slough. French Camp Slough is a tributary to the San Joaquin River south of the City of Stockton. The slough receives waters from Duck Creek and Littlejohn Creek. A map of the watershed is provided as Plate 13. At its confluence with the San Joaquin River, French Camp slough has a drainage area of approximately 430 square miles. The watershed drains the western slopes of the Sierra Nevada foothills and has a maximum elevation of 2,100 feet NAVD88. The watershed is significantly below the average snowline elevation. This slough, with or without upstream reservoirs has no effect on major flood flows in the San Joaquin River (USACE, 1955). Within the study area French Camp Slough is under backwater from the San Joaquin River and has a very slight channel slope.

g. Mosher Slough. Mosher slough is a small tributary to Bear Creek which discharges to Disappointment Slough of the Sacramento-San Joaquin Delta. Mosher Slough is located near the City of Stockton in San Joaquin County, California. A map of the watershed is provided in Plate 14. The majority of the watershed is located in the urbanized area of Stockton between Interstate-5 and Highway 99 with the watershed area totaling approximately 16 square miles (SJAFCA, 2012). The watershed's terrain has moderate slopes and reaches a maximum elevation of 65 feet NAVD88. Based on hydrologic frequency analysis the runoff from the area upstream of Thornton Ave is estimated to be 690cfs for the 10% (1/10) ACE event and 940cfs for a 1% (1/100) ACE event. These flows do not meet the minimum requirements of 800cfs for a 10% (1/10) ACE event and 1800cfs for a 1% (1/100) ACE event required to establish Federal Flood Control Authority in CFR 238.7(a). However, inclusion of flood risk management measures to Thornton Ave to address high stages of the Sacramento-San Joaquin Delta would meet the requirements of CFR238.7 (a) (4). It is estimated that flood risk from the Sacramento-San Joaquin Delta extends to Thornton Ave and this defines the limit of Federal Interest required by CFR238.7. Within the study area Mosher Slough is under backwater from the Sacramento -San Joaquin Delta and has a very slight channel slope.

2.4 Related Federal Flood Risk Management Projects.

Development of water resources in the basin began in the 1850's and currently includes large multiple-purpose reservoirs, extensive levee and channel improvements, bypasses, and local diversion canals (USACE, 1993). Numerous agencies have been involved in water resources development within the study area. Some of these agencies include the USACE, United States Bureau of Reclamation (USBR), State of California, county irrigation districts, local reclamation districts, and local levee districts. Design flows for flood risk management projects within the study area are provided in Table 4. These projects were constructed prior to USACE regulations requiring the use of Risk and Uncertainty (R&U) methods to describe project performance. Therefore, the projects are described by their design flow and freeboard. Reservoir projects upstream of the study area with dedicated federally authorized flood control space are described in Table 5. The following describes existing Federal Flood Risk Management Projects affecting the study area.

Table 4 Project Design Flood Flows

Reach	Design Flow (cfs)	Design Freeboard (feet)	Source:
Mormon Slough			
Bellota to Potter Creek	12,500	3 with levee 1.5 w/o levee	USACE, 1974
Potter Creek to Diverting Canal	13,500 15,143 (b)	3 with levee 3.3 w/ levee (b) 1.5 w/o levee	USACE, 1974 USACE, 2010a
Stockton Diverting Canal			
Mormon Slough to Calaveras River	13,500 14,800(b)	3 3.3 (b)	USACE, 1974 USACE, 2010a
Upper Calaveras River			
At Diverting Canal	1,120 (b)	3.3 (b)	USACE, 2010a
Lower Calaveras River			
Diverting Canal to San Joaquin River	13,500	3	USACE, 1974
Potter Creek			
Jack Tone Road to Mormon Slough	1000 1,320 (b)	3.3 (b)	USACE, 2010a
San Joaquin River			
Stanislaus River to Paradise Dam (at head of Paradise Cut)	52,000	3	USACE, 2011b
Paradise Dam to Old River	37,000 (a)	3	USACE, 2011b
Old River to French Camp Slough	22,000	3	USACE, 2011b
French Camp Slough to Stockton Deep Water Ship Channel	18,000	3	USACE, 2011d
French Camp Slough			
Right Bank French Camp turnpike to San Joaquin River	2000	3	USACE, 2011b
Left Bank French Camp turnpike to San Joaquin River	3000	3	USACE, 2011c
Duck Creek			
Duck Creek Diversion to Mariposa Road	700	Not Available	USACE, 2010b
Mariposa Road to French Camp Slough	900	Not Available	USACE, 2010b
Bear Creek (b)			
Highway 99 to Western Pacific Railroad	5,500	3	USACE, 2012
Western Pacific Railroad to Pixley Slough	6,350	3	USACE, 2012
Pixley Slough to San Joaquin River	7,060	3	USACE, 2012

(a) Design diversion capacity of Paradise Cut is 15,000 cfs
(b) Change in design flows by WRDA 2007 per revised Operations and Maintenance Manual, Federal Project levee ends at Disappointment Slough (about 4000 feet upstream of Pixley Slough).

Table 5 Reservoir Projects with Dedicated Flood Storage, San Joaquin River Basin

Reservoir	Owner	Year Constructed	Objective Flow (cfs)	Objective Flow Location	Gross Pool Storage (ac-ft)	Max Dedicated Flood Space (ac-ft)
Friant	USBR	1942	8,000 6,500	Little Dry Creek at Mendota Gage	520,500	170,000
Big Dry Creek	FMFCD	1948	700	Wasteway	30,200	30,200
Farmington	USACE	1951	2,000	Town of Farmington	52,000	52,000
Camanche	EBMUD	1963	5,000	Below Dam	430,900	200,000
New Hogan	USACE	1963	12,500	at Belota	317,100	165,000
Los Banos	USBR	1965	1,000	Los Banos	34,600	14,000
New Exchequer	Merced ID	1967	6,000	Cressey	1,024,600	350,000
Don Pedro	Turlock ID	1971	9,000	Modesto	2,030,000	340,000
Buchanan	USACE	1975	7,400 7,000	Below Dam Chowchilla River at Madera	150,000	45,000
Hidden	USACE	1975	5,000	at Medara Canal	90,000	65,000
New Melones	USBR	1979	8,000	Orange Blossom	2,400,000	450,000

a. New Hogan Lake. New Hogan Lake was authorized by the Flood Control Act of 1944 (Public Law 534, December 22 1044, 78th Congress, 2nd Session). The project is located on the Calaveras River about 28 miles northeast of Stockton, Ca and comprises a rockfill dam with an impervious earth core and a maximum height of about 200 feet. The project also includes four dikes, with a maximum height of 18 feet, and a gated spillway to create a reservoir with a gross storage capacity of 325,900 acre-feet for flood control, irrigation and other water conservation purposes. Construction was initiated in May 1960, dam closure was made in November 1963, and the project was completed for operational use in June 1964.

b. Stockton and Mormon Slough (Diverting Canal). Improvement of Stockton and Mormon Slough was authorized by the River and Harbor Act of June 13, 1902 (H. Doc. 152, 55th Congress, 3d Session, and Annual Report for 1899, p. 3188), to provide for diversion of the waters of Mormon Slough before reaching the Stockton Deep Water Ship Channel, for the purpose of preventing deposits of material in the navigable portions of the channels and to divert flood flows past the city of Stockton, California. The results were obtained by construction of (1) a dam across Mormon Slough; (2) a diverting canal 150 feet wide, extending 4.63 miles to the north branch of the Calaveras River; (3) enlargement of the Calaveras River to cross-sectional area of 1,550 square feet, thence to its mouth at San Joaquin River, 5 miles; and (4) a levee along the left bank of the diverting canal and Calaveras River, using material excavated for the channel enlargement.

Construction of new work was initiated in November 1908; the initial construction phase was completed in September 1910. No further new work was accomplished until fiscal year 1922; the project was completed in fiscal year 1923. Most of the silt formerly deposited in Turning Basin of the Stockton Deep Water Ship Channel is diverted by this canal, obviating serious inconveniences to navigation in the harbor area.

Federal maintenance of these channels for navigation purposes has been discontinued due to completion of levee and channel improvements constructed under provisions included in the Mormon Slough, Calaveras River, project authorized by the Flood Control Act of 1962 (Public Law 874, October 23, 1962, 87th Congress, 2d Session). No Federal maintenance costs have been incurred since Fiscal Year 1969. The project capacity was increased by the Mormon Slough project which was completed in 1971. The Mormon Slough project is described below.

c. Mormon Slough Project. The Mormon Slough project was authorized by the Flood Control Act of 1962 (Public Law 874, October 23, 1962, 87th Congress, 2nd Session). The project provides for the improvement of the Calaveras River system between the town of Bellota and the city of Stockton, California, and consists of minor channel enlargement of Mormon Slough between Bellota and Jack Tone Road; substantial channel enlargement of lower Mormon Slough and the Diverting Canal; new levees along the north bank of the Diverting Canal, along both banks of lower Mormon Slough, and along the south bank of Potters Creek between Jack Tone Road and Mormon Slough; and bank protection on lower Calaveras River levee. The Federal Levee along the right bank of the Calaveras River extends from the Diverting Canal downstream to the San Joaquin River. The project is an element of the comprehensive development of the Calaveras River basin, contains the flood flows which originate in the area downstream from New Hogan Reservoir and contains the flood control releases for efficient operation of that reservoir.

Preconstruction planning was initiated in January 1964. Construction was initiated in October 1967. Work was substantially completed in February 1970; remaining miscellaneous minor work was completed in December 1971. Project design flows are described in Table 4.

The project was extended with local funding by the San Joaquin Flood Control Agency (SJAFCA) to include levee modifications to achieve 3.3 feet above the median 1% (1/00) ACE water surface along Mormon Slough, Potter Creek, Upper Calaveras River, and Stockton Diverting Canal. Additional project works added include the following:

- Improvement of levees on both banks of the Mormon Slough upstream from the Stockton Diverting Canal to the confluence with Potter Creek. The right bank of Mormon Slough has been modified 400 feet upstream from its confluence with Potter Creek.

- Improvement of levee on left side of Potter Creek from Mormon Slough to Jack Tone Road.

- Improvements of levee on both sides of Stockton Diverting Canal from the Mormon Slough northwest to the confluence with the Upper Calaveras River. Intermittent floodwall construction was also included on the right bank along the same reach.

- Improvements of Levee on both sides of Upper Calaveras River from the junction with the Stockton Diverting Canal to the Central California Traction railroad tracks.

The above improvements to the authorized project were constructed from August 1997 to October 1998.

d. Farmington Dam and Reservoir. Farmington Dam was authorized by the Flood Control Act of 1944 (Public Law, 534, December 22, 1944, 78th Congress, 2nd Session). The project is located on Littlejohn Creek about 2.5 miles upstream from Farmington and about 18 miles east of Stockton, California and consists of an earthfill dam, maximum height 58 feet, and an ungated saddle spillway, creating a reservoir gross storage capacity of 52,000 acre feet (USACE,1974).

Also included in the Farmington project were appurtenant facilities for diverting Duck Creek floodwaters to Littlejohn Creek. However, several of the appurtenant features were later updated by the Little Johns Creek and Calaveras River Stream Group Project and the Duck Creek Project. All facilities are for the exclusive purpose of flood management.

The Duck Creek diversion is located about 0.5 miles east of Farmington California and approximately 3.5 miles downstream from Farmington Dam. The diversion works consist of a low compacted earth dike across Duck Creek with on 72” gated and one 60” ungated outlet discharging into Duck Creek, and an ungated concrete spillway 73 feet long discharging into the diversion channel. According to exhibit B of the operations and maintenance manual, the 72” gate is to remain fully open unless closure is authorized or directed by the District Engineer, Sacramento District, Corps of Engineers (USACE, 1952).

The Duck Creek Diversion Unit also includes dike "B" built across the North Branch of Duck Creek approximately 4 miles downstream from the diversion works; and dike "C" built across the North Branch of Duck Creek approximately 9 miles downstream from the diversion works and just upstream from Jack Tone Road.

Construction was initiated in July 1949; the main dam and spillway were completed in June 1951; the Duck Creek channel improvements were completed in November 1951; and the downstream improvements along Littlejohn Creek were completed in May 1955. Enlargement of the Duck Creek channel downstream of the diversion structure as part of the later Duck Creek Project was authorized under Public Law 685, 84th Congress, 2nd Session. The Duck Creek project is described below.

e. Bear Creek Project. The Bear Creek project is a small tributary of the Sacramento and San Joaquin Delta within the City of Stockton, San Joaquin County. The levee and channel improvements extend along the south channel of Bear Creek from Jack Tone Road about 2 miles south of Lockeford, to Disappointment Slough, a Delta channel which connects with the San Joaquin River. Completed construction provides for channel capacity of 5,500 cfs with 3 feet of freeboard. The project was authorized by the Flood Control Act of 1944 (Public Law 534, December 22, 1944, 78th Congress, 2nd Session). Advance planning on the project was initiated in Fiscal Year 1947 and suspended in Fiscal Year 1951 awaiting agreement with local interests regarding the plan of improvement. The project was classified as "Deferred" in Fiscal Year 1954. A review report was completed during Fiscal Year 1962. Construction was initiated during June 1963 and completed 20 July 1967.

Reclamation Board permits Nos. 15183 and 15214 permitted the diversion of Pixley Slough into Bear Creek and raising the Bear Creek levees to provide 3 feet of freeboard above the 100-yr flow (USACE, 2012). The levees were raised from the downstream end of the project upstream to the Western Pacific Railroad. The modification was completed in about 1990. SJFCA raised the Bear and Pixley levees in 1998.

f. Duck Creek Project. The Duck Creek Project is a small tributary of the San Joaquin River south of the City of Stockton, San Joaquin County, lying between the Calaveras River-Mormon Slough system and Littlejohn Creek. The Duck Creek channel extends from the Duck Creek Diversion (Unit of the Farmington Project) located about 0.5 miles northeast of Farmington California and meanders downstream a distance of about 20 miles to French Camp Slough. Authority to improve the Duck Creek channel was approved by the Chief of Engineers under the small flood control project program authorized by Section 205 of the 1948 Flood Control Act as amended by Public Law 685, 84th Congress, 2nd Session. The project works consist of channel improvements along approximately 20 miles of the Duck Creek channel from 1/2 mile upstream of Escalon-Bellota Road to French Camp Slough. The project includes a short reach of levee on the lower end of Duck Creek along the left and right banks. The design flows are 700 cfs from the Diversion Dam to Mariposa Road and 900cfs below the diversion dam. Construction of the project was initiated May 1965 and completed by January 1967. Project design flows are described in Table 4.

g. Lower San Joaquin River and Tributaries Project. Improvement of lower reaches of the San Joaquin River and Tributaries was authorized by the Flood Control Act of 1944 (Public Law 534, December 22, 1944, 78th Congress, 2nd Session), as modified by Public Law 327, 84th Congress, 1st Session). The project provided for improvement by the Federal Government of the existing channel and levee system on the San Joaquin River from the delta upstream to the mouth of Merced river, and on the lower reaches of the Stanislaus and Tuolumne Rivers, by raising and strengthening of existing levees, construction of new levees, revetment of river banks where required, and removal of accumulated snags in the main river channel. The project also provided for protection of flood plain areas about the mouth of Merced River through local interests construction of levee and channel improvements. The Upper Delta is defined roughly as that portion lying within the influence of flood flows while the lower Delta is that portion influenced mainly by tides. The line of demarcation is considered to be the downstream limits of the San Joaquin Flood Control Project and passes across the Delta from the confluence of the Stockton Deep water ship Channel and the San Joaquin River at the Port of Stockton, to Williams Bridge on Middle River, and to the junction of Paradise Cut and Salmon Slough with Grant Line Canal near Tracy.

The local interest plan of improvement was coordinated with that of the Federal Government to insure the effectiveness of the Federal portion of the projects. In addition to bearing the cost of improvements as required along the San Joaquin River upstream of the mouth of Merced River, Local interests were required for the Federal improvement downstream from Merced River, to furnish flowage rights to overflow certain lands along the San Joaquin River, to furnish all lands, easements, and rights-of-way for construction of improvement of levees; to accomplish all necessary utility alterations and relocations; to hold and save the United States free from damages due to the construction works and their subsequent maintenance and operation; and to maintain all levees and channel improvements after completion in accordance with regulations prescribed by the Secretary of the Army.

Federal construction was initiated in June 1956 and was completed in November 1968 except for the left bank levee along the San Joaquin River, Tuolumne to Merced River reach, which at that time was in the “inactive” category. This work was restored to “active” status on 25 June 1969 as required assurances of local cooperation for the reach were furnished after a change in land ownership. Contract for construction of this reach was initiated in November 1971 and completed in September 1972. The State of California has completed construction of the non-federal portion of the project above the mouth of the Merced River, comprising about 193 miles of new levees, including appurtenant features and about 80 miles of surfacing of existing levees. An evaluation of the project authority by the Sacramento District in July 2015 confirmed that San Joaquin River levees upstream of the Merced River are not part of the Federal Authorized Project (USACE, 2015).

The Federal Project levees within RD17 were improved by local interests as a part of the development of Weston Ranch in the City of Stockton. The purpose of the improvement project was to meet FEMA’s National Flood Insurance Program (NFIP) 1% (1/100) ACE floodplain regulatory requirements. FEMA accredited the levee as meeting the National Flood Insurance Requirements in February 1990.

h. Friant Dam (Millerton Lake). Friant Dam was authorized by the River and Harbor Act (Public Law No. 392) of August 26, 1937 (50 Stat. 850), and the River and Harbor Act of October 17, 1940 (ch 895, 54 Stat. 1198, 1199) extended the authorization to include irrigation distribution systems. The project is located about 25 miles northeast of Fresno and an equal distance east of Madera. It is a concrete gravity structure, 319 feet high and 3,488 feet long at the crest. The spillway is 332 feet wide and is located near the center of the dam. It has three 100 by 18-foot drum gates and a discharge capacity of 83,000 cfs at gross pool elevation.

Initial construction was started in October of 1939 and was completed in November 1942. Work deferred during the war, including spillway gates, outlet valves, Friant-Kern Canal stilling basin, etc., was again started in March of 1946 and the project was completed for operation in 1949.

i. Big Dry Creek Dam. Big Dry Creek Dam was authorized by the Flood Control Act of 1941 (Public Law 288, August 18, 1941, 77th Congress, 1st Session). The project is located about 10 miles northeast of Fresno, California, and about 4 miles northeast of Clovis, California and comprises and earthfill dam across the channel of Big Dry Creek, with a maximum height of 40 feet, creating a reservoir with a maximum capacity of 16,250 acre-feet, all for flood control, together with appurtenant diversion facilities both upstream and downstream from the dam. Construction of the project was initiated in April 1947 and completed in February 1948. Construction of remedial work consisting of erosion control structures to control side-hill erosion was initiated in October 1952 and completed in March 1955.

Modification of the Big Dry Creek Reservoir and Diversion project was included as one of five features that made up the Redbank and Fancher Creeks Flood Control Project in California. The Redbank and Fancher Creeks Flood Control project was authorized for construction on November 17, 1986 by the Water Resources Development Act of 1986. Modifications included raising the dam and spillway crest, constructing a new outlet works on Little Dry Creek and modification to the Big Dry Creek Outlet Works. Construction of the modifications was completed 22 August 1993 (USACE, 1994).

j. Camanche Dam. Federal participation in the construction of Camanche Dam was authorized by the Flood Control Act of 1960 (Public Law 86-645, 14 July 1960, 86th Congress, 2d Session). Camanche Dam and Reservoir is a multiple-purpose dam and reservoir on the Mokelumne River about 20 miles northeast of Stockton. The dam and reservoir was constructed by the East Bay Municipal Utility District which owns and operates the project facilities. Federal interest in the project is in the flood protection afforded by the dam and reservoir commensurate with the flood control benefits to be derived. The project comprises a rock fill dam with impervious earth core, maximum height 171 feet, together with six dikes totaling 19,250 feet in length and a gated spillway, creating a reservoir gross storage capacity of 431,500 acre-feet for flood control and water supply.

In consideration of the Federal contribution toward the first cost of Camanche Reservoir, the East Bay Municipal Utility District provides a flood-control reservation of 200,000 acre-feet, under an agreement with the Department of the Army providing for operation of the reservoir in such manner as will produce the flood-control benefits upon which the monetary contribution is predicated, and will operate the flood-control reservation in accordance with the rules and regulations prescribed by the Secretary of the Army.

The cost allocation for the project was approved by the President on 9 March 1962. Contract for Federal payment for flood control benefits to be attained was consummated 19 March 1962 with the East Bay Municipal Utility District and approved by the Secretary of the Army 19 April 1962. Contract for construction of the main dam and appurtenances was awarded in March 1962; dam closure was completed 7 November 1963. The project was operationally completed in April 1964.

k. Los Banos Dam. Los Banos Dam was authorized by the Central Valley Project, California Act of 1960 (Public Law 488, June 3, 1960, 86th Congress, 2nd Session) and was constructed by the US Bureau of Reclamation, with funds contributed in part by the Federal Government in the interest of flood control, and are operated by the State of California. The project is located on Los Banos Creek, a west side tributary to San Joaquin River, approximately seven miles southwest of the small city of Los Banos in Merced County, California and comprises of a earthfill dam, with a maximum height of 167 feet, creating a reservoir with a maximum capacity of 34,600 acre-feet, most of which is for flood protection, with a provision of a pool for recreation and other purposes. There is also an uncontrolled concrete chute spillway located in the left abutment of the dam with a discharge capacity of 8,600 cfs. Outlet works, including an intake structure, conduit, emergency gate, and control gates are located in the left abutment of the dam and discharge the water into a stilling basin which, in turn, empties into the existing channel of Los Banos Creek downstream from the structure. Construction of the project began in May 1964 and completed by November 1965.

l. New Exchequer Dam (Lake McClure). New Exchequer Dam was authorized by the Flood Control Act of 1960 (Public Law 645, July 14th, 1960, 86th Congress, 2nd Session). The project is located in the southern half of the Central Valley in Mariposa County, California. It is on the Merced River about 60 miles above its confluence with the San Joaquin River. New Exchequer Dam and Reservoir were constructed for the purposes of irrigation, power, recreation, and flood control. The reservoir includes a maximum of 400,000 acre-feet of flood control space. New Exchequer Reservoir has a capacity of 1,024,600 acre-feet. The dam is a rockfill dam, concrete faced with a height of 490 feet and is located immediately downstream from the old concrete Exchequer Dam, which is incorporated into the upstream toe of the embankment. A dike of similar gravel fill construction is located about ¾ of a mile northwest of New Exchequer Dam. A spillway, located approximately one mile northwest of the right abutment of New Exchequer Dam consists of a gated spillway and an ungated emergency spillway, each with a concrete ogee crest. The total combined discharge capacity of the gated and emergency spillways is 375,000 cfs. The outlet works consists of a single conduit under the right abutment of both the old and new portions of the dam. Construction of the project was initiated in June 1964 and completed in December 1967.

m. Don Pedro Dam. Don Pedro Dam was authorized by the Flood Control Act of 1944 (Public Law 534, December 22nd, 1944, 78th Congress, 2nd Session). The project is located on the Tuolumne River about 35 miles east of Modesto. The dam is a combination rock and earthfill dam with a maximum height of 585 feet and a total capacity of 2,030,000 acre-feet which is primarily to store irrigation water and has additional benefits including power generation, flood control, and recreation. A spillway located on the abutment ridge west of the dam, consists of both a gated spillway and an ungated emergency spillway, each with a long concrete ogee section. The total combined discharge capacity of the spillway is 472,500 cfs. The outlet works is located in a concrete plug centered approximately on the axis of the dam. Three separate parallel outlets are provided, each controlled by two high-pressure slide gates in tandem. The combined capacity of the three outlets is 7,370 cfs. Construction of the project was initiated in August 1967 and completed in March 1971.

n. Buchanan Dam (Eastman Lake). Buchanan Dam was authorized by the Flood Control Act of 1962 (Public Law 874, 23 October 1962, 87th Congress, 2d Session). The project provides for construction of a dam on Chowchilla River, about 16 miles northeast of the city of Chowchilla, California, to create a reservoir with gross storage capacity of about 150,000 acre-feet for flood control, irrigation, recreation, and other purposes. The project plan provides for approximately 20 miles of levee and channel improvements along Ash and Berenda Sloughs, distributaries of Chowchilla River. Construction of the project was initiated in June 1972 and completed in June 1978.

o. Hidden Dam and Lake. Hidden Dam and Lake was authorized by the Flood Control Act of 1962 (Public Law 874, 23 October 1962, 87th Congress, 2d Session). The project provides for construction of a dam on Fresno River, about 15 miles northeast of Madera, California, to create a reservoir with gross storage capacity of about 90,000 acre-feet for flood control, irrigation, recreation, and other purposes. The project plan as authorized also provides for approximately 13.3 miles of levee and channel improvements on Fresno River downstream from the damsite. Construction of the project was initiated in June 1972 and completed in June 1978.

p. New Melones Dam. New Melones Lake was authorized by the Flood Control Act of 1944 (Public Law 534. December 22, 1944. 78th Congress, 2d Session), as modified by the Flood Control Act of 1962 (Public Law 874, October 23, 1962, 87th Congress, 2d Session). The project is located on Stanislaus River, about 35 miles northeast of Modesto, California. The project plan provides for construction of a 625 foot high earth and rockfill dam to create a reservoir with a gross storage capacity of 2,400,000 acre-feet for flood control, irrigation, power, recreation, fish and wildlife and water quality control. The plan of improvement also includes construction of a 300,000 KW capacity hydroelectric power plant immediately below the dam. Construction of the project was initiated in 1966 and completed in October 1978.

2.5 Stream Gages.

A list of stream gages applicable to the study area is provided in Table 6. The stream gages are operated by the United States Geological Survey (USGS) and California Department of Water Resources (DWR). A more detailed description of stream gages is provided in the hydrology report.

Table 6 Stream Gages, Lower San Joaquin Study Area

Gage Name	Area (Sq Mi)	Agency	Gage Number	Type
San Joaquin River near Vernalis	13,539	USGS	11303500	S,Q
San Joaquin River at Mossdale	15,809	DWR	B95820	S,Q
San Joaquin River at Brandt Bridge	NA	DWR	B95740	S,Q
San Joaquin River below Garwood Bridge	16,177	USGS	11304810	S,Q
Stockton Ship Channel at Burns Cutoff	NA	DWR	B95660	S
Middle River at Borden Highway	NA	DWR	B95500	S
Middle River at Mowry Bridge	NA	DWR	B95540	S
Old River at Clifton Court Ferry	NA	DWR	B95340	S
San Joaquin River at Ringe Pump	NA	DWR	B95620	S
Grant Line Canal at Tracy Road Bridge	NA	DWR	B95300	S
Calaveras River blw New Hogan Dam	363	USACE	NHGQ	Q
Mormon Slough at Bellota	473	USACE	MRS	S,Q
Littlejohn Creek blw Farmington Dam	212	USACE	FRM	S,Q
Littlejohn Creek at Farmington	248	USACE	FRG	S,Q
Bear Creek near Lockeford	48	USGS	11312000	S,Q
Duck Creek Diversion near Farmington	28	USACE	DUC	S,Q
Duck Creek near Farmington	8	USACE	DCK	S,Q
S - Stage Q - Discharge				

2.6 Climate Change.

The primary impacts of climate change on Flood Risk Management projects are related to changes in sea level, changes in inland flood frequency estimates, and their associated uncertainties. These impacts were included in the analysis by assessing performance and economic analysis for existing (2010) and future (2070) climate conditions.

a. Sea Level Change. The downstream reaches of the study area are within the Sacramento and San Joaquin Delta and are subject to changes in sea level. Hydraulic analysis presented in this study for economic analysis was conducted for existing 2010 sea level conditions and for future conditions in the year 2070. The 2070 condition was selected because it is near the end of the economic period of analysis used for alternative evaluation. In addition, the year 2070 fulfilled the sponsor's objective of determining if the project meets the State of California's Urban Levee of Flood Protection requirements in 2070. The assumption had to be made early in the study, prior to estimates of the beginning and end years for economic analysis. The year used for the hydraulic analysis may not be identical to the economic assumption. However, the change in sea level between 2010 and 2015 is estimated to be only 0.07 feet and would not have a significant impact on the results.

The 2070 conditions used for economic analysis of the final array were based on the sea level trend described in Curve II of ER 1100-2-8162. Qualitative analysis conducted during the comparison of alternatives indicated the recommended plan were more sensitive to the flood risk associated with the riverine reaches which are not subject to Sea Level Change. Sea-level rise is expected to impact all alternatives equally, as each alternative in the final array includes identical improvements for the index points affected by sea-level rise. This was confirmed during sensitivity analysis to other Sea level Change rates for the recommended plan. The performance of the Recommended Plan to different levels of Sea Level Change (Low, Intermediate, and High) as described in ER 1110-2-8162 are described in Section 11 of this report. Additional details are provided in the description of the alternatives.

b. Inland Climate Change. Future changes in the Inland flood flow-frequency estimates related to climate change are less certain than changes in sea level. The flood risk assessment presented in this report is based on the estimates of flood frequency for existing and future without project conditions that do not account for inland climate change. A qualitative description of inland climate change is provided in the Hydrology Addendum. In summary, it is estimated that future peak flood flows may increase in the future and this would result in a gradual reduction in project performance over time.

3.0 FLOOD EVENTS

The frequency of observed historical floods is not directly comparable to each other or to existing conditions due to historical changes in the flood management system. Damage to the study area during most of the known past floods would have been significantly reduced if the floods had occurred with presently existing flood risk management facilities completed and in operation.

The San Joaquin River near Vernalis and Mormon Slough at Belota gages provide a record of large historical floods within the study area. The largest ten floods based on conditions that existed at the time of the flood are provided in Table 7. The largest ten San Joaquin River floods based on regulated conditions is provided in Table 8. Only flood events since 1979 were considered in Table 8 because completion of the last major reservoir project occurred in l979.

Unregulated estimates are useful in the evaluation of hydrologic frequency estimates because they are based on a similar basin condition throughout the record. The largest ten floods based on unregulated conditions from 1930 to 2014 are presented in Table 9. Hypothetical flows, based on unregulated conditions, represent the magnitude of floods without regulation. These are computed by adjusting observed flows to remove the effects of reservoir regulation, which has varied over time as reservoirs were constructed.

The largest flood since 1930 (assuming unregulated conditions) occurred in January 1997. The flood flow was the largest to have occurred since completion of major reservoir projects in 1979. It is estimated the 1997 flood would have been the largest flood since 1930 if the current reservoirs were in place by 1930. The December 1950 flood had a higher peak discharge. However the peak flow would have been less than the 1997 flood if reservoir projects had been completed at that time. A graph of historical floods on the San Joaquin River is provided as Plate 15.

The following are descriptions of significant flood events within the study area.

Table 7
Ten Largest Historical Flood Flows
WY1930-WY2014, San Joaquin River near Vernalis

Annual Ranking	Water Year	Date of Peak	Peak Flow (CFS)
1	1951	12/09/50	79000
2	1997	01/05/97	75600
3	1969	01/27/69	52600
4	1938	03/16/38	51200
5	1955	12/25/55	50900
6	1983	03/07/83	45100
7	1958	04/05/58	41400
8	1943	03/12/43	38900
9	1940	04/02/40	37300
10	1986	03/19/86	36900

Note: Floods prior to 1979 do not reflect existing reservoir regulation system.

Table 8
Ten Largest Floods since completion of Major Reservoir Projects
WY1979-WY2010, San Joaquin River near Vernalis

Annual Ranking	Water Year	Date of Peak	Peak Flow (CFS)	Annual Chance Exceedance
1	1997	01/5/1997	75600	1%
2	1983	3/7/1983	45100	3%
3	1986	3/19/1986	36900	6%
4	1998	2/13/1998	35200	10%
5	2006	4/13/2006	34800	13%
6	1980	2/27/1980	33900	16%
7	1984	01/06/1984	33000	20%
8	1982	04/18/1982	29800	23%
9	1995	3/19/1995	26100	27%
10	1996	03/10/1996	18000	30%

Table 9
Ten Largest Floods based on Unregulated Flow Conditions
WY1930-WY2014, San Joaquin River near Vernalis

Unregulated Condition						
Annual Ranking	Water Year	Date of Peak	1-Day Duration		3-Day Duration	
			1-Day Avg Flow (CFS)	Annual Chance Exceedance	3-Day Avg Flow (cfs)	Annual Chance Exceedance
1	1997	01/4/1997	219,100	1%	191,200	1.1%
2	1956	12/26/1955	187,800	2%	157,200	1.9%
3	1986	2/20/1986	156,600	3%	145,800	3%
4	1951	11/22/1950	135,400	4%	120,800	4%
5	1965	12/25/1964	115,000	6%	98,300	6%
6	1980	01/15/1980	112,300	6%	99,500	6%
7	1963	02/02/1963	101,500	8%	86,900	8%
8	1995	03/13/1995	100,900	8%	91,200	7%
9	1969	01/27/1969	94,400	9%	87,000	8%
10	1938	12/13/1937	90,800	10%	75,000	10%
Unregulated conditions are hypothetical conditions assuming no regulation by upstream reservoirs. Source: Sacramento and San Joaquin River Basins Comprehensive Study (March 2002) Annual Ranking based on average flow over 1-Day duration.						

a. Late 19th Century. Floods that occurred in 1861-62 were the most severe known during the last half of the 19th century. Flooding on the valley floor was deep enough to permit riverboats to reach almost any locality in the inundated area (USACE, 1975). The “Great Flood” of 1862 was remarkable for the exceptionally high stages reached on most streams, repeated large floods, and prolonged and widespread inundation in the San Joaquin Valley (SJAFCA, 2013).

b. Early 20th Century. The major floods that occurred in the earlier part of the 20th Century (March 1907, January 1909, January-February 1911, and January 1921) were all very similar on their impact on the study area (USACE, 1975). In the Calaveras system, flooding was widespread, frequently extending across the area between Mormon Slough and the Calaveras River in the vicinity of Linden, which was entirely flooded a number of times during the period (USACE, 1975). Subsequent to construction of the Stockton Diverting Canal in 1910, floodwater ponded on its north side and extended far to the north and east (USACE, 1975). In 1911 floodwater extended in a solid sheet west from the Southern Pacific crossing of Mormon Slough to the Diverting Canal, a distance of about 7 miles. During that flood the levee along the south side of the Diverting Canal was overtopped. During all the floods of the first quarter of the 20th century, the study area was frequently described as an inland sea (USACE, 1975).

c. February 1938. Completion of New Hogan Dam and Reservoir in 1936 had a tempering effect on flooding in the study area. A flood that would have reached major proportions was largely averted by the project in February 1938. Runoff was estimated to be the greatest since 1911, but detention of floodwater in the reservoir and opportune cold weather and snowfall in the mountains, which halted runoff, limited overflow in the study area to such an extent that only a few roads were closed at the Diverting Canal and flood damage was minimal (USACE, 1975). The 1938 flood on Bear Creek was severe and a large area was inundated in the vicinity of the Highway 99 crossing. Levees in the Delta breached on Mandeville, Quimby, Rhode, and Venice

Islands and Pescadero and Stewart Tracts. A total of about 21,000 acres were inundated. The 100-acre Rhode Island was never reclaimed. Franks Tract was flooded and never reclaimed (SJFCA, 2013).

d. December 1950. The December 1950 flood was the fourth largest unregulated peak flow recorded at the San Joaquin River at Vernalis Gage from 1930 to 2010. The following description of the December 1950 flood is provided in the reference USACE, 1975. A series of unusually severe storms from November 13 to December 8, 1950 resulted in extensive flooding in the study area in early December. Rainfall which extended to high elevations in the Sierra Nevada and melted most of the shallow snowpack, averaged 31.58 inches over the major tributary areas of the San Joaquin River and totaled 15 inches over the tributary areas of Littlejohns and Duck Creeks. Regulation of runoff to the lower San Joaquin River was such that flow was not exceptionally great in November. In early December, however, upstream reservoirs were nearly full or already spilling, and maximum releases were being made to maintain flood control space. The result was a record breaking 79,000 cubic foot per second flow at Vernalis on December 9. High flows, combined with the highest tides in 10 years, breached the east levee along the San Joaquin River and inundated a large part of Reclamation District 17. Ultimately, most of the study area west of Highway 50 (now Interstate 5) and French Camp road was inundated. Floodwaters remained on the land for as long as 2 weeks and were reported as 17.5 feet deep in the vicinity of Mossdale.

San Joaquin River floodwater inundated thousands of acres of prime farmland, forced the evacuation of about 2000 persons from rural residences, closed and severely damaged highways and roads, inundated the County Honor Farm and threatened the County Hospital. Flood damage totaled about $900,000 in Reclamation District 17. Agricultural losses (about 750,000) included damage to crop and pasture land by erosion, deposition of sand and debris, and weed infestations; damage to farmsteads, including irrigation facilities; destruction of livestock and poultry; increased cost of upkeep and operation, and the cost incurred for protection, evacuation, cleanup and reconstruction.

Calaveras River floodwaters did not contribute to flooding in the study area. Duck Creek overflow inundated residential areas on the edge of Stockton and forced the evacuation of about 300 families. Runoff from Littlejohns and Duck Creeks caused high flows in Walker and French Camp Sloughs where extensive sandbagging was required to prevent overflows and further inundation. Flow in French Camp Slough also threatened the County Hospital which was enclosed by a temporary ring dike, and ultimately protected from flooding by a cut made in the slough levee to prevent breaching or overtopping and flooding south towards the hospital.

The west levee of Paradise Cut breached, causing Delta flooding on the Pescadero Tract and the Stewart Tract, and washed out the Southern Pacific Railway tracks. Levees breached and flooded 3,220 acres on Venice Island and 5,490 acres on Webb Tract. (SJFCA, 2013).

e. December 1955. The December 1955 flood was the second largest unregulated peak flow recorded at the San Joaquin River at Vernalis Gage from 1930 to 2010. Photographs of 1955 flooding within the study area are provided in Plates 16 and 17. The following description of the 1955 flood is presented in the effective FEMA Flood Insurance Study. In December of 1955, approximately 1500 acres along Mormon Slough were inundated by floodwaters. Residential and commercial damage in Stockton amounted to $1,500,000. Damage to utilities and public facilities such as roads and streets totaled about $370,000. During the flood, 3000-3500 residents of Stockton were evacuated from their homes, traffic was severely interrupted and telephone service was disrupted. About $250,000 was spent to aid flood victims. The floodwaters remained in the city for as long as 8 days and reached a depth of 6 feet in some areas. In total, 125 city blocks were flooded; the most severely damaged area was south of Charter Way and east of French Camp Turnpike. The flood occurred prior to flood management improvements made to Calaveras River, Mormon Slough, Duck Creek, Littlejohn Creek, Farmington Dam, and the New Hogan Dam and Reservoir. Therefore, the flood does not reflect existing hydrologic conditions.

f. April 1958. The following description of the April 1958 flood was obtained from USACE, 1975. During the 1958 floods, runoff on the Calaveras River was the greatest experienced since 1911. Hogan Reservoir filled and spilled for the first time since its completion in 1936. In total, about 22,000 acres in the study area were flooded. Most of the area was farm, crop and orchard land except for some developing rural residential and commercial areas along Highway 99 and north of the Diverting Canal. About 3,000 acres of farmland in the vicinity of Linden were flooded by the Calaveras River where two levee breaks occurred. Linden was threatened but not damaged. Levees along Mormon Slough were breached in a number of locations and about 7,000 acres of land flooded in a strip extending from Bellota to the Diverting Canal. A major levee break occurred near the head of the Diverting Canal. Flooding also occurred on 1500 acres along the north side of the Diverting Canal. About 11,000 acres were flooded by Bear Creek; the areas inundated extended across the entire study area and ranged from about 3 miles wide in the upper portion to about 5 miles wide at Highway 99. Floodwaters averaged about 2 feet deep and remained on the land for 2-10 days in the Calaveras River portion of the study area. They reached a maximum depth of 3 feet and remained on the land for as long as 3 weeks in the Bear Creek portion.

g. December 1964-January 1965. Widespread flooding occurred in northern and central California and western Nevada in December 1964 and January 1965. Severe storms occurred over the watershed tributary to the study area. However flooding and flood damage was minimal because the levee and channel improvement project was nearly finished at the time and functioned effectively to prevent an estimated $500,000 damage to agricultural and suburban residential developments. Flood losses in the Bear Creek study area during the flood period consisted of minor damage to electrical utility facilities and cost of levee repair. New Hogan Lake, which became operational just prior to the flood season stored runoff from a moderate large flood and controlled flows downstream to non damaging amounts.

h. November 1982 - March1983. Water year 1983 was a result of the "El Niño" weather phenomenon. Northern and Central California experienced flooding incidents from November through March due to numerous storms. In early May, snow water content in the Sierra exceeded 230 percent of normal, and the ensuing runoff resulted in approximately four times the average volume for Central Valley streams. Reservoir releases into the Delta resulting in prolonged high waters over period of weeks with very high Spring Tide peaks. Venice Island subsequently failed on November 30th and Mildred and Shima Tracts in January. High Lower SJR flows in March from continuing rainfall and snowmelt led to flooding of RD2064 at the confluence of the Stanislaus and San Joaquin Rivers (SJFCA, 2013).

i. February 1986. Local runoff and releases from New Hogan Dam during the February 1986 flood produced a short duration peak of 16,700 cfs in Mormon Slough at Bellota (USACE, 1999). This flow exceeded the design capacity of 12,500 cfs by 4,200 cfs, but remained in the channel. New Hogan Dam held back the majority of the volume, preventing extensive flooding downstream. Without New Hogan Dam, peak flows at Bellota could have been as high as 40,000 cfs.

The peak flow at Bellota exceeded 12,500 cfs during the February 1986 flood because a portion of the release from New Hogan Dam contributed to the peak flows at Bellota before releases could be reduced to minimum flow. Releases ranged from 6,000 cfs several hours prior to the peak at Bellota to 2,000 cfs during the peak. (The travel time from the dam to Bellota is about three hours). However, the flows above 12,500 cfs occurred for only a very short duration and therefore no failures or major damages were experienced.

Since 1986, several improvements have benefitted flood control operation of New Hogan Dam. A real-time model of the river above Bellota was developed and a telemetered gage was installed on Cosgrove Creek, a tributary just downstream of New Hogan Dam. The real-time flow at the Cosgrove Creek location provides a good indication of timing and magnitude of downstream local flows.

j. January 1997. December 1996 was one of the wettest Decembers on record. Watersheds in the Sierra Nevada were already saturated by the time three subtropical storms added more than 30 inches of rain in late December 1996 and early January 1997. The third and most severe of these storms lasted from December 31, 1996, through January 2, 1997. Rain in the Sierra Nevada caused record flows that stressed the flood management system to capacity in the Sacramento River Basin and overwhelmed the system in the San Joaquin River Basin. Emergency releases from Friant and Don Pedro Dams occurred on the San Joaquin River system. RD 2095, 2058, 2107 & 2062 on the west bank of the San Joaquin River all flooded in 1997. Major flood fight efforts on Mokelumne and Lower San Joaquin Rivers with lesser event in the tidal Delta (SJFCA, 2013). Photographs of flooding upstream of RD17 are provided in Plate 10.

k. December 2005 - January 2006. Between 28 December 2005 and 9 January 2006, the State of California experienced a series of severe storms which impacted the levees within the Sacramento District's boundaries. Water rose a second time in April 2006, and remained high in some parts of the system until June. Many rivers and streams within the Sacramento and San Joaquin River systems ran above flood stage during these events, and there were significant erosion and seepage problems with the levees. The State of California Department of Water Resources and/or their maintaining agencies conducted the actual flood fight activities while the U.S. Army Corps of Engineers provided technical assistance to the State.

4.0 ALTERNATIVE 1 (No Action Plan)

4.1 Hydraulic Design Summary

The no action alternative is based on the without project conditions and does not include any proposed project features. The following describes the assumptions used to evaluate the existing conditions.

a. General Design. All project features in the no action plan are assumed to be the same as existed in 2014.

b. Levee Design Height. All existing levees are assumed to be maintained to the existing height or federally authorized height (federal project levees) whichever is higher. The design top of levee is based on the authorized design water surface profiles and the minimum freeboard specified in the Operations and Maintenance Manuals.

The San Joaquin River design water surface profiles are described in the drawing set, San Joaquin River and Tributaries Project, California, Levee Profiles, Drawing File Number SJ-20-30, 23 December 1955. The derivation of the 1955 water surface profiles is described in the general design memorandum. The 1955 design freeboard is described in the Operations and Maintenance manuals. The project adopted multiple existing levees of varying height. The Operations and Maintenance manuals indicates the adopted levee segments met or exceeded the design freeboard. The design levee height is referenced to the NGVD29 vertical datum which is unaffected by Sea Level Rise or subsidence. Therefore it is assumed the sponsors will maintain the levee profile and it will not degrade over time due to Sea Level Rise (SLR) or subsidence.

c. Upstream Reservoir Operation. The hydraulic analysis assumes all upstream reservoirs are operated as described in their respective water control manuals.

d. Interior Drainage Facilities. The hydraulic analysis assumes all drainage facilities are maintained to their design capacities.

e. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions.

f. Geotechnical Performance. The hydraulic analysis assumes the geotechnical performance is represented by the no action fragility curves presented in the geotechnical addendum to the feasibility study. The curves assess the probability of levee failure from under-seepage, through-seepage, stability, vegetation, animal burrows, encroachments, utilities, erosion, and judgment.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (*Design of I-Walls*, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The upstream end of the RD17 and French Camp slough tieback levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile rather than overtopped. The outflanking is considered to be a safer condition because it would occur only during the peak of the event and would reduce the flow and stage along the levee reaches.

h. Erosion Protection. The existing levee system includes erosion protection along several reaches. In most cases, this erosion protection appears to be along the lower 1/3 of levee height.

i. Diversion structures. The Mormon Slough and Duck Creek diversion structures are assumed to be operated as described in the operations and maintenance manual.

4.2 Hydrology

Hydrology for the San Joaquin River was based on analysis conducted by the California Department of Water Resources (DWR) and USACE for the 2002 Sacramento-San Joaquin Comprehensive Study. Hydrology for the Calaveras River and Mormon Slough was based on analysis conducted for the feasibility study between 2010 and 2014 by the Local Sponsors and USACE and followed procedures compatible with the California Department of Water Resources Central Valley Hydrology Study (CVHS). The following provides a summary of the hydrologic flow frequency analysis utilized as inputs to hydraulic analysis. The hydrology addendum provides additional details.

a. San Joaquin River. The upstream boundary for the San Joaquin River hydraulic model is the USGS stream gage San Joaquin River near Vernalis. The drainage area at the stream gage is 13,536 square miles. Records at the USGS stream gage only account for flow in the channel and do not account for overbank flow. During large floods, flow on the waterside of the right bank levee outflanks the gage before discharging into the main channel at the RD17 tieback levee. Hydrologic frequency analysis presented herein accounts for all flow passing the gage, including channel and right overbank flow.

The Sacramento-San Joaquin Comprehensive study included the entire Sacramento and San Joaquin Valleys. Balanced 30-day regulated flow hydrographs developed for 50% (1/2) Annual Chance Exceedance (ACE), 10% (1/10) ACE, 4% (1/25) ACE, 2% (1/50) ACE, 1% (1/100) ACE, 0.5% (1/200) ACE, and 0.2% (1/500) was used in the hydraulic analysis.

The synthetic hydrology investigated unregulated flood frequencies at mainstem and tributary locations throughout the San Joaquin Basin. The flood frequency analysis involved evaluations of long term historical records at the stream gages. The unregulated flow frequency statistics and period of record for the San Joaquin River near Vernalis were used to estimate hydrologic uncertainty for San Joaquin River reaches within the study area. The adopted statistics and period of record for the unregulated conditions are provided in Table 10. A tabulation of the flood frequency estimates for flood durations between 1-day and 30-days is provided in Table 11.

Table 10
Rain Flood Frequency Statistics, San Joaquin River near Vernalis
Unregulated Conditions

Flood Duration	Adopted Log Mean	Adopted Log Standard Deviation	Adopted Log Skew	Record (Years)	
				Years Evaluated	Years Used
1-Day	4.375	0.450	-0.1	1917 - 1998	82
3-Day	4.333	0.445	-0.1	1917 - 1998	82 (1/)
7-Day	4.251	0.433	-0.2	1917 - 1998	82
15-Day	4.148	0.412	-0.2	1917 - 1998	82
30-Day	4.042	0.392	-0.2	1917 - 1998	82
(1/) 82 year Equivalent Record adopted for use in FDA analysis					

Table 11
Flood Flow Frequency Estimates, San Joaquin River near Vernalis
Unregulated Conditions

Flood Duration	Duration Average Discharge by ACE (CFS)						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
1-Day	24100	88400	140300	188300	244700	310400	412900
3-Day	21900	79100	124900	167000	216500	273900	363100
7-Day	18400	62500	95200	124000	156500	193000	247300
15-Day	14500	46400	69200	89000	111100	135600	171700
30-Day	11400	34300	50200	63800	78700	95200	119200

The Comp Study formulated 5 mainstem and 22 tributary storm centerings to represent the many different possibilities of aerial storm distributions and antecedent watershed conditions. For each centering, synthetic 30-day natural flow hydrographs were computed at locations throughout the Central Valley. Typically, each tributary basin was composed of several hydrographs representing inflow to headwater dams, flood control dams, and local flow. The various hydrographs were then routed to specific index points to create an unregulated hydrograph (such as San Joaquin River at Vernalis). These natural flow hydrographs represent flood time series produced by a wholly unimpaired drainage area. The unimpaired hydrographs do not reflect the influence of headwater reservoirs. The hydrographs were balanced so the average flow for all durations matched the given frequency. For example, the peak, 1-day, 3-day, 5-day, 15-day, and 30-day volumes match the family of unregulated frequency curves computed for this location.

To simulate existing conditions, a 3-step process was required to conduct simulations of reservoir regulations for each storm centering. To begin the sequence, the headwaters reservoirs upstream of the flood control reservoirs were simulated. Then, using the resulting storage time series for select headwater facilities, top of conservation storage for those flood damage reduction projects with established credit space agreements were computed. Next, using the results of the headwater simulations and the computed top of conservation series, the lower basin reservoir models were simulated, thereby completing the reservoir simulation procedure.

A regulated set of hydrographs was obtained from "hand off" points in the lower basin reservoir simulation model. These hydrographs were then used as input to a UNET unsteady flow hydraulic model of the San Joaquin River. A review of the mainstem storm centerings found that the highest peak stages along the San Joaquin River within the study area are generated by the San Joaquin River at Vernalis storm centering. Therefore, hydraulic models for only one centering were evaluated in the feasibility study.

The sensitivity of downstream peak flows to upstream levee failures was conducted to determine if it would have a significant impact the evaluation of flood risk. The model was run for three different upstream levee failure scenarios.

- Infinite levee with no overtopping (Infinite). This is considered the extreme high estimate of peak flow and stage related to levee assumptions because no floodplain storage is allowed. All flow is confined to the leveed channel.

- Overtopping without Failure (No Fail). This model assumed all levees would overtop but would not fail. This may not be the most likely condition because some levees would likely fail prior to overtopping (probability of failure indicated by the fragility curve).

- With levee failure condition (With Fail). This model assumed all levees would fail at the 50% fragility point. This may not be the most likely condition because not all levees would fail at the 50% Fragility Point (FP) during the same flood.

A comparison of peak flows for the different levee overtopping assumptions is described in Table 12. The comp study models were only run for floods larger than 10% ACE.

Table 12
Sensitivity of Upstream Levee Failures, San Joaquin River near Vernalis
Regulated Conditions

Levee Scenario	Peak DIscharge by ACE (CFS)						
	50% ACE	**10% ACE**	**4% ACE**	**2% ACE**	**1% ACE**	**0.5% ACE**	**0.2% ACE**
Infinite Levee	NA	36900	47000	58400	90800	145500	233700
No Failure	NA	35100	42300	47700	78200	144500	224100
With Failure at 50% FP	NA	32900	43000	50300	77300	113300	166600
Source: 2002 Sacramento-San Joaquin Comprehensive Study UNET model results.							

The peak flow of infinite height assumption was found to always be greater for a given ACE event. The greatest difference between infinite height and no fail scenarios occurred at the 2% (1/50) ACE to 1% (1/100) ACE event which is probably around the flood magnitude that most system levees are overtopped. The No-Fail and With-Fail conditions are similar for floods smaller than 1% (1/100) ACE. The No-fail is larger than the with-fail condition for floods larger than 1% (1/100) ACE. The peak flow of the No-Fail is smaller than the With-Fail scenario for 4% (1/25) ACE and 2% (1/50) ACE events which appears counter intuitive. Model results indicate this is influenced by the storage within floodplain. Whereas overtopping floodwaters might be stored in the floodplain for the No-Fail condition, they are allowed to flow back into the channel through a breach in the With-Failure Scenario. The most likely condition is probably between the no-fail and with-fail conditions. The with-failure scenario also describes the relatively small influence that upstream transitory storage would have on reducing peak flows within the study area for floods as large as a 1% (1/100) ACE.

The overtopping with no failure scenario for areas outside the project area was adopted as the most likely hydraulic condition for this study to support the risk analysis. The probability of overtopping levee failure within the study area is accounted for in the FDA model using a fragility curve that assumes 100% failure probability at the levee crest. This assumption helps make a breach probability more statistically independent rather than dependent on each other and is consistent with historical observations that the probability of a breach does not appear to be highly dependent on other breaches occurring. There is no specific guidance on how to apply overtopping assumptions to system wide risk analysis. However, the approach taken is consistent with the other risk and uncertainty assumptions in the FDA model developed following the procedures in EM 1110-2-1619. The overtopping without failure assumption for areas outside the project area is also consistent with the DWR Urban Levee Design Criteria and FEMA mapping approaches.

A table of adopted regulated peak flows for this study is provided in Table 13. Due to upstream conditions, hydrographs for channel and right overbanks are required for events greater than a 1% (1/100) ACE event. A period of record of 82-yrs should be utilized in performance analysis to account for uncertainty in estimating the unregulated flow at Vernalis. A plot of the resulting flood frequency estimates and historical regulated flows is provided as Plate 18.

Table 13
Flood Flow Frequency Estimates, San Joaquin River near Vernalis
Regulated Conditions

Peak Flow	Peak Discharge by ACE (CFS)						
	50% ACE	**10% ACE**	**4% ACE**	**2% ACE**	**1% ACE**	**0.5% ACE**	**0.2% ACE**
Channel	6400	35100	42300	47700	78200	124600	165200
Right Overbank	0	0	0	0	0	20400	60500
Total	6400	35100	42300	47700	78200	144500	224100
Note: Time of peak channel flow is different than time of peak overbank flow. As a result, the peak total flow is not equal to the sum of the channel peak flow and overbank peak flow.							

The California Department of Water Resources is currently conducting a study of Central Valley Hydrology. The Central Valley Hydrology Study (CVHS) will provide more recent hydrologic frequency estimates throughout the study area. However, the results were not finalized at the time of this study. The draft flood frequency estimates from the CVHS study were compared to the comp study estimates and found to be similar.

b. Calaveras River and Mormon Slough. The upstream hydraulic model boundary for and Calaveras River and Mormon Slough is the USACE stream gage Mormon Slough at Bellota. The drainage area at the gage is 470 square miles. Hydrologic analysis is described in the hydrology addendum dated April 2014. Flood frequency curves and a suite of 10-day hydrographs were developed for the Mormon Slough at Bellota gage. The unregulated frequency analysis was performed with PeakfqSA software which uses the Expected Moments Algorithm (EMA) and Multiple Grubbs Beck outlier test. The method is approved for use by HQ USACE. The period of record analyzed is 104 years from 1907 to 2010. Unregulated flow frequency statistics for the Mormon Slough at Bellota Gage are provided in Table 14. Unregulated discharges by frequency and duration are provided in Table 15.

Table 14
Rain Flood Frequency Statistics, Mormon Slough at Bellota
Unregulated Conditions

Flood Duration	Adopted Log Mean	Adopted Log Standard Deviation	Adopted Log Skew	Record (Years)	
				Years Evaluated	Years Used for Statistics
1-Day	3.775	0.482	-0.810	1907 - 2010	104 (1/)
3-Day	3.608	0.475	-0.753	1907 - 2010	104
7-Day	3.417	0.464	-0.666	1907 - 2010	104
15-Day	3.240	0.461	-0.671	1907 - 2010	104
30-Day	3.079	0.448	-0.668	1907 - 2010	104
(1/) To account for local inflow uncertainty, 52 year Equivalent Record adopted for use in FDA analysis					

Table 15
Flood Flow Frequency, Mormon Slough at Bellota
Unregulated Conditions

Flood Duration	Duration Average Discharge by ACE (CFS)						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
1-Day	6900	21700	29700	35300	40500	45400	51300
3-Day	4600	14600	20200	24200	28000	31600	36100
7-Day	2900	9300	13000	15800	18500	21100	24500
15-Day	2000	6100	8600	10300	12100	13800	16000
30-Day	1300	4100	5700	6800	7900	9000	10400

The analysis involved routing scaled versions of four large historic flood events (reservoir inflow plus local flow hydrographs) through an HEC-ResSim reservoir routing model. Four unregulated to regulated transforms were derived and then averaged to produce a final adopted peak regulated flow frequency curve. Selected regulated hydrographs at Bellota based on the 1997 flood pattern and matching the regulated peak flow frequency curve were adopted for input into HEC-RAS model for modeling specific frequency events at Bellota. A rainfall runoff model was used to derive concurrent local flow hydrographs as internal boundary conditions in the HEC-RAS hydraulic model reaches downstream of Mormon Slough at Bellota. A table of adopted regulated peak flows for this study is provided in Table 16. Although the frequency analysis utilized 104 years of record, an equivalent period of record of 52-yrs should be utilized in performance analysis to account for uncertainty in estimating the ungaged unregulated flow between New Hogan Dam and Bellota. It was reduced in half because of uncertainty about how efficiently the dam can operate to local flow conditions. This equivalent record was also adopted for multiple index points downstream of Bellota since approximately 75% or more of the total flow in the downstream levee reach is from sources upstream of Bellota. A plot of the resulting flood frequency estimates and historical regulated flows is provided as Plate 19.

Table 16
Flood Flow Frequency, Mormon Slough at Bellota
Regulated Conditions

	Duration Average Discharge by ACE (CFS)						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
Peak Flow	3520	9530	10640	12500	12500	12500	16000

c. Delta Stage-Frequency. A stage frequency analysis was conducted at four stage gages in the Sacramento-San Joaquin Delta that serve as downstream boundary conditions in the hydraulic models. The stage-frequency analysis was conducted for DWR stream gages; Old River at Clifton Court Ferry (B95340), Middle River at Bowden Highway (B95500), San Joaquin River at Ringe Pump (B95620), and Stockton Ship Channel at Burns Cutoff (B95660) . Stage-frequency estimates were developed for future sea level conditions including 2010 and 2070. The frequency analysis is described in detail in the USACE Memorandum for File, Delta Stage-Frequency Analysis for Alternative Comparisons, 9 May 2014 (USACE, 2014A). The stage frequency curves are provided as Plate 20 and Tables 17 and 18. A map of the study area showing gage locations is presented in Plate 21.

The stage frequency analysis was based on stage data from the period from 1953 to 2009. Historical peak stages would have been higher under existing (2010) sea level conditions. Historical stage data were adjusted to 2010 sea level conditions for use in the frequency analysis. Each data set was adjusted by increasing historical recorded elevations to 2010 conditions using the eustatic rate of sea level rise of 0.0056 ft/yr (1.7mm/yr). The rate of eustatic sea level rise was obtained from ER 1100-2-8162 and agrees with the reported value in NOAA, 2013 as the estimated rate of sea level rise over the 20th century.

Graphical stage-frequency curves were developed for each gage by plotting the historical stage records using Weibul plotting positions. Extrapolation of the stage frequency curves from 2% ACE to 0.2% ACE events was based on hydraulic model simulations of the San Joaquin River system. For larger flood events the stage-discharge relationship at each gage was based on DSM2 model results presented in the March 2002 report "Sacramento and San Joaquin River Basins Comprehensive Study, Existing Hydrodynamic Conditions in the Delta during Floods". These relationships between stage and flow at each gage site are currently the best available analysis of hydraulic conditions in the delta for extreme flood events. While suitable for economic analysis, estimates should be refined for design purposes.

Future Sea level Rise was computed following the method outlined in ER 1100-2-8162 for four scenarios. The USACE Low estimate is based on the historical rate of sea level rise. The USACE Intermediate estimate is based on Curve I and reflects an intermediate estimate of the future rate of sea level rise. The USACE Curve II estimate reflects a rate greater than the intermediate rate. The USACE high estimate is based on curve III and reflects a high estimate of the future rate of sea level rise. Estimated increases in sea level for each scenario are provided in Table 19.

The Curve II rates were used to estimate future increases in sea level over the period 2010 through 2070 in the economic analysis. The Curve II rate is higher than the intermediate rate and was selected for these four locations considering the uncertainty and consequences of flooding in a highly urbanized area. As described below, stages at the boundary locations are based on a combination of flow and tide elevations and increased flow could further increase the stage at these index points. Estimates of potential inland climate change are described in the Hydrology Addendum. Future sea level rise was assumed to impact all flood frequencies the same amount because the Delta consists of a network of channels that would have similar hydraulic characteristics for higher sea level conditions.

All elevations presented in this report are provided relative to the NAVD88 vertical datum as required by ER1110_2_816. The NAVD88 datum is maintained by the National Geodetic Survey (NGS) to be free from changes related to subsidence and plate tectonics. The NAVD88 datum reflects a constant geopotential surface which is the basis for hydrodynamic and hydraulic modeling, design, and construction. Future rates of Sea Level Rise relative to the NAVD88 datum were based on the changes of Sea level.

Elevation changes related to subsidence or plate tectonics are accounted for as a reduction in the ground elevation relative to the NAVD88 datum over time. This approach is considered to provide a more accurate assessment of the impacts of sea level rise and localized ground subsidence than application of relative sea level rise estimates derived from nearby tidal gages which do not reflect the soil conditions underlying the proposed levee locations. In addition, the hydraulic effects of stage increases are different than subsidence in hydraulic models used to derive stages throughout the study area.

The subsidence component was estimated by reviewing NAVD88 elevations for three NGS benchmarks along the San Joaquin River within the study area. All three benchmarks indicated a similar amount of vertical change from the period 1998 to 2012. It was assumed the vertical change was due to subsidence but the value is also within the expected range of uncertainty in height modernization surveys conducted by the NGS over the period. In other words the differences might not be related to subsidence. All three benchmarks indicated a subsidence rate of about 0.02 feet (0.6 mm) per year during the period 1998 to 2012. This was considered to be a high estimate of the subsidence rate and would translate to about 2 feet of subsidence over 100 years. It is also important to consider that the observed differences at the NGS benchmarks may not reflect conditions for all features throughout the study area. For example, bridges are typically set on pile foundations that may not subside at the same rate as the natural ground and that might be different than a levee. It was assumed that a reasonable medium rate would be 0.01 feet per year and a low rate would be 0.005 feet per year.

For all alternatives it was assumed the design elevation would be maintained by the sponsor through normal operation and maintenance activities over the 100 year project life. As part of Operation and Maintenance the sponsor would be required to verify the crest elevation by conducting a high order survey every 10-years to update the National Levee Database. The sponsor would be required to restore the levee profile if it was found to have subsided more than 0.5 feet. This approach to addressing subsidence related issues is described as the "Managed adaptive approach" in ETL 1100-2-1. To support PED analysis, it is recommended that the National Levee Database Survey be re-conducted and compared to confirm the assumption of levee subsidence rates in the project area. This information would then be incorporated into the settlement portion of the design, or addressed in operations and maintenance. It is estimated the crest elevation would need to be restored every 25 years for reaches that subsided at the high rate and 50 years for reaches that subsided at the medium rate. No restoration would be anticipated for reaches that subsided at the low rate.

Table 17
Mean Stage estimates by Annual Chance of Exceedance, No Action Alternative
2010 Sea Level Conditions

ACE	Mean Stage (Feet-NAVD88)			
	Old River at Clifton Court Ferry (B95340)	Middle River at Borden Hwy (B95500)	Stockton Ship Channel at Burns Cutoff (B95660)	San Joaquin River at Ringe Pump (B95620)
0.002 (1/500)	13.08*	11.20*	13.01*	12.91*
0.005 (1/200)	12.12*	9.90*	12.12*	12.02*
0.010 (1/100)	11.44*	9.80*	10.10*	10.00*
0.020 (1/50)	9.95	9.57	9.90	9.80
0.040 (1/25)	9.75	9.50	9.70	9.60
0.100 (1/10)	9.35	9.10	9.30	9.20
0.200 (1/5)	8.70	8.55	8.70	8.60
0.300 (1/3)	7.70	7.80	8.15	8.05
0.500 (1/2)	7.15	7.25	7.70	7.60
0.950 (1/1.05)	6.35	6.45	6.70	6.60
* Stage estimates for events larger than 0.02 (1/50) ACE are based on hydraulic model extrapolation. While suitable for economic analysis, estimates should be refined for design purposes.				

Table 18
Future Stage estimates by Annual Chance of Exceedance, No Action Alternative Curve II Rate of Sea Level Change

ACE	Stage (Feet-NAVD88)			
	Old River at Clifton Court Ferry (B95340)	Middle River at Borden Hwy (B95500)	Stockton Ship Channel at Burns Cutoff (B95660)	San Joaquin River at Ringe Pump (B95620)
Sea Level Conditions in Year 2020				
0.002 (1/500)	13.24*	11.36*	13.17*	13.07*
0.005 (1/200)	12.28*	10.06*	12.28*	12.18*
0.010 (1/100)	11.60*	9.96*	10.26*	10.16*
0.020 (1/50)	10.11	9.73	10.06	9.96
0.040 (1/25)	9.91	9.66	9.86	9.76
0.100 (1/10)	9.51	9.26	9.46	9.36
0.200 (1/5)	8.86	8.71	8.86	8.76
0.300 (1/3)	7.86	7.96	8.31	8.21
0.500 (1/2)	7.31	7.41	7.86	7.76
0.950 (1/1.05)	6.51	6.61	6.86	6.76
Sea Level Conditions in Year 2030				
0.002 (1/500)	13.45*	11.57*	13.38*	13.28*
0.005 (1/200)	12.49*	10.27*	12.49*	12.39*
0.010 (1/100)	11.81*	10.17*	10.47*	10.37*
0.020 (1/50)	10.32	9.94	10.27	10.17
0.040 (1/25)	10.12	9.87	10.07	9.97
0.100 (1/10)	9.72	9.47	9.67	9.57
0.200 (1/5)	9.07	8.92	9.07	8.97
0.300 (1/3)	8.07	8.17	8.52	8.42
0.500 (1/2)	7.52	7.62	8.07	7.97
0.950 (1/1.05)	6.72	6.82	7.07	6.97
Sea Level Conditions in Year 2070				
0.002 (1/500)	14.74*	12.86*	14.67*	14.57*
0.005 (1/200)	13.78*	11.56*	13.78*	13.68*
0.010 (1/100)	13.10*	11.46*	11.76*	11.66*
0.020 (1/50)	11.61	11.23	11.56	11.46
0.040 (1/25)	11.41	11.16	11.36	11.26
0.100 (1/10)	11.01	10.76	10.96	10.86
0.200 (1/5)	10.36	10.21	10.36	10.26
0.300 (1/3)	9.36	9.46	9.81	9.71
0.500 (1/2)	8.81	8.91	9.36	9.26
0.950 (1/1.05)	8.01	8.11	8.36	8.26
Sea Level Conditions in Year 2120				
0.002 (1/500)	17.38*	15.50*	17.31*	17.21*
0.005 (1/200)	16.42*	14.20*	16.42*	16.32*
0.010 (1/100)	15.74*	14.10*	14.40*	14.30*
0.020 (1/50)	14.25	13.87	14.20	14.10
0.040 (1/25)	14.05	13.80	14.00	13.90
0.100 (1/10)	13.65	13.40	13.60	13.50
0.200 (1/5)	13.00	12.85	13.00	12.90
0.300 (1/3)	12.00	12.10	12.45	12.35
0.500 (1/2)	11.45	11.55	12.00	11.90
0.950 (1/1.05)	10.65	10.75	11.00	10.90

* Stage estimates for events larger than 0.02 (1/50) ACE are based on hydraulic model extrapolation. While suitable for economic analysis, estimates should be refined for design purposes.
Future Sea Level based ER 1100-2-8162 Curve II. Low, Intermediate, and High estimates can be computed using values in Table 19.

Table 19
Sea Level Rise and Ground Subsidence from 2010 Conditions

Year	Sea Level Rise from 2010 Conditions (Feet)				Potential Ground Subsidence from 2010 Conditions (Feet)		
	USACE Low (Historic)	USACE Intermediate Curve I	**Adopted Curve II**	USACE High Curve III	Low	Medium	High
2010	0.00	0.00	**0.00**	0.00	0.00	0.00	0.00
2015	0.03	0.05	**0.07**	0.10	0.03	0.05	0.10
2020	0.06	0.10	**0.16**	0.23	0.05	0.10	0.20
2025	0.08	0.15	**0.26**	0.37	0.08	0.15	0.30
2030	0.11	0.21	**0.37**	0.53	0.10	0.20	0.40
2035	0.14	0.28	**0.49**	0.70	0.13	0.25	0.50
2040	0.17	0.34	**0.62**	0.90	0.15	0.30	0.60
2045	0.20	0.42	**0.77**	1.12	0.18	0.35	0.70
2050	0.22	0.49	**0.92**	1.35	0.20	0.40	0.80
2055	0.25	0.58	**1.09**	1.60	0.23	0.45	0.90
2060	0.28	0.66	**1.27**	1.87	0.25	0.50	1.00
2065	0.31	0.75	**1.46**	2.16	0.28	0.55	1.10
2070	0.34	0.85	**1.66**	2.47	0.30	0.60	1.20
2075	0.36	0.95	**1.87**	2.80	0.33	0.65	1.30
2080	0.39	1.05	**2.09**	3.14	0.35	0.70	1.40
2085	0.42	1.16	**2.33**	3.50	0.38	0.75	1.50
2090	0.45	1.27	**2.58**	3.89	0.40	0.80	1.60
2095	0.47	1.39	**2.84**	4.29	0.43	0.85	1.70
2100	0.50	1.51	**3.11**	4.71	0.45	0.90	1.80
2105	0.53	1.64	**3.39**	5.14	0.48	0.95	1.90
2110	0.56	1.77	**3.68**	5.60	0.50	1.00	2.00
2115	0.59	1.90	**3.99**	6.07	0.53	1.05	2.10
2120	0.61	2.04	**4.30**	6.57	0.55	1.10	2.20
Rate of Sea Level Rise based on ER 1100-2-8162.							

d. Interior Drainage. An interior drainage analysis was performed by Peterson-Brustad Incorporated (PBI) for Bear Creek, Mosher Creek, and French Camp Slough sub-basins impacting the study area. A storm centered over the urban area of Stockton was utilized for the analysis. The interior drainage analysis evaluated rainfall runoff and flood depths for 50% (1/2) ACE through 0.2% (1/500) ACE flood events. Storm events with 72-hour durations were evaluated. The analysis utilized an HEC-HMS model to compute sub basin runoff and a FLO-2D two dimensional hydraulic model to route the runoff through the study area. The results indicated that residual damages from interior drainage would not influence alternative selection and would not meet the 800cfs rule. In addition, the analysis indicated that damages from interior drainage are negligible in comparison to flooding from the principle sources of flooding described in this report and improvements would not be cost justified. Therefore, interior drainage was not examined in detail for this study.

The effects of sea level change are estimated to have minor impacts on interior drainage because the affected interior drainage area is currently drained by a system of detention ponds and pumps that discharge the water to the adjacent delta sloughs. Increases in sea level are likely to result in a gradual increase in pump head and a corresponding reduction in the performance of the pumps over time. However, any increase in pump sizes necessary to maintain similar capacity is unlikely to be economically justified and are likely to be addressed by the local community as the pumps reach the end of their normal service life.

4.3 Hydraulic Models

Four separate hydraulic models, adapted from existing hydraulic models, were utilized to evaluate the no action plan for this study. Water surface profiles for the San Joaquin River were computed using an HEC-RAS unsteady one-dimensional flow model (version 4.1.0) of the San Joaquin River system. The model extents are shown on Plate 21. Water surface profiles for Calaveras River and Mormon Slough were computed using an HEC-RAS (version 4.1.0) unsteady flow model of the system. The model extents are shown on Plate 22.

Flooding was only modeled for breach locations impacting the economic impact areas shown in Plate 4. The selection of the breach locations was based on analysis conducted during plan formulation screening. The breach locations were selected to single out the primary sources of comingled flooding within the study area. Flood risk to areas outside these economic impact areas was found unlikely to support federal interest. The selection of the study area is described in the Feasibility Study report. Levee breach simulations for the area North of French Camp Slough were conducted using the North FLO-2D model shown on Plate 23. Levee breach simulations for the area south of French Camp Slough were conducted using the south FLO-2D model and are shown on Plate 24. Both FLO-2D models utilized version 2009.06 build number 09-13.05.13.

The computer model HEC-RAS calculates steady or unsteady gradually varied flow in natural and manmade channels by performing step-backwater calculations of the 1-D flow energy equation through a series of input geometric cross-sections with empirically defined hydraulic roughness coefficients. The computer model FLO-2D is a 2-dimensional, dynamic flood routing model that simulates movement of water across the ground surface while reporting volume conservation. It numerically routes flood hydrographs over a system of grid elements, and predicts the area of inundation and flood wave attenuation.

Without project conditions were evaluated using an uncoupled 1-d and 2-d modeling approach that has been standard procedure on multiple studies within the Sacramento District. River stages and profiles and breaches were simulated using an HEC-RAS model because RAS incorporates more detailed hydraulic capabilities for channel flow and breaches. The breach outflow hydrographs were then transferred to a 2-dimensional FLO-2D model of the floodplain. The FLO-2D model has more detailed capabilities than HEC-RAS for simulating the distribution of the breach hydrographs on the floodplain. This process leverages the most robust capabilities of both models.

a. San Joaquin River. Water surface profiles and breaches for the San Joaquin River were computed using an HEC-RAS (version 4.1.0) unsteady one-dimensional flow model of the San Joaquin River system. The origin of the model was the HEC-UNET model developed as part of the 2002 comp study. The model was updated to HEC-RAS by the California Department of Water Resources for use in Task Order 120 (TO120) of the Central Valley Flood Protection Plan (CVFPP). The model was updated to address the needs of the feasibility study. The primary updates were to extend the model downstream to three stage gages in the Sacramento San Joaquin Delta and truncate the upstream end of the model at the Vernalis gage. A map of the HEC-RAS hydraulic model domain is provided as Plate 21. A detailed description of the changes made to the model is provided in the Technical Memorandum, San Joaquin River Main Stem HEC-RAS model setup by Peterson Brustad Incorporated, 13 September 2013 (PBI, 2013A).

(1) Cross Sections. The model contains a total of 530 cross sections. The cross sections are spaced at roughly ¼-mile intervals along the river reaches. Cross section geometry data were obtained from the 2002 Sacramento-San Joaquin Comprehensive Study and updated to the NAVD88 datum using conversion values in the NGS Vertcon computer program.

(2) Storage Areas. The model contains a total of 31 storage areas throughout the domain.

(3) Bridges and Inline Structures. The model contains a total of 25 bridges, 1 inline structure and 1 major weir diversion (Paradise Dam).

(4) Lateral Structures (Levees). The HEC-RAS model utilizes the lateral weir option to simulate overtopping of the levee crest. The structures were manually coded into each HEC-RAS model based upon Top of Levee (TOL) elevation data from the USACE National Levee Database (NLDB) survey data. The lateral structure outflow is linked to the storage areas described above.

(5) Blocked Obstructions. Blocked obstructions were used throughout the model to eliminate the cross section area on the landward side of the levee. The landward areas are modeled as storage areas and lateral weirs along the crest of the levee control the flow over and into and out of the storage areas. The blocked obstructions are needed because the cross sections extend approximately 100 feet landward of the levee and this is not a conveyance area under this approach. The levee card is not suitable in this case because the conveyance area on the landward side of the cross section would incorrectly become conveyance area once overtopped. The heights of the blocked obstructions were made sufficiently high to insure the levee overtopping was consistent with the lateral structure levee approach described above.

(6) Ineffective Flow Areas. Ineffective flow areas were incorporated into the model to simulate areas where water is stored, but is not an active conveyance area.

(7) Manning's Roughness Values. Manning's n-values provided in the source model by DWR were adopted for this study. The model calibration is described in the DWR documentation described above. Values were selected based on model calibration to high water marks collected during the March 1995 event. Boundary condition inflows for the model

calibration were based on DWR and USGS stream gage records. Manning's roughness values range from 0.035 to 0.058 in the main channel and 0.042 to 0.110 in the overbanks.

(8) Upstream Boundary Conditions. Upstream boundary conditions are a set of regulated flow hydrographs for the Channel and Right Overbank at Vernalis. The channel and right overbank flow split were obtained from the 2002 Sacramento-San Joaquin Comprehensive Study UNET model.

(9) Downstream Boundary Conditions. The model includes three downstream stage-discharge rating boundary conditions; 1) Old River at Clifton Court Ferry 2) Middle River at Bowden Bridge, and 3) Stockton Deep Water Ship Channel at Burns Cutoff . The stage-discharge rating curves were developed through an initial set of model runs. For each ACE flow event a constant stage with the same ACE stage was set at each of the downstream boundary conditions. The system model was then run to determine the peak computed flow at each downstream boundary for the ACE event. The resulting peak stage and peak flow formed an ordinate of the final stage-discharge curve. This process was repeated for 50% ACE through 2% ACE events.

For larger flood events the stage-discharge relationship at each gage was based on DSM2 model results presented in the March 2002 report "Sacramento and San Joaquin River Basins Comprehensive Study, Existing Hydrodynamic Conditions in the Delta during Floods". DWR developed DSM2 based on the USGS's FourPt model for hydrodynamics and Branch Lagrangian Transport Model for water quality. DSM2 can calculate water stage, flow, and velocity in the Delta waterways under tidal influences and local consumptive use, CVP-SWP operations, and flow management operations for ecosystem protections. These hydrodynamic results facilitate the evaluation of mass transport processes for salts, non-conservative constituents, temperature, THM formation potential and individual particles. The portion of DSM2 used in the Comprehensive Study is the hydrodynamic module.

The modeling area of DSM2 includes all areas in the legal Sacramento-San Joaquin Delta. DWR has completed a re-calibration for DSM2 in year 2000 through an IEP effort to incorporate major upgrades in model resolution, data management, and utility features. The flow boundaries at the following locations: San Joaquin River at Vernalis, Sacramento River at I Street, Yolo Bypass at Shag Slough, Cosumnes River at Franklin Road, Mokelumne River at Franklin Road, Calaveras River at San Joaquin River. At the downstream end, DSM2 uses the tide stages at Martinez as the downstream boundary conditions. DSM2 also incorporates the consumptive use in the Delta, and the exports of the Central Valley Project and the State Water Project.

These relationships between stage and flow at each gage site are currently the best available analysis of hydraulic conditions in the delta for extreme flood events. The resulting combined stage-discharge relationships define the downstream boundary conditions of the hydraulic model.

The development of the stage-frequency curves is described in the hydrology section above. Models were developed assuming 2010 and 2070 sea level conditions at the downstream boundary condition.

(10) Model Calibration. The model was calibrated to the March 1995 flood event. Details on the model calibration are provided in DWR, 2009. A comparison of model results to the USGS stream flow measurements at the San Joaquin River near Vernalis gage is provided on Plate 22.

(11) Stage Uncertainty. The total SD of stage uncertainty was computed at the four index points along the San Joaquin River. A SD of 1.5 feet is recommended for all reaches of the San Joaquin River.

Stage uncertainty was estimated following methods described in EM-1110-2-1619. The total stage uncertainty was estimated from natural and model uncertainty. A detailed description of the stage uncertainty analysis is provided in the 13 September 2013 Technical Memorandum San Joaquin River Main Stem HEC-RAS modeling by Peterson Brustad Inc. (PBI, 2013A). The standard deviation (SD) of total stage uncertainty was calculated using Equation 5-6 of EM 1110-2-1619.

$$\mathrm{SD}_{\mathrm{total}} = \sqrt{\mathrm{SD}^2_{\mathrm{natural}} + \mathrm{SD}^2_{\mathrm{model}}}$$

The natural uncertainty, *SD natural*, was computed using equation 5-5 of EM 1110-2-1619. The equation is based on streambed type, drainage area, maximum expected stage range, and 1% ACE discharge. SD natural was estimated to be 0.7 feet. The model uncertainty, *SD model,* was estimated using Table 5-2 of EM 1110-2-1619. Because several sections of the Main Stem HEC-RAS model have not been calibrated, Manning's n reliability was judged to be "Poor". Topography for the model is relatively accurate and is primarily based on Comp Study surveys and CVFED LiDAR and bathymetry data. With these parameters, the minimum *SD model* value was estimated at 1.3 feet.

b. Calaveras River and Mormon Slough. Water surface profiles for Calaveras River and Mormon Slough system were computed using an existing draft version of an HEC-RAS (version 4.1.0) steady one-dimensional flow model. The draft model was developed under the California Department of Water Resources (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program. The model was reviewed and modified for the Feasibility Study by Peterson Brustad Incorporated (PBI). Development and review of the model is described in the PBI Technical Memorandum "Review and Update of the CVFED Calaveras River HEC-RAS Model, 9 September 2013 (PBI, 2013B). A map of the HEC-RAS hydraulic model domain showing cross sections and hydrograph boundary locations is provided as Plate 22. The hydraulic model extends from Belota to the San Joaquin River.

(1) Cross Sections. The model contains 425 cross sections with an average spacing of 500 feet. Cross section geometry data were obtained from the LiDAR data acquired by the State of California for their Central Valley Floodplain Evaluation and Delineation (CVFED) program. The data were collected over several weeks between March 17, 2008 and April 4, 2008. The underwater portion of each cross section was adjusted to reflect recent NAVD88 ground surveyed bathymetric cross section data obtained by the State of California Department of Water Resources in 2010.

(2) Storage Areas. The model includes 14 storage areas to account for overland flooding. Storage areas were not defined for the entire study area because overbank flooding is transferred to a FLO-2D model of the floodplain area.

(3) Bridges and Inline Structures. The model contains 62 Bridges and 9 inline structures coded into the model from field surveys and sketches.

(4) Lateral Structures (Levees). The HEC-RAS model utilizes the lateral weir option to simulate overtopping of the levee crest. The structures were manually coded into each HEC-RAS based upon Top of Levee (TOL) elevation data from the USACE National Levee Database (NLDB) survey data. The lateral structure outflow is linked to the storage areas described above.

(5) Levees. The levee crest elevation was specified for each cross section. The top of levee elevation was obtained from the NAVD88 National Levee Database (NLDB) ground survey conducted in 2007-2008.

(6) Blocked Obstructions. Blocked obstructions were used throughout the model to eliminate the cross section area on the landward side of the levee. The landward areas are modeled as storage areas and lateral weirs along the crest of the levee control the flow over and into and out of the storage areas. The blocked obstructions are needed because the cross sections extend approximately 100 feet landward of the levee and this is not a conveyance area under this approach. The levee card is not suitable in this case because the conveyance area on the landward side of the cross section would incorrectly become conveyance area once overtopped. The heights of the blocked obstructions were made sufficiently high to contain a 0.2% (1/500) ACE flood event.

(7) Ineffective Flow Areas. Ineffective flow areas were incorporated into the model to simulate areas where water is stored, but is not active conveyance area.

(8) Manning's Roughness Values. Manning's roughness values range from 0.030 to 0.035 in the main channel and 0.035 to 0.045 in the overbanks. The roughness values were based on limited calibration to high water observations made during a high-water event in 6 April 2006. High water mark staking was not available for the event. The calibration was based on photographs of the high water and anecdotal evidence.

(9) Upstream Boundary Conditions. The primary upstream boundary condition is the regulated flow at the San Joaquin River at Belota gage. Development of the inflow hydrographs is summarized in the hydrology section above. The model also includes inflows from localized drainage at internal boundary conditions throughout the model.

(10) Downstream Boundary Conditions. The downstream boundary condition was the stage-frequency relationship at the Stockton Deep Water Ship Channel at Burns Cutoff. The development of the boundary conditions is described in the 15 August 2013 technical memorandum, Delta Stage-Frequency Analysis for Alternative Comparisons by CESPK-ED-HA. Models were developed assuming 2010 and 2070 sea level conditions at the downstream boundary condition.

(11) Model Calibration. As described above, the model calibration to the 6 April 2006 event was limited by available information.

(12) Stage Uncertainty. The total SD of stage uncertainty was computed at seven index points along Calaveras River and Mormon Slough. A total SD of 0.9 feet is to be used for all reaches of the Calaveras River and Mormon Slough system.

Stage uncertainty was estimated following methods described in EM-1110-2-1619. The total stage uncertainty was estimated from natural and model uncertainty. A detailed description of the model is provided in the PBI Technical Memorandum "Review and Update of the CVFED Calaveras River HEC-RAS Model, 9 September 2013 (PBI, 2013B). The standard deviation (SD) of total stage uncertainty was calculated using Equation 5-6 of EM 1110-2-1619.

$$\mathrm{SD}_{\mathrm{total}} = \sqrt{\mathrm{SD}^2_{\mathrm{natural}} + \mathrm{SD}^2_{\mathrm{model}}}$$

The natural uncertainty, *SD natural*, was computed using equation 5-5 of EM 1110-2-1619. The equation is based on streambed type, drainage area, maximum expected stage range, and 1% ACE discharge. The model uncertainty, *SD model,* was estimated using Table 5-2 of EM 1110-2-1619. The model calibration was estimated to result in a "fair" reliability of Manning's Roughness values. Topography for the model is relatively accurate and is primarily based on Comp Study surveys and CVFED LiDAR and bathymetry data. With these parameters, the minimum *SD model* value was estimated at 0.7 feet.

c. North FLO-2D Model. An existing FLO-2D (version 2009.06) model was utilized to evaluate water surface elevations resulting from levee breaches within the study area. The FLO-2D model was developed by HDR, Inc. as part of the Department of Water Resources' (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program. The model underwent extensive quality control review by DWR and USACE. This model was used in the Feasibility Study to analyze levee breach scenarios at each of the 7 LSJRFS index points along the Calaveras River and Stockton Diverting Canal. A detailed description of the model is provided in the Technical Memorandum, San Joaquin Area Flood Control Agency, Two-Dimensional (FLO-2D) Hydraulic Model of the Lower San Joaquin River System. 3 December 2013. A map of the model domain is provided in Plate 23.

(1) Computational Domain**.** The valid computational domain is defined as the Lower San Joaquin Basin Feasibility study area. The model's domain extends beyond the valid computational domain in order to establish model boundary conditions. All results outside the valid domain were truncated from the results.

(2) Grid Elements**.** A 250-ft grid size was selected in order to keep the number of grid elements down to a workable number and to avoid long model run times. Model geometry was based on LiDAR data acquired by the State of California for their Central Valley Floodplain Evaluation and Delineation (CVFED) program. The data were collected over several weeks between March 17, 2008 and April 4, 2008.

(3) Channel Elements**.** The model includes channel elements for Bear Creek and its tributaries, Fivemile Slough, Mosher Slough, Calaveras River and Mormon Slough, Stockton Deep Water Ship Channel, and French Camp slough and its tributaries.

(4) Floodplain Roughness and Reduction Factors. Overland n-values and area reduction factors (ARF) were developed for a variety of different land uses. Values ranged from 0.04 to 0.11 within urban areas and 0.04 to 0.25 for non-urban areas. The model includes Area Reduction Factors (ARFs) to account for the reduction in storage associated with buildings. The model also includes Width Reduction Factors (WRFs) to account for the reduction in conveyance areas associated with buildings and other structures.

(5) Levees and Embankments. Levees and embankments are included in the model as FLO-2D levee features. However, channels with levees were modeled entirely as channel sections that included their levees as part of the channel.

(6) Hydraulic Structures. Hydraulic structures were coded into the FLO-2D model by adjusting the geometry or utilizing stage-discharge rating curves. The 1-d channel portions of the model included 82 hydraulic structures which included bridges, gate structures or weirs. The hydraulic losses at these structures were developed by applying a discharge-stage relationships developed in HEC-RAS. Structures within the floodplain were modeled by applying a width reduction factor or by including a channel between grid elements. The size of these features was based on aerial topography or ground surveys.

(7) Pump Stations. The model does not include interior pump stations.

(8) Boundary Condition Inflows. The inflow hydrographs for the FLO-2D model consist of levee overtopping and breach hydrographs obtained from HEC-RAS model simulations.

(9) Boundary Condition Outflows. The purpose of the FLO-2D model is to simulate the movement of breach floodwaters within the study area on the interior side of levee system. Outflow elements were specified along the edge of the model boundary.

(10) Stage Uncertainty. Stage uncertainty was not computed for the FLO-2D model results. The FDA model only accounts for uncertainty in the channel stage-discharge relationship. The channel stage-discharge uncertainty is described in the HEC-RAS model description above.

d. South FLO-2D Model. An existing FLO-2D (version 2009.06) model was utilized to evaluate water surface elevations resulting from levee breaches within the study area. The FLO-2D model was developed by HDR, Inc. as part of the Department of Water Resources' (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program. The model underwent extensive quality control review by DWR and USACE. This model was used in the Feasibility Study to analyze levee breach scenarios at each of the 4 LSJRFS index points along the Lower San Joaquin River. A detailed description of the model is provided in the Technical Memorandum, Lower San Joaquin River and Tributaries Two-Dimensional (FLO-2D) Hydraulic

Model of the Lower San Joaquin River System. 20 November 2013. A map of the model domain is provided in Plate 24.

(1) Computational Domain. The valid computational domain is defined as the Lower San Joaquin Basin Feasibility study area. The model's domain extends beyond the valid computational domain in order to establish model boundary conditions. All results outside the valid domain were truncated from the results.

(2) Grid Elements. A 400-ft grid size was selected in order to keep the number of grid elements down to a workable number and to avoid long model run times. Model geometry was based on LiDAR data acquired by the State of California for their Central Valley Floodplain Evaluation and Delineation (CVFED) program. The data were collected over several weeks between March 17, 2008 and April 4, 2008.

(3) Channel Elements. The model includes channel elements for the San Joaquin River and tributaries.

(4) Floodplain Roughness and Reduction Factors. Overland n-values and area reduction factors (ARF) were developed for a variety of different land uses. Values ranged from 0.04 to 0.20 for non-urban areas. The model includes Area Reduction Factors (ARFs) to account for the reduction in storage associated with buildings. The model also includes Width Reduction Factors (WRFs) to account for the reduction in conveyance areas associated with buildings.

(5) Levees and Embankments. Levees and embankments are included in the model as FLO-2D levee features. However, the levees along the San Joaquin River were modeled entirely as channel sections that included their levees as part of the channel.

(6) Hydraulic Structures. Hydraulic structures were coded into the FLO-2D model by adjusting the geometry or utilizing stage-discharge rating curves. The only 1-D channel hydraulic structure included in the model was Paradise Weir. The remaining 1-D channel hydraulic structures were did not significantly impact water surface profiles and were not included in the 1-D channel portion of the model. Structures within the floodplain were modeled by applying a width reduction factors or by including a channel between grid elements. The size of these features was based on aerial topography or ground surveys.

(7) Pump Stations. The model does not include interior pump stations.

(8) Boundary Condition Inflows. The inflow hydrographs for the FLO-2D model consist of levee overtopping and breach hydrographs obtained from HEC-RAS model simulations.

(9) Boundary Condition Outflows. The purpose of the FLO-2D model is to simulate the movement of breach floodwaters within the study area on the interior side of levee system. Outflow elements were specified along the edge of the model boundary.

(10) Stage Uncertainty. Stage uncertainty was not computed for the FLO-2D model results. The FDA model only accounts for uncertainty in the channel stage-discharge

relationship. The channel stage-discharge uncertainty is described in the HEC-RAS model description above.

4.4 Hydraulic Model Results.

The hydraulic models described above were utilized to compute water surface profiles and breach simulations. Water surface profiles and breach simulations were performed for 50% (1/2) ACE, 10% (1/10) ACE, 4% (1/25) ACE, 2% (1/50) ACE, 1% (1/100) ACE, 0.5% (1/200) ACE, and 0.2% (1/500) events.

a. Water surface profiles. Computed water surface profiles for 2010 conditions are presented in Plates 25 for San Joaquin River, Plate 26 for Lower Calaveras River, Plate 27 for Upper Calaveras River, and Plate 27 for Mormon Slough. Computed water surface profiles for 2070 conditions are presented in Plates 29 for San Joaquin River, Plate 30 for Lower Calaveras River. The 2010 and 2070 profiles are similar for the other reaches. Stage-Discharge-Frequency plots at the index points within and outside the study area are shown in Plate31A through 31N and 32A through 32E respectively. The plots include stage estimates for 2010 and 2070 sea level conditions. The Stage-Discharge-Frequency plots also show with project conditions described later in this report.

b. Levee Breach Scenarios. Levee breaches are used to define the inundation if a breach were to occur. Breach simulations were conducted using two methods. A two dimensional method was used where the flood inundation is characterized as shallow unconfined type flooding. A simplified one dimensional level pool method was used for breach locations where the flooded area would equalize to a level water surface elevation. The breach simulation locations and formation parameters are shown on Plate 4 and Table 20.

(1) Two Dimensional Method: This method involved an uncoupled simulation using the one-dimensional HEC-RAS models and FLO2D models described above. A major assumption in this approach is the floodplain flows are not largely influenced by channel hydraulics except at the breach. Therefore, the uncoupled model approach is sufficiently accurate. The levee breach was simulated in a HEC-RAS hydraulic model of the system. The resulting breach hydrograph served as input to a FLO-2D model used to compute the inundation.

Breach formation parameters such as width and time to develop were estimated following the procedures described in the August 2013 Sacramento District Hydraulic Design report "Development of Levee Breach Parameters for HEC-RAS Application". The resulting inundation maps are hypothetical simulations of levee failures and do not represent the probability of occurrence. Breach simulations performed using the two dimensional method are shown on Plates 33A through 33J.

(2) One Dimensional Level Pool Method: This method was utilized for the Delta breach locations where the volume of the inundated area was relatively small with respect to the flow or stage hydrograph. The peak stage in the channel of the HEC-RAS model was assumed to define a level pool. The level pool was mapped using the FLO-2D floodplain elevation elements and computing the depth below the level pool for each grid element. This approach was used for

breach simulations at index points D-BS, D3, D4, and D5 which are shown on Plates 34A through 34D.

Table 20
Levee Breach Simulation Parameters

Flood Source	Breach Location	Levee Height at Breach Location (Feet)	Breach Width (Feet)	Time to Develop full Breach (Minutes)	Economic Impact Area
San Joaquin River	LRTB	1/	1/	1/	RD17
	LR4	17.1	190	27	RD17
	LR3	18.8	210	29	RD17
	LR2	16.5	180	27	RD17
	LR1	16.8	190	27	RD17
French Camp Slough	FR1-1/FR1-2	14.0	155	25	CS-02
	FL1	12.2	1/	1/	RD17
Stockton Diverting Canal	SL1	10.7	118	22	CS-01,CS03
	SL2	10.7	118	22	CS-01,CS-02,CS-03
Calaveras River	CR2	8.0	88	19	NS-04, NS-03
	Cl2	8.5	94	19	CS-01,CS-02,CS-03
Delta Front	D3	11.2	2/	2/	NS-02
	D4	13.5	2/	2/	CS-01
	D5	13.4	2/	2/	NS-03
	D-BS	14.5	2/	2/	NS-03
1/ A breach at LR4 was used to simulate a breach at LRTB 2/ Delta breaches assumed level pool flooding.					

c. Natural Floodplains. Natural floodplains were developed to address planning requirements of ER 1165-2-26. The natural floodplains were developed by plotting the maximum inundation depth from all simulated breaches for a given ACE event. The inundation area represents the maximum extent of areas with potential risk of being flooded from the primary flood sources described in this study. The floodplains are provided in Plates 35 through 42. These floodplains include the effects of unnatural features in the floodplain (bridges, berms, roadways, levees). Therefore, they do not represent the actual "natural conditions".

4.5 Wind-Wave Analysis.

An analysis of wind-wave run-up, wind setup, overtopping discharge, and wind-wave erosion was conducted for levee reaches within the study area. Previous analysis for the Sutter Basin Feasibility study found that wind-wave runup and setup were largely independent of water surface in the top 2/3 of the levee height. Therefore, wind-wave runup and setup were computed assuming the top of levee stage. An assessment of stable rock diameter was also conducted to evaluate the potential for wind-wave erosion.

a. Wind Speed. Wind speed data reviewed for this study included, frequency analysis performed for other studies, and wind speeds observed during historical floods.

(1) Wind frequency. The wind speed frequency analysis for the Sacramento Executive Airport was obtained from the NHC report for the American River Common Features General Reevaluation Report (NHC, 2010). The wind analyses were based on 80 years of record. The Sacramento Executive Airport is located approximately 40 miles north of the study area and wind frequencies are assumed similar to the winds experienced in the study reaches. The winds were developed for six frequencies using the generalized extreme value distribution (NHC, 2010). A review of available wind data for the Orestimba Feasibility Study (USACE, 2012) determined that closer stations were limited for evaluation of wind speeds do to short record lengths and missing values. The wind speed frequency estimates are provided in Table 21. More refined estimates of wind frequency using wind data closer to the study area should be considered in detailed design.

Table 21
Estimated Extreme Event Frequency
1-Hour Wind Speeds for Sacramento Executive Airport

Annual Chance Exceedance	1-Hour Wind Speed by Direction (mph)							
	N	NE	E	SE	S	SW	W	NW
20% (1/5)	32	17	21	37	33	29	25	32
10% (1/10)	35	22	25	42	40	34	30	34
5% (1/20)	39	27	30	47	47	39	36	36
2% (1/50)	44	37	38	54	62	50	48	38
1.3% (1/72.6)	47	42	42	58	69	56	54	38
1% (1/100)	49	46	46	60	76	62	60	39
0.5% (1/200)	54	59	55	66	95	79	77	40
Source: NHC, 2010 Period of Record 1931-2008								

(2) Correlation with Flood Events. Wind speeds observed at the Sacramento Executive Airport during historical annual peak floods were evaluated. The historical record of the San Joaquin River near Vernalis stream gage operated by the United States Geological Survey was used to evaluate annual peak floods. The gage measures flow entering the upstream reach of the study area. Peak 1-hour wind speeds concurrent with each annual peak flood are provided in Table 22. The wind speeds are the maximum recorded in each direction over a 5-day period following the peak flood event. Although the temporal distribution of each flood is unique, hydrologic analysis of historical flood events indicates typical floods are near peak stage for 5 days. Table 22 shows the strongest and prevailing winds are from south and southeast. The table indicates no correlation between high winds and large floods which is consistent with observations made for other feasibility studies in the Sacramento-San Joaquin valley. Thus high wind and high flood stages are assumed to be independent factors for this analysis.

Table 22
Concurrent 1-Hour Wind Speeds Observed during
San Joaquin River near Vernalis Annual Peak Flows

Date/Year of Annual Peak Flow	San Joaquin River Peak Instantaneous Flow (cfs)	Maximum 1-Hour Wind Speed during flood 1/ (Miles per Hour)								
		N	NE	E	SE	S	SW	W	NW	All Directions
3/8/1983	45100	0	0	9	17	17	12	12	7	17
1/7/1984	33000	23	6	7	7	9	6	7	14	23
12/21/1984	5920	13	7	7	9	9	6	5	12	13
3/20/1986	36900	13	3	9	8	10	9	9	14	14
3/8/1987	6410	7	0	9	14	46	13	12	7	46
4/28/1988	2740	18	0	5	10	18	17	17	17	18
5/4/1989	2630	8	0	6	7	21	16	8	13	21
5/30/1990	2050	16	0	0	10	15	21	13	14	21
3/29/1991	4130	13	9	7	12	12	17	10	12	17
2/17/1992	5570	9	10	10	21	16	17	6	17	21
1/20/1993	10300	17	10	10	22	17	21	12	18	22
10/20/1993	4440	14	5	7	6	5	0	6	12	14
3/20/1995	26100	9	0	23	28	23	18	12	21	28
3/11/1996	18000	18	16	12	12	17	17	8	23	23
1/6/1997	75600	22	6	7	9	6	7	14	23	23
2/14/1998	35200	0	6	17	23	23	14	12	0	23
2/15/1999	16100	24	5	8	17	15	10	17	26	26
3/8/2000	16800	10	3	13	18	10	14	9	10	18
2/27/2001	6050	23	21	10	28	8	10	5	22	28
1/5/2002	6370	8	7	7	7	7	8	7	10	10
5/5/2003	3540	8	3	6	8	12	17	13	17	17
3/21/2004	4560	6	0	3	12	14	13	14	7	14
6/7/2005	15400	16	0	3	9	9	20	12	16	20
4/14/2006	34800	10	5	8	7	16	21	14	14	21
10/21/2006	4210	26	3	0	5	5	7	7	30	30
1/29/2008	4760	0	0	6	26	19	11	10	3	26
5/13/2009	3095	15	0	0	5	11	11	9	15	15
	Minimum	0	0	0	5	5	0	5	0	
	Average	13	5	8	13	14	13	10	15	
	Maximum	26	21	23	28	46	21	17	30	

1/ Flood duration assumed to extend 5-days after peak
Wind speed data from computer data files supporting NHC,2010

(3) Adopted Wind-Wave Scenarios. Five wind-wave scenarios were adopted for this study. The scenarios were developed to be compatible with economic analysis, evaluation of potential NFIP accreditation, and address the requirements of the DWR ULDC. The five scenarios are described below. The adopted wind speeds for each scenario are provided in Table 23.

- 95% ACE Wind-Wave Scenario. The 95% ACE Wind-wave speed was assumed to be the minimum wind measured in any direction during the historical floods shown in Table 22.
- 50% ACE Wind-Wave Scenario. The 50% ACE wind speed is the average wind speed expected to be concurrent with flood stages along the project levee. The 50% ACE wind was assumed to be the average wind measured from the fetch direction during the historical floods shown in Table 22.
- 20% ACE Wind-Wave Scenario. The 20% ACE wind speed is based on the wind frequency analysis presented in Table 21 for the given fetch direction.
- 5% ACE Wind-Wave Scenario. The 5% ACE wind speed is based on the wind frequency analysis presented in Table 21 for the given fetch direction.
- 1.3% ACE Wind-Wave Scenario. The 1.3% ACE wind speed is based on the wind frequency analysis presented in Table 21 for the given fetch direction. The assumption is that a wind of this magnitude, concurrent with floodwaters along the project levee, is a sufficiently rare combination that the runup will be exceeded only a small percent of the time.

Table 23
Adopted wind speeds for analysis

Reach	Site	Fetch Dir.	1-Hr Wind Speed Scenario (mph)				
			95% ACE	50% ACE	20% ACE	5% ACE	1.3% ACE
San Joaquin River Main Stem	SJR_160_R	South	5	14	33	47	69
RD17 Tieback	SJR_200_R	South	5	14	33	47	69
Delta Front- Shima Tract	ST_20_R	West	5	10	25	36	54
Delta Front- Fourteen Mile Slough	FM_30_L	West	5	10	25	36	54
RD 17 Weston Ranch Ring Levee	SJR_80_R	South	5	14	33	47	69
RD 17 Set Back South Levee	SJR_120_R	North	5	13	32	39	47
RD 17 Set Back East Levee	SJR_110_R	West	5	10	25	36	54
RD 17 Set Back North Levee	SJR_80_R	South	5	14	33	47	69

(4) Effective Fetch Length. The effective fetch is the horizontal distance in miles, in the direction of the wind, over which the wind generates waves or creates a wind setup. The effective fetch was computed by averaging an arc of radial lines from the embankment to the opposite shore, centered on the wind direction. The effective fetch was based on the method outlined in EM 1110-2-1420 and has a total angle of 24 degrees and uses 9 radials (including the central radial). For this analysis, the central radial was placed such that it is within the range of directions for peak winds and close to perpendicular to the levee.

The average fetch length was calculated using the following equation:

$$F = \frac{\sum X_i \cos(\alpha_i)}{\sum \cos(\alpha_i)}$$

Where: F = Average Fetch Length [L]; X_i = length of radial i; α_i = angle between the radial and the central radial

More refined estimates of wind-wave runup for detailed design should consider the full range of potential wind and fetch directions.

(5) Average Fetch Depth. The average fetch depth is the average depth of water over the effective fetch. Wind-wave analyses were performed assuming the water surface elevation was at the levee crest. Levee elevations were determined using terrain data from the DWR CVFED project (DWR, 2013) in GIS. The average depth was calculated by subtracting the ground elevation at several points along the fetch from the top of levee elevation and averaging the result. More refined estimates of fetch length and depth should be considered for detailed design.

(6) Wave Prediction. For each wind-wave scenario, wind-wave characteristics including the significant wave height and peak wave period were developed using the Hurdle and Stive method. The significant wave height estimates were based on, wind speed, wind duration, average fetch length, and average fetch depth described above.

(7) Wind-Wave Setup. Wind-wave setup was determined by EM 1110-2-1420, formula 15-1, and is defined as the wind tide (setup caused by the wind on the water surface), the vertical rise in feet above the Stillwater Level. Formula 15-1 to determine the wind setup is:

$$S = \frac{U^2 xF}{1400 xD}$$

where:

S is Wind Setup in feet above the Stillwater level
U is the wind speed in miles per hour
F is the single fetch length in miles
D is the average water depth in feet over the fetch

In cases where the waves are fetch limited; the wind-wave setup was based on the fetch limited wind speed. The fetch limited wind speed was calculated from the 1-hour wind speed and the fetch limited duration using Figure II-2-1 in the Coastal Engineering Manual - Part II, EM 1110-2-1100.

(8) Wave Runup. Wave runup was calculated using the USACE Coastal Engineering Manual (CEM) method and the EurOtop method. The EurOtop method is described by Equation 5.4 in the Report, “EurOtop, Wave Overtopping of Sea Defenses and Related Structures, Assessment Manual”, (HR Wallingford, 2008). The CEM results were found to be similar and only the EurOtop wave results are provided in this memo. A more detailed description of the method is provided in Ford, 2011.

Wind-wave runup was calculated for the typical levee design slope. The typical water side design slope of levees within the study area is 3ft horizontal to 1 foot vertical. The slope roughness varies throughout the study area from grass lined to rock revetment. Wave runup was calculated for both conditions. Surface roughness values were obtained from Table VI-5-3 of EM 1110-2-1100. A slope roughness factor of 1.0 and 0.55 was used to estimate wave runup on grass lined and rock revetment slope conditions respectively. The angle of incidence is the angle at which the waves approach the shore. For this analysis the angle of incidence was assumed to be normal to the levee. The actual angle of incidence varies throughout each reach.

(9) Overtopping discharge. The height above still water at which a levee failure would likely occur from wind-wave overtopping was estimated for the wind scenarios. The overtopping discharge was calculated using the EurOtop method. To be consistent with EC 1110-2-6067 an overtopping discharge of 0.05 cfs/ft was used to determine the likely failure point due to wave overtopping.

(10) Wind-Wave Erosion. For each site the estimated rock revetment size to prevent wind-wave erosion was evaluated using the Hudson method provided in Design Guideline 17 Riprap Design for Wave Attack by the Federal Highways Administration (FHWA, 2011). For riprap protected levees, the equation for determining rock size is:

$$W_{50} = \frac{\gamma_r H^3 (\tan\theta)}{K_d (S_r - S_w)^3}$$

$$d_{50} = \sqrt[3]{\frac{W_{50}}{0.85\gamma_r}}$$

Where: W_{50} = weight of the median riprap particle size (lb); γ_r = unit weight of riprap (lb/ft^3); H = design wave height (ft) ; K_d = Empirical coefficient for riprap (= 2.2); S_r = specific gravity of riprap; S_w = specific gravity of water (1.0 for fresh water); θ = angle of slope inclination; d_{50} = median diameter of riprap particles (ft). The design guidance recommends the minimum design wave height for use with the Hudson equation should be the 10 percent wave. Design guideline 17 recommends multiplying the significant wave height by 1.27 to determine the 10 percent wave.

Estimated rock revetment sizes to prevent wind-wave erosion for each wind-wave frequency are provided in Table 24.

(11) Wind-Wave Results. Analysis was performed for two representative levee reaches within the study area. Wind-wave analyses were not conducted for Calaveras River, Mosher Slough, Stockton Diverting Canal, and Smith Canal because fetch lengths were less than 500 feet and not considered long enough for wind-waves to be a significant performance consideration in this study. The names of the typical sites described below are based on cost estimating reach number designations described in Plates 9A through 9D.

The San Joaquin River Main stem location is considered to be representative of all San Joaquin River, Stockton Deep Water Ship Channel, and French Camp Slough levee reaches considered in the alternatives. Run-up estimates assumed the levee slope was grass lined. The RD17 Tieback Levee location is representative of the Tieback levee at the upstream reach of RD17. Run-up estimates assumed the levee slope was grass lined.

Estimated stable rock sizes for the two sites are provided in Table 24. Results for wind-wave run up and setup up for a hypothetical water level at the levee crest are summarized in Table 25. The complete analysis is described in the Technical Memorandum "Wind-wave Analysis for LSJRFS Alternative Comparisons", 14 February 2014.

Table 24
Estimated Stable Rock Revetment Sizes

Reach (Representative Wind-wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Hs Significant Wave Height (Feet)	H10 10% Wave Height (Feet)	Stable Rock Revetment Size	
							Median Weight (lbs)	Median Diameter (Feet)
San Joaquin River Main Stem (SJR_160_R)	1.3%	69	1900 ft	18.0 ft	1.3 ft	1.7 ft	25 lbs	0.6 ft
	5%	47			0.9 ft	1.1 ft	8 lbs	0.4 ft
	20%	33			0.6 ft	0.8 ft	3 lbs	0.3 ft
	50%	14			0.3 ft	0.4 ft	0.3 lbs	0.1 ft
	95%	5			0.1 ft	0.1 ft	0.01 lbs	0.04 ft
RD17 Tieback (SJR_200_R)	1.3%	69	24300 ft	14.0 ft	3.9 ft	5.0 ft	680 lbs	1.7 ft
	5%	47			2.6 ft	3.3 ft	200 lbs	1.1 ft
	20%	33			1.7 ft	2.2 ft	56 lbs	0.7 ft
	50%	14			0.6 ft	0.8 ft	3 lbs	0.3 ft
	95%	5			0.2 ft	0.3 ft	0.1 lbs	0.09 ft

Notes:

* Wave Runup calculated using EurOtop method
**Stable Rock Size based on Hudson Method.

Table 25
Summary of Wind-wave Run-Up and Set Up, Alternative 1

Reach (Representative Wind-wave Reaches and Cover)	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) (Grass Lined)	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
RD17 Tieback (SJR_200_R) (Grass Lined)	1.3%	69	24300 ft	14.0 ft	9.5 ft	1.1 ft	7.2 ft
	5%	47			6.4 ft	0.4 ft	4.1 ft
	20%	33			4.4 ft	0.2 ft	2.3 ft
	50%	14			1.7 ft	0.0 ft	0.5 ft
	95%	5			0.5 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method

**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

4.6 Sedimentation and Channel Stability

a. Sedimentation. A stage 1 sediment impact assessment was conducted for the study area. The assessment included a review of existing information, sedimentation problems, and the anticipated impact of sedimentation on project performance and maintenance.

(1) San Joaquin River. A sediment and geomorphic assessment of the San Joaquin River was conducted for the 2002 Sacramento San Joaquin Comprehensive Study. The study included the San Joaquin River from the Old River to the Merced River, Stanislaus River from San Joaquin River to Knights Ferry, Tuolumne River from San Joaquin River to LaGrange, and Merced River from San Joaquin River to Merced Falls. The report included a comparison of historical invert profiles and analysis of bed load transport capacities. The analysis indicated the lower San Joaquin River within the study is relatively stable.

An invert profile provided on figure 3.19 of the report compares a 1914 survey to a 1998 survey. Although the profile does not extend downstream of the Vernalis gage, the profile indicates a relatively stable profile in the reach directly upstream from the study area. The study included an evaluation of bed load transport capacities throughout the study reaches. Reach averaged hydraulic characteristics developed from an HEC-RAS model were used in combination with a flow duration curve and COE SAM program to estimate an average annual bed material transport capacity. The analysis was conducted for historical (pre-Friant Dam condition) and existing conditions. For the historical condition the bed material transport capacity was estimated to range from 62,700 tons/year in the upstream reach to 102,200 tons/year at in the downstream reach. The same approach was used to estimate the bed load transport capacity using flow duration curves developed from post reservoir (existing) conditions. The bed load transport capacity for existing conditions was found to range from 139,600 tons/year in the upstream San Joaquin River reach to 291,100 tons/year at in the downstream reach.

Comparison of the historic and present annual bed material transport capacities indicated that the transport capacities have increased by about 60 percent in the upstream reach and 185 percent in the downstream reach between 1914 and 1998. The indicated increase is likely caused by increased hydraulic energy associated with deepening and general narrowing of the channel between 1914 and the present. The available information indicates that the channel is stabile to degrading.

An evaluation of stream flow measurements from 1987 to 2015 and annual peak flows from 1932 to 2015 was conducted for the USGS Gage San Joaquin River near Vernalis. This gage is located in the reach directly upstream of the study area. The comparison was adjusted to account for two gage datum changes that have occurred since 1932. The comparison indicates that the stage-discharge relationship has lowered a few tenths of a foot over the period. However, this is within the range of measurement uncertainty and does not indicate a significant degradation trend.

Downstream of Stockton, the San Joaquin River is utilized as the Stockton Deep Water Ship Channel. Periodic dredging of the channel is conducted to maintain the authorized navigation depth and width. From 2000 through 2014 the average annual amount of sediment removed from the ship channel was 175,000 yards per year or about 225,000 tons/year assuming 95lbs/ft^3. This number agrees fairly well with the transport capacity estimates described above and also supports an assessment of a relatively stable channel.

(2) Mosher Slough. Research of past studies found no sediment transport analysis of Mosher Slough. The drainage area is highly urbanized and has gentle slopes. The project sponsors did not identify any issues or problems with sediment aggregation or maintenance. The reach is not considered to have a large source of sediment or sediment issues. Therefore, this reach was not studied in detail.

(3) Lower Calaveras River and Morman Slough. A review of past studies of Lower Calaveras River and Mormon Slough found no detailed sediment studies. Much of the historical sediment load is now captured by New Hogan Dam. The project sponsors did not identify any issues or problems with sediment aggregation or maintenance. Therefore, it is not considered to have a large source of sediment and was not studied in detail.

b. Erosion. A qualitative assessment of the potential for erosion-induced levee breach within the study area was conducted by the State of California’s Urban Levee Evaluation Program. Each assessment was conducted in three tiers of increasing level of detail and is described in Erosion Screening reports for RD404, Shima Tract, RD17, Stockton Diverting Canal/Mormon Slough, and Smith Canal.

The Tier 1 analysis involved a review of available records and did not include filed inspections (DWR, 2014). The Tier 1 analysis was a first-cut evaluation of the levees in the study area and identified areas where erosion risk potential exists due to factors such as the presence of erosion-susceptible materials, reduced levee cross section, and the potential for wind-wave scour (DWR, 2014). If the levee met current geometric design standards, a wind fetch length less than 1000 feet, no historical erosion performance issues, and met soil erodibility test criteria, it was found to meet the Tier I criteria and was categorized as a low erosion risk segment (DWR, 2014). Levee segments not meeting Tier 1 criteria were advanced to a Tier 2 analysis.

The Tier 2 analysis involved further evaluations of flow velocity in comparison to surface erosion adequacy, wind-wave shear in comparison to surface erosion adequacy, and field reconnaissance (DWR, 2014). If the levee segment was found to meet the criteria for all three areas of the Tier 2 analyses, it was designated as having low erosion risk, because the levee embankment demonstrates sufficient resistance to velocity and wind shear stress (DWR, 2014). If a levee segment did not meet any of the three Tier 2 analysis criteria, it was advanced to Tier 3 for further study.

The Tier 3 analysis involved calculating the ratio of estimated Total Erosion (TE) to the Levee Width (LW) at the estimated 0.5% (1/200) ACE water surface elevation. The levee width was based on the total levee width available to prevent an erosion failure at this water surface elevation. For this analysis a representative cross section was selected for each segment and rates of erosion were estimated for a flood hydrograph similar to the largest flood on record (January 1997). A low erosion risk was assigned if the TE/LW ratio was less than 5%. A medium erosion risk was assigned if the TE/LW ratio was between 5% and 25%. A high erosion risk was assigned if the TE/LW ratio was greater than 25%.

The erosion analysis identified approximately 5 miles of the levee within RD17 with a high erosion risk. The high risk erosion sites were found within segments SJR_90_R, SJR_100_R, SJR 110_R, SJR 120_R, SJR_140_R, SJR_150_R, and SJR_160R. None of the other ULE levee reaches evaluated within the study area were found to have a high erosion risk.

4.7 Performance and Flood Risk

Performance is described by Annual Exceedance Probability and the assurance of preventing damages from a range of flood frequencies. Flood risk is defined as the probability of a flood event occurring and the consequences of occurrence. Performance and Flood Risk were assessed using the USACE FDA model version 1.2.5a (USACE, 2010). The FDA model combines flow-frequency, stage-discharge, geotechnical fragility, and stage-damage relationships to estimate damages. Uncertainty in each relationship is incorporated by assigning uncertainty estimates and applying a Monte Carlo type approach to combine the results.

Flow-frequency, stage discharge, and geotechnical frequency relationships reflect the exterior (probability) portion of the flood risk calculations. Inundation depth and stage-damage relationships reflect the interior (consequence) portion of the flood risk calculations.

For the probability portion of the risk calculations, the hydraulic model assumptions are based on flows contained to the channel (allowed to overtop without failure). This assumption makes the breach probability statistically independent rather than dependent on another breach occurring (or not occurring). This is consistent with historical observations that indicate the probability of a breach does not appear to be highly dependent on other breaches occurring. There is no specific guidance on how to apply overtopping assumptions to system wide risk analysis and the approach is consistent with USACE risk and uncertainty guidance in EM 1110-2-1619. A sensitivity analysis to this assumption is provided in the Hydrology Section.

For the consequence portion of the risk calculations, the hydraulic model assumptions are based on levee breach failure or simply the depth for natural overbank (non-levee) conditions.

The risk assessment approach included an evaluation of potential flood sources with respect to geotechnical fragility, channel hydrology, channel hydraulics, and potential inundation patterns of a levee breach or natural overbank (non-levee). Fifteen index points were identified to reflect the reach characteristics within the study area. Within each reach a representative geotechnical fragility curve was developed. At the geotechnical curve location a stage-discharge relationship was developed using the system based hydraulic models described above. Selection of the geotechnical reaches is described in detail in the geotechnical analysis report.

a. Performance. Performance is described by Annual Exceedance Probability (AEP), assurance of passing a given Annual Chance Exceedance (ACE) hydrologic event, and Long Term Risk. AEP describes the probability of the design being exceeded over the full range of flood events and their uncertainties. The reliability of Flood Risk Management (FRM) features within the study area is expressed as an assurance level (conditional non-exceedance probability) for a given median ACE hydrologic event. The Long Term Risk describes the probability of being flooded over a given period of time (For example, 10, 30, or 50 years). The performance varies over levee reaches due to variations in geotechnical fragility, hydrology, and hydraulic characteristics and their uncertainties.

Performance was computed for the 15 index points within the study area using the HEC-FDA computer program. The index points are shown on Plate 3. Performance was calculated at the representative geotechnical fragility curve location and assumed to represent the performance at the breach location. Performance was calculated with the HEC-FDA program using an unregulated flow-frequency curve, unregulated to regulated transform, stage-discharge relationships, and geotechnical fragility curves. Uncertainty in each relationship was incorporated in the FDA model. The probability of failure due to wind-wave runup and setup was not included in the performance calculations because it found to be relatively small compared to the other modes of failure and would have no influence on plan selection. The fragility curves are provided in Attachment A. FDA input assumptions are described in Table 26.

Flow-frequency curves were based on the analytical statistics computed for unregulated conditions. Uncertainty in the flow-frequency curve is based on the period of record described in the hydrology section above. The nearest upstream analytical curve statistics were utilized in combination with an unregulated-regulated transform. The unregulated flow in the transform is computed directly from the flow frequency statistics. The regulated flow used in the transform was obtained from the hydraulic model at the index location. The transforms are used to translate the uncertainty in flow frequency estimates to the regulated condition.

The geotechnical fragility curves were based on geotechnical analysis and are presented in the geotechnical addendum and provided as Attachment A to this report. The curves are assumed to have a 100% probability of failure at the levee crest. The crest elevation was modified in the FDA model to represent the Hydraulic Top of Levee (HTOL). The hydraulic top of levee at the index point is defined as the elevation corresponding to the first point of overtopping within the reach. The HTOL is lower than the actual top of levee at index points with high localized crest elevations. The probability of failure due to wind-wave runup and setup was not included in the geotechnical fragility curve because it was found to be relatively small compared to the other modes of failure and would have no influence on plan selection.

Stage discharge relationships used in the analysis are described in Plates 31A through 31N. The uncertainty in the stage discharge curves was calculated using methods described in EM 1110-2-1619, Risk Analysis for Flood Damage Reduction Studies.

Table 26
FDA Input for San Joaquin River Performance Calculations
Alternative 1 - No Action

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	1/	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR4	33.9	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR3	31.0	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR2	27.8	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR1	25.0	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1-1	21.8 (2/)	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	FR1-2	15.9 (3/)	Not Applicable	Scenario A	SJR nr Vernalis	EPR = 82yrs
	FL1	21.4	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	39.2	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
	SL2	44.6	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	29.7	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
	Cl2	31.4	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
Delta Front	D3	13.2	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D4	18.8	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D5	17.5	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D-BS	18.0	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs

1/ Parameters at LR4 used to estimate performance of LRTB
2/ Elevation of Top of Levee at index point.
3/ Elevation of natural ground at upstream end of levee and fragility curves are not applicable.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Flood Depths. Maps showing composite floodplains were developed to demonstrate FRM assurance relative to a standard assurance criterion. The maps show inundation from any flood source that would not meet a risk and uncertainty based assurance criterion. The assurance criterion was based on the NFIP levee system analysis criteria described in EC 1110-2-6067 and was adopted for use in describing the performance of all ACE events. This criterion is described as “Option 2” in the DWR Urban Levee Design Criteria. The assurance criterion utilized for this study does not account for wind-wave overtopping.

- For assurance less than 90% the levee does not pass criteria
- For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria.
- For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria.

The composite floodplains are provided in Plates 43 through 50. Table 27 provides performance values at simulated breach locations for 2010 conditions. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation of project performance.

Table 27
Performance at Simulated Levee Breach Locations, Alternative 1
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0117	0.0110	0.2973	0.4446	0.9999	0.9984	0.9918	0.8749	0.5090	0.1908	0.0384
LR4	0.0073	0.0706	0.1971	0.3064	0.9999	0.9731	0.9525	0.9241	0.8826	0.8423	0.8095
LR3	0.0095	0.0913	0.2496	0.3803	0.9999	0.9761	0.9394	0.8998	0.7938	0.6627	0.5650
LR2	0.0211	0.1923	0.4731	0.6563	0.9999	0.9289	0.8683	0.7922	0.6831	0.5788	0.5161
LR1	0.0126	0.1188	0.3158	0.4688	0.9999	0.9610	0.9400	0.8830	0.7439	0.5772	0.4620
French Camp Slough											
FR1-1	0.0353	0.3015	0.6592	0.8337	0.9999	0.9136	0.8716	0.8150	0.6554	0.4122	0.2349
FR1-2	0.0148	0.1381	0.3597	0.5243	0.9999	0.9939	0.9545	0.8100	0.4548	0.1471	0.0129
FL1	0.0132	0.1245	0.3290	0.4857	0.9999	0.9629	0.9460	0.9208	0.8269	0.6239	0.3857
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8090	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	.01680	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.1519	0.8074	0.9929	0.9997	0.8276	0.7477	0.7230	0.7021	0.6330	0.4968	0.3859
D4	0.0646	0.4872	0.8652	0.9645	0.9460	0.8776	0.8283	0.7876	0.7291	0.6462	0.5608
D5	0.1197	0.7206	0.9782	0.9983	0.8758	0.7806	0.7593	0.7426	0.7206	0.6890	0.6545
D-BS	0.1521	0.8079	0.9929	0.9997	0.8720	0.8005	0.7712	0.7522	0.7085	0.6381	0.5848

FR1-1 describes performance of levee
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.
Cell shaded if assurance is less than criteria.

c. Flood Velocities. Flood velocities are an indicator of life safety risk. If a levee breach were to occur, inundation velocities and depths within the study area would vary by proximity to a breach, breach location, and magnitude of flood event. The velocity field for a levee breach can be characterized as highest near the breach due to the rapidly varying flow conditions. The remaining area would have lower velocities associated with the slope of the topography and floodplain roughness. For evaluation of life loss consequence the study area can be divided into a breach zone, zone with rapidly rising water, and a remaining zone (Yonkman, 2008). Simulations of levee breaches at the peak stage of a 1% ACE event were used to evaluate characteristics of each zone.

(1) Breach zone. The breach zone is characterized by destruction of buildings and the highest life safety consequence. Yonkman describes this area as having velocities greater than 6 feet per second and the product of depth and velocity greater than 22 ft^2 per second. For the Lower San Joaquin Feasibility study, the limit of this zone is estimated to range from 250 feet to 7,600 feet from the breach location. The results indicate a breach zone of approximately 250 feet for the Calaveras River, Mormon Slough, and upper reaches of French Camp slough. The breach zone for Lower San Joaquin River, Delta, and Lower French Camp Slough could be as much as

7600 feet. This was based on the evaluation of the maximum velocity and maximum depths in breach simulations. The characteristics of simulated breaches are shown Table 28.

(2) Zone with rapidly rising water. This zone is characterized by rapidly changing velocity and depth. Model results indicate velocities of less than 3 feet per second within a few thousand feet from the levee for most breach simulations. Within this zone, the product of depth and velocity would be greatest adjacent to the Delta Front and San Joaquin River levees and would be the highest life safety concern within this zone.

(3) Remaining zone. This zone is characterized by slower onset of flooding. The majority of the study area is defined as the remaining zone. Models of breaches indicate velocities of less than 2fps for the remaining portion of the inundation area. Higher velocities are indicated where flows overtop linear features. Additional locations with higher velocities may occur. However, they would be localized and uncertain.

Table 28
Levee Breach Simulations, 1% (1/100) ACE

Economic Impact Area	Breach ID	Grid Element	Breach Width (Feet)	Time to Develop full Breach (Minutes)	Breach Initiation Time (Hour)	Peak Breach Outflow (1% ACE) (cfs)	Maximum Grid Element Depth at Breach (1% ACE) (Feet)	Estimated Radial extent of Breach Zone (1% ACE) (Feet)
North Stockton	CR2	70712	88	19	308	1250	2.0	250
	CR1	74635	79	18	309	1060	1.8	250
Central Stockton	SL2	85232	118	22	311	3130	3.0	250
	SL1	77803	118	22	310	900	1.5	250
	CL2	72302	94	19	271	610	1.7	250
	CL1	78512	95	19	311	880	1.2	250
	FR1	114492	155	25	123	4500	7.4	250
RD17	LR1	2343	190	27	129	7800	10.3	400
	LR2	6064	180	27	133	6400	13.3	1600
	LR3	9580	210	29	135	11,700	9.7	400
	LR4	14469	190	27	133	10,200	11.5	7600
	FL1	1/	1/	1/	1/	1/	1/	1/
1/ The LR1 breach simulations were used because FL1 was found to be similar.								

d. Flood Warning Time. Flood warning time varies throughout the area and is dependent on the source and type of flood event. The principle sources of flood warnings are advisories by the National Weather Service (NWS) and river stage forecasts by the California Nevada River Forecast Center (CNRFC). The flood warning time would likely be greater for an overtopping related breach than a geotechnical failure type breach.

Flood warnings/small river and stream flood warnings are issued by the NWS when flooding of main stem rivers is occurring or imminent (CNRFC, 2013). Main stem river flooding refers to flooding of gauged and forecasted rivers (CNRFC, 2013). The product can also be used to issue Small River and Stream Flood Warnings for smaller rivers/streams which do not have forecast points.

Flash Flood Warnings are issued when flooding is reported; when precipitation capable of causing flooding is observed by radar and/or satellite; when observed rainfall exceeds flash flood guidance or criteria known to cause flooding; or when a dam or levee failure has occurred or is imminent (CNRFC, 2013). A flash flood is defined as a flood caused by heavy or excessive rainfall in a short period of time, and occurring generally within 6 hours of the causative event (CNRFC, 2013).

In addition to the advisories described above, the NWS in coordination with the California Department of Water Resources issues forecasts and guidance for river flows through the CNRFC. In general, river forecasts are based on modeled runoff from observed precipitation, snowmelt estimates, and reservoir operations. The forecast length varies depending on the location. River guidance is based on modeled runoff from forecasted precipitation, snowmelt estimates, and reservoir operations. The forecasts and guidance are issued for a forecast site in a graphical format that compares the future river stage to a monitor stage, flood stage, and danger stage. The combined forecast and guidance are made 5 days into the future.

Flooding from interior drainage sources within the study area is likely to be the result of localized concentrated rainfall. It is assumed these floods would be preceded by a general flood watch issued by the NWS 12 to 24 hours in advance and a flash flood warning 6 hours in advance of the localized flooding.

Flooding from a levee overtopping event along the San Joaquin River would result from a large regional storm event in the San Joaquin River Watershed. CNRFC river flood forecast points on the San Joaquin River are located at Vernalis and Mossdale. It is assumed that an overtopping flood would be preceded by a flood warning and river guidance issued by the NWS and CNRFC five days in advance. A more accurate warning of potential levee overtopping, based on river forecasts, would likely be made 48 hours in advance. This estimate was based on a review of the flood guidance plots for December 2005-January2006 flood which indicate the forecasted peak flow was similar to the observed flow approximately 48 hours prior.

Flooding from a levee overtopping event along the Calaveras River, Stockton Diverting Canal, or Mormon Slough, would result from a large regional storm event in the Calaveras River watershed. There are no CNRFC forecast points in the Calaveras River watershed. It is assumed these floods would be preceded by a flood warning by the NWS and CNRFC five days in advance. Forecasted releases from New Hogan Dam would likely be posted to the California Data Exchange Center and the Sacramento Districts Website. However, there is no standard operating procedure or requirements to make these forecasts available to the public.

It is estimated that flooding from a geotechnical levee breach would have little to no advance warning (less than 1 hour) and the floodwave would rapidly inundate the adjacent areas.

4.8 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

The evaluation of induced flooding and potential for transfer of risk followed the method described in ER 1165-2-216 “Policy and Procedural Guidance for Processing Requests to Alter U.S. Army Corps of Engineers Civil Works Projects Pursuant to 33 USC 408. Section F3 of ER 1165-2-216 requires the hydraulic analysis will consider the full range of loading conditions. For loading conditions where flood waters exceed the project’s system capacity, the analysis is to assume weir flow. The policy stipulates that under no circumstances will the analysis assume breach or malfunction of any existing or altered component of the project system for the flood up to the top of containment as a means of relieving system impacts. The project is to be considered stable and functional to top of containment. The assumption is that the project can be stabilized to the authorized condition. Based on this assumption, fragility curves are not required. Impacts will be determined by comparing performance parameters (annual exceedance probability (AEP), assurance (conditional non-exceedance probability (CNP), etc.) for the existing and authorized conditions, if they are different, to the conditions resulting from the project alteration.

When a project results in induced damages, mitigation should be investigated and recommended if appropriate. Mitigation is appropriate when economically justified or there are overriding reasons of safety, economic or social concerns, or a determination of a real estate taking (flowage easement, etc.) has been made. Remaining induced damages are to be accounted for in the economic analysis and the impacts should be displayed and discussed in the report. (ER 1105-2-100, para.3-3.b.(5)).

There is no induced flooding for the no-action plan. However, a description of flood depth, duration, and frequency, are provided below for comparison with the other plans.

a. Flood Depth. Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Four index points were selected outside the study area to demonstrate the potential change in flood depths outside the study area. Middle River at Borden Highway index point is located at a recording stage gage and was selected to represent potential changes to the stage of middle River downstream of the study area. Old River at Clifton Court Ferry index point is located at a recording stage gage and was selected to represent potential changes to the stage of Old River downstream of the study area. Paradise Cut at Paradise Road index point was selected to represent potential changes to stage in Paradise Cut adjacent to the planned River Islands development. The Stockton Deep Water Ship Channel (SDWSC) at Burns Cutoff index point is located at a recording stage gage and was selected to represent potential changes to the stage of San Joaquin River downstream of the study area.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration. The duration of a high flood stages depends on storm duration, antecedent watershed conditions, and antecedent reservoir storage. The duration of high stages along the delta front and San Joaquin River would likely be one week. The duration of high stages along the Calaveras River would likely be several days. The duration of high stages from interior runoff would likely be less than 1 day.

If a levee was to breach during a flood the floodwater would flow to the lowest portions of the study area and would pond behind the levees of the San Joaquin or Delta Front. Interior drainage facilities are designed to only address rainfall runoff from the interior areas and the volume of floodwaters would overwhelm these facilities. Due to high stages on the exterior the ponded floodwaters would have to be evacuated using portable pumps or by waiting until the exterior stage receded to allow gravity flow through a relief cut in the levee. It is estimated that these floodwaters could pond in the interior for several weeks to a month. The duration of each flood event is unique. In general, the ponding would be longer for larger events.

c. Frequency. The change in flood frequency is described by changes in Annual Exceedance Probability (AEP) and Assurance. The change in stage and flow frequency at index points is provided in Plates 31 and 32. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

The performance values associated with hydrologic and hydraulic parameters are provided in Table 29. For purposes of evaluating induced flooding the risk analysis is limited to hydrologic and hydraulic parameters and their uncertainties. This approach is consistent with Section 3.b (2) of the memorandum "Clarification Guidance on the Policy and Procedural Guidance for the Approval of Modifications and Alterations of Corps of Engineers Projects" (USACE, 2008).

Table 29
2010 Performance at Selected Locations, Alternative 1
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0113	0.1075	0.2892	0.4338	0.9999	0.9999	0.9957	0.8808	0.5134	0.1915	0.0374
LR4	0.0001	0.0007	0.0022	0.0037	0.9999	0.9999	0.9999	0.9998	0.9975	0.9858	0.9693
LR3	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9992	0.9982
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9998	0.9986	0.9972
LR1	0.0005	0.0050	0.0148	0.0245	0.9999	0.9999	0.9999	0.9990	0.9838	0.9251	0.8565
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.9999	0.9999	0.9999	0.9970	0.9485	0.7612	0.5501
FR1-2	0.0148	0.1381	0.3597	0.5243	0.9999	0.9939	0.9545	0.8100	0.4548	0.1471	0.0129
FL1	0.0031	0.0306	0.0889	0.1437	0.9999	0.9999	0.9999	0.9964	0.9407	0.7268	0.4865
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0029	0.0288	0.0839	0.1358	0.9999	0.9982	0.9931	0.9814	0.9172	0.7624	0.6203
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0010	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9994	0.9994	0.9994	0.9994
Paradise Cut at I-5 F-PCI5	0.0014	0.0140	0.0415	0.0682	0.9999	0.9995	0.9995	0.9995	0.9995	0.9952	0.9779
Paradise Cut at Paradise Rd. F-PCPR	0.0017	0.0167	0.0492	0.0807	0.9999	0.9995	0.9995	0.9995	0.9995	0.9867	0.8641
SDWSC blw Burns Cutoff F-B95660	0.0002	0.0016	0.0049	0.0081	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.
FR1-1 describes performance of levee
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.
SDWSC- Stockton Deep Water Ship Channel

4.9 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 30. Composite floodplain maps were not developed for 2070 conditions.

Table 30
Performance at Simulated Levee Breach Locations, Alternative 1, 2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0118	0.1122	0.3002	0.4483	0.9999	0.9984	0.9912	0.8707	0.5026	0.4440	0.5153
LR4	0.0075	0.0726	0.2023	0.3139	0.9999	0.9725	0.9509	0.9228	0.8819	0.8454	0.8093
LR3	0.0101	0.0968	0.2632	0.3990	0.9999	0.9715	0.9362	0.8962	0.7875	0.6712	0.5652
LR2	0.0257	0.2295	0.5426	0.7285	0.9999	0.9153	0.8415	0.7718	0.6711	0.5826	0.5153
LR1	0.0141	0.1326	0.3475	0.5091	0.9999	0.9567	0.9334	0.8764	0.7412	0.5910	0.4616
French Camp Slough											
FR1-1	0.0350	0.0314	0.6590	0.8336	0.9999	0.9136	0.8717	0.8153	0.6562	0.4100	0.2356
FR1-2	0.0148	0.1381	0.3597	0.5243	0.9999	0.9939	0.9545	0.8100	0.4548	0.1471	0.0129
FL1	0.0202	0.1849	0.4586	0.6403	0.9999	0.9443	0.9244	0.9005	0.8055	0.5999	0.3647
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8090	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	0.0168	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.2091	0.9043	0.9991	0.9999	0.7935	0.6418	0.5907	0.5516	0.4483	0.2832	0.1665
D4	0.0962	0.6361	0.9518	0.9936	0.9199	0.8140	0.7601	0.7164	0.6577	0.5820	0.5067
D5	0.1582	0.8214	0.9943	0.9998	0.8232	0.7473	0.7262	0.7097	0.6851	0.6431	0.5926
D-BS	0.1890	0.8769	0.9981	0.9999	0.8490	0.7013	0.6723	0.6544	0.6076	0.4655	0.4655

4.10 California State Urban Levee Design Criteria

Although the California State Urban Levee Design Criteria (ULDC) is not a federal objective of the study, it is a local sponsor objective. Two options are offered in the ULDC requirements for determining if a levee meets the urban and urbanizing area levee system design. The freeboard option (option 1) requires 3 feet of freeboard above the mean 0.5% (1/200) ACE flood event. The risk and uncertainty option (option 2) allows for a lesser amount of freeboard (2 feet) if a high level of assurance (95%) can be demonstrated. The hydraulic performance of the no-action alternative relative to the ULDC requirements for 2070 conditions is provided in Table 31. The ULDC also requires minimum geotechnical design requirements. However, these are not accounted for in the assessment conducted for in the hydraulic analysis.

Table 31
Alternative 1 Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind-wave Run up and Setup (FT)	Minimum ULDC Required Freeboard	0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	33.9	10.6	10.6	30.0	3.9	99%
	LR4	RD17	33.9	2.4	3.0	30.0	3.9	99%
	LR3	RD17	31.0	2.4	3.0	25.6	5.4	99%
	LR2	RD17	27.8	2.4	3.0	23.0	4.8	99%
	LR1	RD17	25.0	2.4	3.0	22.6	2.4	92%
French Camp Slough	FR1-1	CS-02	21.8	<3.0	3.0	20.4	1.4	76%
	FL1	RD17	21.4	<3.0	3.0	20.4	1.0	70%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	13.2	<3.0	3.0	13.6	-0.4	45%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
FR1-2 is natural channel bank upstream of levee and levee criteria are not applicable.
LRTB assurance based on LR4 index point

5.0 ALTERNATIVE 7A

Alternative 7A provides flood risk reduction benefits to portions of North and Central Stockton economic impact areas. The alternative includes new delta front levee segments, Fix-in-Place levee segments along the Delta front and San Joaquin River, a closure structure at Fourteenmile Slough, and a closure structure at Smith Canal. A summary of the design features associated with Alternative 7A are described below and shown on Plate 51.

5.1 Hydraulic Design Summary

a. General Design. All project features, including improvements to existing features, would be designed to meet all current USACE design requirements. For example, if a levee was improved to meet slope requirements it must also meet seepage requirements etc. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 51. The levee height would be based on the authorized design profile, the existing profile, or three feet above the median 0.5% median water surface profile 2070 sea level conditions, whichever is higher. The models used to define the improvements assumed the levees in RD17 also met these height criteria. However improvements to the RD17 levees are not included in Alternative 7A and were not included in models used to assess the project performance. The height required to meet the height assumption was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 7A would extend the levee along the right bank of French Camp Slough upstream to the UPRR rail yard. The design height of new levees is described above. It was estimated that extension of the levee further upstream would require extensive modifications to the rail yard and result in a large increase in the cost relative to the benefits. Therefore, the performance of the levee was assumed to optimize at this configuration and further levee extension or height increases were not evaluated.

d. Upstream Reservoir Operation. Alternative 7A does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same as no-action conditions.

e. Interior Drainage Facilities. Alternative 7A does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions.

Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (Design of I-Walls, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The RD17 and French Camp slough tieback levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

As described above, this alternative would extend the levee along the right bank of French Camp Slough further upstream. However, the natural ground upstream of the levee would remain lower than the proposed levee extension to maintain levee superiority.

h. Erosion Protection. Rock revetment erosion protection would be placed along the proposed delta front levees with long fetches. The results of wind-wave analysis conducted for Alternative 7A are presented below. The assessment of the existing conditions indicated that the proposed levee improvement reaches did not require additional erosion protection. In some locations, existing erosion protection may be disturbed during construction of the levee improvements. Any disturbed erosion protection will be replaced to the same height and meet current design standards.

i. Diversion structures. Alternative 7A does not include any additional diversion structures beyond the no action alternative.

j. Closure Structures. Alternative 7A includes two closure structures.

(1) Smith Canal Closure Structure. A gate type closure structure would be constructed on Smith Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The proposed closure structures would consist of a fixed sheet pile wall structure with an opening gate structure to allow for navigation. The opening portion of the closure structure would be a 50' wide miter gate structure. As needed, a sheet pile floodwall would be constructed adjacent to the control structures to tie the structures into the adjacent levee or high ground areas

The structure would be closed during peak flood events when the stage reached approximately 8.0 feet NAVD88 or in the event of a levee breach along Smith Canal. The closure structures would prevent the extremely large volume of floodwaters in the Delta from flowing to the breach opening. As a result, the volume of floodwaters from a breach would be restricted to only the volume held in the canal.

(2) Fourteenmile Closure Structure. A gate type closure structure would also be constructed on Fourteenmile Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The structure design is similar to the Smith Canal closure structure.

5.2 Hydrology.

The hydrology associated with Alternative 7A is similar to Alternative 1 (no-action conditions).

5.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 7A were modified to reflect increased levee height within several reaches of the alternative. Height increases were limited to only the levees providing FRM to the study area. Stage and flow frequency curves are provided in Plates 31A through 31N and Plates 32A through 32E.

5.4 Wind-wave Analysis

Additional Wind-wave analysis was performed for the proposed delta front levee segments. The analysis was performed following the methods described in the no action plan. An assessment of stable rock diameter was also conducted to evaluate the potential for wind-wave erosion. The results of the wind-wave analysis are presented in Tables 32 and 33.

a. Delta Front – Shima Tract. This location is representative of Shima Tract reaches ST_10_R through ST_30_R, Fourteenmile slough reach FM_60_L, and Five mile Slough reach FS_10R. The wind-wave runup estimates assume a levee failure has occurred outside the proposed project reaches and Shima Tract has completely flooded. Based on the results of the wind-wave erosion analysis provided in Table 29, 1-foot median diameter rock revetment was specified along these levee segments.

b. Delta Front – Fourteenmile Slough. This location is representative of Fourteenmile Slough reaches FM_30_L and FM_40_L and Ten Mile Slough reach TS_30L. The wind-wave runup conditions assume a levee failure has occurred outside the proposed project reaches and Wright-Elmwood Tract has completely flooded. Based on the results of the wind-wave erosion analysis presented in Table 29, 1-foot median diameter rock revetment was specified along these levee segments.

Table 32
Stable Rock Revetment Sizes, Proposed Delta Front Levees

Representative Wind-wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Hs Significant Wave Height (Feet)	H10 10% Wave Height (Feet)	Stable Rock Revetment Size	
							Median Weight (lbs)	Median Diameter (Feet)
Delta Front-Fourteenmile Slough FM_30_L	1.3% 5% 20% 50% 95%	54 36 25 10 5	9300 ft	17.0 ft	2.2 ft 1.7 ft 1.0 ft 0.4 ft 0.2 ft	2.8 ft 2.2 ft 1.3 ft 0.5 ft 0.3 ft	121.7 lbs 56.1 lbs 11.4 lbs 0.7 lbs 0.1 lbs	1.0 ft 0.7 ft 0.4 ft 0.2 ft 0.09 ft
Delta Front-Shima Tract ST_20_R	1.3% 5% 20% 50% 95%	54 36 25 10 5	10100 ft	14.0 ft	2.3 ft 1.5 ft 1.1 ft 0.4 ft 0.2 ft	2.9 ft 1.9 ft 1.4 ft 0.5 ft 0.3 ft	139 lbs 38.6 lbs 15.2 lbs 0.7 lbs 0.1 lbs	1.0 ft 0.7 ft 0.5 ft 0.2 ft 0.09 ft

Notes:

* Wave Runup calculated using EurOtop method
**Stable Rock Size based on Hudson Method.

Table 33
Wind-wave Run-Up and Set Up Results, Alternative 7A

Representative Wind-wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3% 5% 20% 50% 95%	69 47 33 14 5	1900 ft	18.0 ft	2.36 ft 1.72 ft 1.28 ft 0.63 ft 0.26 ft	0.07 ft 0.03 ft 0.02 ft 0.0 ft 0.0 ft	1.0 ft 0.6 ft 0.3 ft 0.1 ft 0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3% 5% 20% 50% 95%	54 36 25 10 5	9300 ft	17.0 ft	2.7 ft 1.9 ft 1.4 ft 0.6 ft 0.3 ft	0.2 ft 0.1ft 0.0 ft 0.0 ft 0.0 ft	1.6 ft 1.0 ft 0.6 ft 0.1 ft 0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3% 5% 20% 50% 95%	54 36 25 10 5	10100 ft	14.0 ft	2.8 ft 2.0 ft 1.5 ft 0.6 ft 0.3 ft	0.3 ft 0.1 ft 0.1 ft 0.0 ft 0.0 ft	1.8 ft 1.0 ft 0.7 ft 0.1 ft 0.0 ft
RD17 Tieback SJR_200_R (Grass Lined)	1.3% 5% 20% 50% 95%	69 47 33 14 5	24300 ft	14.0 ft	9.5 ft 6.4 ft 4.4 ft 1.7 ft 0.5 ft	1.1 ft 0.4 ft 0.2 ft 0.0 ft 0.0 ft	7.2 ft 4.1 ft 2.3 ft 0.5 ft 0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

5.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 7A is similar to Alternative 1 (no action conditions).

5.6 Performance and Flood Risk

Flood risk to portions of North and Central Stockton would be reduced by Alternative 7A. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height for the FR1-2 breach location was modified to account for the extension of the French Camp Slough levee further upstream. The levee height at the D3 breach location was modified to account for levee height increases. These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate performance were identical to Alternative 1, the no action condition. The FDA input assumptions are described in Table 34. The performance of the project at index points throughout the study area is provided in Table 35.

Table 34
FDA Input for San Joaquin River Performance Calculations
Alternative 7A

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR4	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1-1	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	FR1-2	***Raise to 18.5 (b)***	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
	SL2	No Action	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 7A. The composite floodplains are provided in Plates 52 to 59. Table 35 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

c. Flood Velocities. Flood velocities for a levee beach would be similar to Alternative 1.

Table 35
Performance at Simulated Levee Breach Locations, Alternative 7A
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0117	0.0110	0.2973	0.4446	0.9999	0.9984	0.9918	0.8749	0.5090	0.1908	0.0384
LR4	0.0073	0.0706	0.1971	0.3064	0.9999	0.9731	0.9525	0.9241	0.8826	0.8423	0.8095
LR3	0.0095	0.0913	0.2496	0.3803	0.9999	0.9761	0.9394	0.8998	0.7938	0.6627	0.5650
LR2	0.0211	0.1923	0.4731	0.6563	0.9999	0.9289	0.8683	0.7922	0.6831	0.5788	0.5161
LR1	0.0126	0.1188	0.3158	0.4688	0.9999	0.9610	0.9400	0.8830	0.7439	0.5772	0.4620
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.9999	0.9999	0.9999	0.9970	0.9485	0.7612	0.5501
FR1-2	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.3260	0.0673
FL1	0.0132	0.1245	0.3290	0.4857	0.9999	0.9629	0.9460	0.9208	0.8269	0.6032	0.3857
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8148	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	.01680	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996

Cell shaded if assurance is less than criteria.
FR1-1 describes performance of levee
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.

d. Flood Warning Time. Alternative 7A will result in a significant increase in warning time to the population within North and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

5.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

USACE policy allows mitigation for induced flooding to be recommended as a project feature when it is economically justified or there are overriding reasons of safety, economic or social concerns, or a determination of a real estate taking has been made (ER 1105-2-100, para.3-

3.b.(5)). Based on the evaluation presented below it was determined that the changes were not significant and no mitigation features would be required.

a. Flood Depth. Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 7A includes fix in place levees, levee raises along the Delta Front, and an extension of French Camp slough levees upstream. Flood depths in the channel at all index points would be the same as the no action condition. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach. This condition would be similar for higher sea level conditions.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration. It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises and extension of French Camp slough levees upstream are unlikely to have hydraulic impacts that would impact flood frequency. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 36. Changes to AEP and assurance values are presented in Table 37. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 36
2010 Performance at Selected Locations, Alternative 7A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0113	0.1075	0.2892	0.4338	0.9999	0.9999	0.9957	0.8808	0.5134	0.1915	0.0374
LR4	0.0001	0.0007	0.0022	0.0037	0.9999	0.9999	0.9999	0.9998	0.9975	0.9858	0.9693
LR3	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9992	0.9982
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9998	0.9986	0.9972
LR1	0.0005	0.0050	0.0148	0.0245	0.9999	0.9999	0.9999	0.9990	0.9838	0.9251	0.8565
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.9999	0.9999	0.9999	0.9970	0.9485	0.7612	0.5501
FR1-2	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.2432	0.0673
FL1	0.0031	0.0306	0.0889	0.1437	0.9999	0.9999	0.9999	0.9964	0.9407	0.7268	0.4865
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0010	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9994	0.9994	0.9994	0.9994
Paradise Cut at I-5 F-PCI5	0.0014	0.0140	0.0415	0.0682	0.9999	0.9995	0.9995	0.9995	0.9995	0.9952	0.9779
Paradise Cut at Paradise Rd. F-PCPR	0.0017	0.0167	0.0492	0.0807	0.9999	0.9995	0.9995	0.9995	0.9995	0.9867	0.8641
SDWSC blw Burns Cutoff F-B95660	0.0002	0.0016	0.0049	0.0081	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.

FR1-1 describes performance of levee

FR1-2 describes performance associated with overtopping the natural ground upstream of levee.

SDWSC- Stockton Deep Water Ship Channel

Table 37
2010 Change in Performance at Selected Locations, Alternative 7A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0	0	0	0	0	0	0	0	0	0	0
LR4	0	0	0	0	0	0	0	0	0	0	0
LR3	0	0	0	0	0	0	0	0	0	0	0
LR2	0	0	0	0	0	0	0	0	0	0	0
LR1	0	0	0	0	0	0	0	0	0	0	0
French Camp Slough											
FR1-1	0	0	0	0	0	0	0	0	0	0	0
FR1-2	-0.0070	-0.0628	-0.1504	-0.2005	0	0.006	0.0449	0.1579	0.2853	0.0961	0.0544
FL1	0	0	0	0	0	0	0	0	0	0	0
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0	0	0	0	0	0	0	0	0	0	0
Calaveras River											
CR2	0	0	0	0	0	0	0	0	0	0	0
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0026	-0.0263	-0.0763	-0.1232	0	0.0017	0.0067	0.0175	0.0724	0.1960	0.3023
D4	0	0	0	0	0	0	0	0	0	0	0
D5	0	0	0	0	0	0	0	0	0	0	0
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry	0	0	0	0	0	0	0	0	0	0	0
Paradise Cut at I-5	0	0	0	0	0	0	0	0	0	0	0
Paradise Cut at Paradise Rd.	0	0	0	0	0	0	0	0	0	0	0
SDWSC blw Burns Cutoff	0	0	0	0	0	0	0	0	0	0	0

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.
FR1-1 describes performance of levee
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.
SDWSC- Stockton Deep Water Ship Channel

5.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 38. Composite floodplain maps were not developed for 2070 conditions.

Table 38
Performance at Simulated Levee Breach Locations, Alternative7A
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0118	0.1122	0.3002	0.4483	0.9999	0.9984	0.9912	0.8707	0.5026	0.4440	0.5153
LR4	0.0075	0.0726	0.2023	0.3139	0.9999	0.9725	0.9509	0.9228	0.8819	0.8417	0.8093
LR3	0.0101	0.0968	0.2632	0.3990	0.9999	0.9715	0.9362	0.8962	0.7875	0.6593	0.5652
LR2	0.0257	0.2295	0.5426	0.7285	0.9999	0.9153	0.8415	0.7718	0.6711	0.5788	0.5153
LR1	0.0141	0.1326	0.3475	0.5091	0.9999	0.9567	0.9334	0.8764	0.7412	0.5757	0.4616
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.999	0.9999	0.9999	0.9970	0.9485	0.7611	0.5501
FR1-2	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.3260	0.0673
FL1	0.0202	0.1849	0.4586	0.6403	0.9999	0.9443	0.9244	0.9005	0.8055	0.5790	0.3647
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8148	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	0.0168	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.0021	0.0207	0.0608	0.9992	0.9999	0.9968	0.9919	0.9830	0.9331	0.8107	0.6974
D4	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9999	0.9992	0.9952	0.9826	0.9642
D5	0.0005	0.0047	0.0139	0.0231	0.9999	0.9998	0.9992	0.9965	0.9831	0.9402	0.8794
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9969	0.9938

5.9 California State Urban Levee Design Criteria

The hydraulic performance of Alternative 7A relative to the ULDC requirements for 2070 conditions is provided in Table 39.

Table 39
Alternative 7A Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind-wave Run up and Setup (FT)	Minimum ULDC Required Freeboard	0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	33.9	10.6	10.6	30.0	3.9	99%
	LR4	RD17	33.9	2.4	3.0	30.0	3.9	99%
	LR3	RD17	31.0	2.4	3.0	25.6	5.4	99%
	LR2	RD17	27.8	2.4	3.0	23.0	4.8	99%
	LR1	RD17	25.0	2.4	3.0	22.6	2.4	93%
French Camp Slough	FR1-1	CS-02	21.8	<3.0	3.0	20.4	1.4	72%
	FL1	RD17	21.4	<3.0	3.0	20.4	1.0	70%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	13.6	1.3	81%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
FR1-2 is natural channel bank upstream of levee and levee criteria are not applicable.
LRTB assurance based on LR4 index point

6.0 ALTERNATIVE 7B

Alternative 7B is similar to 7A but includes additional levee fixes in RD17 and improvements to the RD17 tieback levee. A summary of the design features associated with Alternative 7B are described below and shown on Plate 60.

6.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 60. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 7B would extend and raise the RD17 tieback levee at Walthall Slough. The levee would be extended to where the natural ground elevation was equivalent to the 0.5% (1/200) ACE median water surface. The design height of new levees is described above. The extension of Duck Creek levees described in Alternative 7A would not be included in this alternative.

d. Upstream Reservoir Operation. Alternative 7B does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same way as the no-action alternative.

e. Interior Drainage Facilities. Alternative 7B does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (Design of I-Walls, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The RD17 and French Camp slough tieback levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

h. Erosion Protection. Erosion protection would be similar to Alternative 7A. However, additional rock revetment erosion protection would be placed along the RD17 tieback levee to address wind-wave erosion. A high erosion potential was identified for several reaches of RD17 during the evaluation of existing conditions. Placement of additional rock revetment within these reaches should be evaluated in greater detail if this alternative is selected as the recommended plan. The results of wind-wave analysis conducted for Alternative 7B are presented below.

i. Diversion structures. Alternative 7B does not include any additional diversion structures beyond the no action alternative.

j. Closure Structures.

(1) Smith Canal Closure Structure. The Smith Canal Closure Structure is the same as Alternative 7A.

(2) Fourteenmile Closure Structure. The Fourteenmile Closure Structure is the same as Alternative 7A.

6.2 Hydrology.

The hydrology associated with Alternative 7B is similar to Alternative 1 (no-action conditions).

6.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 7B were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area. Levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

6.4 Wind-wave Analysis

Additional Wind-wave analysis was performed for the RD17 tieback levee assuming a rock lined slope. The analysis was performed following the methods described in the no action plan. The wind-wave estimates for Alternative 7B are provided in Table 40.

Table 40
Wind-wave Run-Up and Set Up Results, Alternative 7B

Representative Wind-wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3% 5% 20% 50% 95%	69 47 33 14 5	1900 ft	18.0 ft	2.36 ft 1.72 ft 1.28 ft 0.63 ft 0.26 ft	0.07 ft 0.03 ft 0.02 ft 0.0 ft 0.0 ft	1.0 ft 0.6 ft 0.3 ft 0.1 ft 0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3% 5% 20% 50% 95%	54 36 25 10 5	9300 ft	17.0 ft	2.7 ft 1.9 ft 1.4 ft 0.6 ft 0.3 ft	0.2 ft 0.1ft 0.0 ft 0.0 ft 0.0 ft	1.6 ft 1.0 ft 0.6 ft 0.1 ft 0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3% 5% 20% 50% 95%	54 36 25 10 5	10100 ft	14.0 ft	2.8 ft 2.0 ft 1.5 ft 0.6 ft 0.3 ft	0.3 ft 0.1 ft 0.1 ft 0.0 ft 0.0 ft	1.8 ft 1.0 ft 0.7 ft 0.1 ft 0.0 ft
RD17 Tieback SJR_200_R (Rock Lined)	1.3% 5% 20% 50% 95%	69 47 33 14 5	24300 ft	14.0 ft	5.2 ft 3.5 ft 2.4 ft 0.9 ft 0.3 ft	1.1 ft 0.4 ft 0.2 ft 0.0 ft 0.0 ft	4.5 ft 2.4 ft 1.4 ft 0.3 ft 0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

6.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 7B is similar to Alternative 1 (no action conditions).

6.6 Performance and Flood Risk

Flood risk to portions of RD17, North Stockton, and Central Stockton would be reduced by Alternative 7B. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the the with-project fragility curves so they had no probability of failure until overtopped. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). The levee height of the LRTB index point was modified to account for the extension of the RD17 tieback levee. These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of

alternatives. All other inputs to calculate assurance were the same as Alternative 1, the no action condition.
The FDA input assumptions are described in Table 41. The performance of the project at index points throughout the study area is provided in Table 42.

Table 41
FDA Input for San Joaquin River Performance Calculations
Alternative 7B

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	***Raise to 34.9***	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR4	***Raise to 34.9***	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1-1	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	FR1-2	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	Scenario B	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	No Action	Scenario B	MS at Bellota	EPR = 52 yrs
	SL2	No Action	No Action	Scenario B	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	No Action	Scenario B	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	No Action	Scenario B	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 7B. The composite floodplains are provided in Plates 61 to 68. Table 42 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

c. Flood Velocities. Flood velocities for a levee beach would be similar to Alternative 1.

Table 42
Assurance at Simulated Levee Breach Locations, Alternative 7B
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9906	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9999	0.9987	0.9954	0.9917
French Camp Slough											
FR1-1	0.0001	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9995	0.9990
FR1-2	0.0012	0.1137	0.3037	0.4530	0.9999	0.9938	0.9549	0.8333	0.5886	0.3618	0.2332
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8148	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	.01680	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0012	0.0019	0.9999	0.9999	0.9999	0.9999	0.9998	0.9997	0.9996

FR1-1 describes performance of levee
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.
Cell shaded if assurance is less than criteria.

d. Flood Warning Time. Alternative 7B will result in a significant increase in warning time to the population within RD17, North Stockton, and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

6.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

USACE policy allows mitigation for induced flooding to be recommended as a project feature when it is economically justified or there are overriding reasons of safety, economic or social concerns, or a determination of a real estate taking has been made (ER 1105-2-100, para.3-

3.b.(5)). Based on the evaluation presented below it was determined that the changes were not significant and no mitigation features would be required.

a. Flood Depth. Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 7B includes fix in place levees, levee raises along the Delta Front, and upstream extension of the RD17 tieback levee. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures.

It is unlikely that improvements along French Camp Slough would increase water levels. For these increases to occur a breach of the San Joaquin levee would have had to already occur and the area would already be flooded. Improvement to the RD17 tieback levee was found to increase stages for events larger than 1% ACE for index points along the San Joaquin River, Old River, Middle River, and Paradise cut. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration. It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises are unlikely to impact flood frequency. However, improvements to the RD17 tieback levee would impact stages for events more rare than 1% ACE. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 43. Changes to AEP and assurance values are presented in Table 44. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 43
2010 Performance at Selected Locations, Alternative 7B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9983	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9990	0.9987	0.9951	0.9917
French Camp Slough											
FR1-1	0.0001	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9995	0.9990
FR1-2	0.0012	0.1137	0.3037	0.4530	0.9999	0.9939	0.9549	0.8333	0.5886	0.3618	0.2332
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0002	0.0023	0.0067	0.0112	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Paradise Cut at I-5 F-PCI5	0.0024	0.0240	0.0703	0.1143	0.9999	0.9995	0.9995	0.9995	0.9986	0.9952	0.5404
Paradise Cut at Paradise Rd. F-PCPR	0.0038	0.0376	0.1085	0.1743	0.9999	0.9995	0.9995	0.9995	0.9993	0.6660	0.1373
SDWSC blw Burns Cutoff F-B95660	0.0001	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.
FR1-1 describes performance of levee
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.
SDWSC- Stockton Deep Water Ship Channel

Table 44
2010 Change in Performance at Selected Locations, Alternative 7B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	-0.011	-0.1041	-0.2791	-0.417	0	0	0.0042	0.1187	0.4754	0.8019	0.817
LR4	0.0002	0.0027	0.0079	0.0131	0	0	0	-0.0003	-0.0087	-0.0527	-0.1149
LR3	0	0.0004	0.0013	0.0022	0	0	0	0	-0.0016	-0.0009	-0.0201
LR2	0	0	0	0	0	0	0	0	0	0.0003	0.0006
LR1	-0.0005	-0.0047	-0.0138	-0.0228	0	0	0	0	0.0149	0.0700	0.1352
French Camp Slough											
FR1-1	-0.0025	-0.0261	-0.0762	-0.1238	0	0	0	0.0029	0.0513	0.2383	0.4489
FR1-2	-0.0136	-0.0244	-0.056	-0.0713	0	0	0.0004	0.0233	0.1338	0.2147	0.2203
FL1	-0.0031	-0.0302	-0.0876	-0.1415	0	0	0	0.0035	0.0591	0.2728	0.5128
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0	0	0	0	0	0	0	0	0	0	0
Calaveras River											
CR2	0	0	0	0	0	0	0	0	0	0	0
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0029	-0.0285	-0.083	-0.1344	0	0.0017	0.0067	0.0182	0.0821	0.2366	0.3784
D4	0	0	0	0	0	0	0	0	0	0	0
D5	0	0	0	0	0	0	0	0	0	0	0
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	-0.0008	0.0013	0.0037	0.0062	0	0	0	0.0001	0.0001	0.0001	0.0001
Paradise Cut at I-5 F-PCI5	0.001	0.01	0.0288	0.0461	0	0	0	0	-0.0009	0	-0.4375
Paradise Cut at Paradise Rd. F-PCPR	0.0021	0.0209	0.0593	0.0936	0	0	0	0	-0.0002	-0.3207	-0.7268
SDWSC blw Burns Cutoff F-B95660	-0.0001	-0.0006	-0.0019	-0.0031	0	0	0	0	0	0	0

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.
FR1-1 describes performance of levee
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.
SDWSC- Stockton Deep Water Ship Channel

6.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 45. Composite floodplain maps were not developed for 2070 conditions.

Table 45
Performance at Simulated Levee Breach Locations, Alternative7B
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR4	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR3	0.0000	0.0002	0.0005	0.0008	0.9999	0.9999	0.9999	0.9999	0.9995	0.9983	0.9781
LR2	0.0000	0.0005	0.0014	0.0024	0.9999	0.9999	0.9999	0.9999	0.9996	0.9993	0.9991
LR1	0.0013	0.0128	0.0380	0.0626	0.9999	0.9999	0.9999	0.9958	0.9554	0.8735	0.8231
French Camp Slough											
FR1-1	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9995	0.9990
FR1-2	0.0120	0.1137	0.3037	0.4530	0.9999	0.9938	0.9549	0.8333	0.5886	0.3619	0.2332
FL1	0.0000	0.0001	0.0003	0.0010	0.9999	0.9999	0.9999	0.9999	0.9997	0.9987	0.9987
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8090	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	0.0168	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.0001	0.0099	0.0294	0.0485	0.9999	0.9967	0.9917	0.9873	0.9824	0.9777	0.9742
D4	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9999	0.9992	0.9952	0.9826	0.9642
D5	0.0005	0.0047	0.0139	0.0231	0.9999	0.9998	0.9992	0.9965	0.9831	0.9402	0.8794
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9969	0.9938

6.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 7B relative to the ULDC requirements for 2070 conditions is provided in Table 46.

Table 46
Alternative 7B Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind-wave Run up and Setup (FT)	Minimum ULDC Required Freeboard	0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	38.2	6.3	6.3	31.9	6.3	99%
	LR4	RD17	34.9	2.4	3.0	31.9	3.0	99%
	LR3	RD17	31.0	2.4	3.0	27.0	4.0	99%
	LR2	RD17	27.8	2.4	3.0	22.7	5.1	99%
	LR1	RD17	25.0	2.4	3.0	20.8	4.2	87%
French Camp Slough	FR1-1	CS-02	21.8	<3.0	3.0	16.8	5.0	99%
	FL1	RD17	21.4	<3.0	3.0	16.8	4.6	99%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	11.9	3.0	98%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
FR1-2 is natural channel bank upstream of levee and levee criteria are not applicable.
LRTB assurance based on LR4 index point

7.0 ALTERNATIVE 8A

Alternative 8A provides flood risk reduction benefits to portions of North and Central Stockton economic impact areas. The alternative includes new delta front levee segments, Fix-in-Place levee segments along the Delta front and San Joaquin River, a closure structure at Fourteenmile Slough, and a closure structure at Smith Canal. The alternative also includes levee improvements to the Calaveras River and Stockton Diverting Canal. A summary of the design features associated with Alternative 8A are described below and shown on Plate 69.

7.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 69. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The models used to define the height of the levee improvements assumed the levees in RD17 also met ULDC requirements. However improvements to the RD17 levees are not included in Alternative 8A and were not included in models used to assess the project performance. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 8A would extend the levee along the right bank of French Camp Slough upstream along Duck Creek to higher ground near the UPRR rail yard. The design height of new levees is described above. It was estimated that extension of the levee further upstream of the UPRR rail yard would require extensive modifications to the rail yard and result in a large increase in the cost relative to the benefits. Therefore, the performance of the levee was assumed to optimize at this configuration and further levee extension or height increases were not evaluated.

d. Upstream Reservoir Operation. Alternative 8A does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same as no-action conditions.

e. Interior Drainage Facilities. Alternative 8A does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions.

Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (Design of I-Walls, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The RD17 and French Camp slough tieback levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

As described above, this alternative would extend the levee along the right bank of French Camp Slough further upstream. However, the natural ground upstream of the levee would remain lower than the proposed levee extension to maintain levee superiority.

h. Erosion Protection. Rock revetment erosion protection would be placed along the proposed delta front levees with long fetches. The results of wind-wave analysis conducted for Alternative 8A are presented below. The assessment of the existing conditions indicated that the proposed levee improvement reaches did not require additional erosion protection. In some locations, existing erosion protection may be disturbed during construction of the levee improvements. Any disturbed erosion protection will be replaced to the same height and meet current design standards.

i. Diversion structures. Alternative 8A does not include any additional diversion structures beyond the no action alternative.

j. Closure Structures.

(1) Smith Canal Closure Structure. A gate type closure structure would be constructed on Smith Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The proposed closure structures would consist of a fixed sheet pile wall structure with an opening gate structure to allow for navigation. The opening portion of the closure structure would be a 50' wide miter gate structure. As needed, a sheet pile floodwall would be constructed adjacent to the control structures to tie the structures into the adjacent levee or high ground areas.

The structure would be closed during peak flood events when the stage reached approximately 8.0 feet NAVD88 or in the event of a levee breach along Smith Canal. The closure structures would prevent the extremely large volume of floodwaters in the Delta from flowing to the breach opening. As a result, the volume of floodwaters from a breach would be restricted to only the volume held in the canal.

(2) Fourteenmile Closure Structure. A gate type closure structure would also be constructed on Fourteenmile Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The structure design is similar to the Smith Canal closure structure.

7.2 Hydrology.

The hydrology associated with Alternative 8A is similar to Alternative 1 (no-action conditions).

7.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 8A were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area and assume the upstream levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

7.4 Wind-wave Analysis

The wind-wave analysis performed for Alternative 7A is applicable to Alternative 8A. No additional analysis was required to address the additional Calaveras River and Diverting Canal reaches in Alternative 8A because of the relatively short fetch lengths. The estimated wind-wave runup results are presented in Table 47.

Table 47
Wind-wave Run-Up and Set Up Results, Alternative 8A

Representative Wind-wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Grass Lined)	1.3%	69	24300 ft	14.0 ft	9.5 ft	1.1 ft	7.2 ft
	5%	47			6.4 ft	0.4 ft	4.1 ft
	20%	33			4.4 ft	0.2 ft	2.3 ft
	50%	14			1.7 ft	0.0 ft	0.5 ft
	95%	5			0.5 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

7.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 8A is similar to Alternative 1 (no action conditions).

7.6 Performance and Flood Risk

Flood risk to portions of North and Central Stockton would be reduced by Alternative 8A. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height for the FR1-2 breach location was modified to account for the extension of the French Camp Slough levee further upstream along Duck Creek. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate assurance were the same as Alternative 1, the no action condition. The FDA input assumptions are described in Table 48. The performance of the project at index points throughout the study area is provided in Table 49.

Table 48
FDA Input for San Joaquin River Performance Calculations
Alternative 8A

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR4	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1	***Raise to 18.5 (b)***	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	***No Fragility***	Scenario A	MS at Bellota	EPR = 52 yrs
	SL2	No Action	***No Fragility***	Scenario A	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	***No Fragility***	Scenario A	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	***No Fragility***	Scenario A	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 8A. The composite floodplains are provided in Plates 70 to 77. Table 49 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

c. Flood Velocities. Flood velocities for a levee beach would be similar to Alternative 1.

Table 49
Performance at Simulated Levee Breach Locations, Alternative 8A
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0117	0.0110	0.2973	0.4446	0.9999	0.9984	0.9918	0.8749	0.5090	0.1908	0.0384
LR4	0.0073	0.0706	0.1971	0.3064	0.9999	0.9731	0.9525	0.9241	0.8826	0.8423	0.8095
LR3	0.0095	0.0913	0.2496	0.3803	0.9999	0.9761	0.9394	0.8998	0.7938	0.6627	0.5650
LR2	0.0211	0.1923	0.4731	0.6563	0.9999	0.9289	0.8683	0.7922	0.6831	0.5788	0.5161
LR1	0.0126	0.1188	0.3158	0.4688	0.9999	0.9610	0.9400	0.8830	0.7439	0.5772	0.4620
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.9999	0.9999	0.9999	0.9970	0.9485	0.7612	0.5501
FR1-2	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.3260	0.0673
FL1	0.0132	0.1245	0.3290	0.4857	0.9999	0.9629	0.9460	0.9208	0.8269	0.6032	0.3857
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9828
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Cell shaded if assurance is less than criteria.											

d. Flood Warning Time. Alternative 8A will result in a significant increase in warning time to the population within North and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

7.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

USACE policy allows mitigation for induced flooding to be recommended as a project feature when it is economically justified or there are overriding reasons of safety, economic or social concerns, or a determination of a real estate taking has been made (ER 1105-2-100, para.3-3.b.(5)). Based on the evaluation presented below it was determined that the changes were not significant and no mitigation features would be required.

a. Flood Depth. Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 8A includes fix in place levees, levee raises along the Delta Front, and an extension of French Camp slough levees upstream along Duck Creek. Flood depths in the channel at all index points would be the same as the no action condition. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration. It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises and extension of French Camp slough levees upstream are unlikely to have hydraulic impacts that would impact flood frequency. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 50. Changes to AEP and assurance values are presented in Table 51. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 50
2010 Performance at Selected Locations, Alternative 8A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0113	0.1075	0.2892	0.4338	0.9999	0.9999	0.9957	0.8808	0.5134	0.1915	0.0374
LR4	0.0001	0.0007	0.0022	0.0037	0.9999	0.9999	0.9999	0.9998	0.9975	0.9858	0.9693
LR3	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9992	0.9982
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9998	0.9986	0.9972
LR1	0.0005	0.0050	0.0148	0.0245	0.9999	0.9999	0.9999	0.9990	0.9838	0.9251	0.8565
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.9999	0.9999	0.9999	0.9970	0.9485	0.7612	0.5501
FR1-2	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.2432	0.0673
FL1	0.0031	0.0306	0.0889	0.1437	0.9999	0.9999	0.9999	0.9964	0.9407	0.7268	0.4865
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0010	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9994	0.9994	0.9994	0.9994
Paradise Cut at I-5 F-PCI5	0.0014	0.0140	0.0415	0.0682	0.9999	0.9995	0.9995	0.9995	0.9995	0.9952	0.9779
Paradise Cut at Paradise Rd. F-PCPR	0.0017	0.0167	0.0492	0.0807	0.9999	0.9995	0.9995	0.9995	0.9995	0.9867	0.8641
SDWSC blw Burns Cutoff F-B95660	0.0002	0.0016	0.0049	0.0081	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Assurance estimates account for stage uncertainty, hydrologic uncertainty only.											
FR1-1 describes performance of levee											
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.											
SDWSC- Stockton Deep Water Ship Channel											

Table 51
2010 Change in Performance at Selected Locations, Alternative 8A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0	0	0	0	0	0	0	0	0	0	0
LR4	0	0	0	0	0	0	0	0	0	0	0
LR3	0	0	0	0	0	0	0	0	0	0	0
LR2	0	0	0	0	0	0	0	0	0	0	0
LR1	0	0	0	0	0	0	0	0	0	0	0
French Camp Slough											
FR1-1	0	0	0	0	0	0	0	0	0	0	0
FR1-2	-0.0070	-0.0628	-0.1504	-0.2005	0	0.006	0.0449	0.1579	0.2853	0.0961	0.0544
FL1	0	0	0	0	0	0	0	0	0	0	0
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0	0	0	0	0	0	0	0	0	0	0
Calaveras River											
CR2	0	0	0	0	0	0	0	0	0	0	0
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0026	-0.0263	-0.0763	-0.1232	0	0.0017	0.0067	0.0175	0.0724	0.196	0.3023
D4	0	0	0	0	0	0	0	0	0	0	0
D5	0	0	0	0	0	0	0	0	0	0	0
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	0	0	0	0	0	0	0	0	0	0	0
Paradise Cut at I-5 F-PCI5	0	0	0	0	0	0	0	0	0	0	0
Paradise Cut at Paradise Rd. F-PCPR	0	0	0	0	0	0	0	0	0	0	0
SDWSC blw Burns Cutoff F-B95660	0	0	0	0	0	0	0	0	0	0	0
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. FR1-1 describes performance of levee FR1-2 describes performance associated with overtopping the natural ground upstream of levee. SDWSC- Stockton Deep Water Ship Channel											

7.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 52. Composite floodplain maps were not developed for 2070 conditions.

Table 52
Performance at Simulated Levee Breach Locations, Alternative 8A
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0118	0.1122	0.3002	0.4483	0.9999	0.9984	0.9912	0.8707	0.5026	0.4440	0.5153
LR4	0.0075	0.0726	0.2023	0.3139	0.9999	0.9725	0.9509	0.9228	0.8819	0.8417	0.8093
LR3	0.0101	0.0968	0.2632	0.3990	0.9999	0.9715	0.9362	0.8962	0.7875	0.6593	0.5652
LR2	0.0257	0.2295	0.5426	0.7285	0.9999	0.9153	0.8415	0.7718	0.6711	0.5788	0.5153
LR1	0.0141	0.1326	0.3475	0.5091	0.9999	0.9567	0.9334	0.8764	0.7412	0.5757	0.4616
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.999	0.9999	0.9999	0.9970	0.9485	0.7611	0.5501
FR1-2	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.3260	0.0673
FL1	0.0202	0.1849	0.4586	0.6403	0.9999	0.9443	0.9244	0.9005	0.8055	0.5999	0.3647
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.0	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9828
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0021	0.0207	0.0608	0.9992	0.9999	0.9968	0.9919	0.9830	0.9331	0.8107	0.6974
D4	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9999	0.9992	0.9952	0.9826	0.9642
D5	0.0005	0.0047	0.0139	0.0231	0.9999	0.9998	0.9992	0.9965	0.9831	0.9402	0.8794
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9969	0.9938

7.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 8A relative to the ULDC requirements for 2070 conditions is provided in Table 53.

Table 53
Alternative 8A Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind-wave Run up (FT)	Minimum ULDC Required Freeboard	0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	33.9	10.6	10.6	30.0	3.9	99%
	LR4	RD17	33.9	2.4	3.0	30.0	3.9	99%
	LR3	RD17	31.0	2.4	3.0	25.6	5.4	99%
	LR2	RD17	27.8	2.4	3.0	23.0	4.8	99%
	LR1	RD17	25.0	2.4	3.0	22.6	2.4	93%
French Camp Slough	FR1-1	CS-02	21.8	<3.0	3.0	20.4	1.4	76%
	FL1	RD17	21.4	<3.0	3.0	20.4	1.0	70%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	13.6	1.3	81%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
FR1-2 is natural channel bank upstream of levee and levee criteria are not applicable.
LRTB assurance based on LR4 index point

8.0 ALTERNATIVE 8B

Alternative 8B is similar to 8A but includes additional levee fixes in RD17. A summary of the design features associated with Alternative 8B are described below and shown on Plate 78.

8.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 78. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 8B would extend and raise the RD17 tieback levee at Walthall Slough. The design height of new levees is described above. The levee would be extended to where the natural ground elevation was equivalent to the 0.5% (1/200) ACE median water surface. The extension of French Camp Slough levees described in Alternative 8A would not be included in this alternative.

d. Upstream Reservoir Operation. Alternative 8B does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same way as the no-action alternative.

e. Interior Drainage Facilities. Alternative 8B does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (Design of I-Walls, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority.

The RD17 and French Camp slough tieback levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

h. Erosion Protection. Erosion protection would be similar to Alternative 8A. However, additional rock revetment erosion protection would be placed along the RD17 tieback levee to address wind-wave erosion. A high erosion potential was identified for several reaches of RD17 during the evaluation of existing conditions. Placement of additional rock revetment within these reaches should be evaluated in greater detail if this alternative is selected as the recommended plan. The results of wind-wave analysis conducted for Alternative 8B are presented below.

i. Diversion structures. Alternative 8B does not include any additional diversion structures beyond the no action alternative.

j. Closure Structures.

(1) Smith Canal Closure Structure. The Smith Canal Closure Structure is the same as Alternative 8A.

(2) Fourteenmile Closure Structure. The Fourteenmile Closure Structure is the same as Alternative 8A.

8.2 Hydrology.

The hydrology associated with Alternative 8B is similar to Alternative 1 (no-action conditions).

8.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 8B were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area. Levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

8.4 Wind-wave Analysis

The wind-wave analysis performed for Alternative 7A and 7B is applicable to Alternative 8B. No additional analysis was required to address the additional Calaveras River and Diverting Canal reaches in Alternative 8B because of the relatively short fetch lengths. The wind-wave estimates for Alternative 8B are provided in Table 54.

Table 54
Wind-wave Run-Up and Set Up Results, Alternative 8B

Representative Wind-wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Rock Lined)	1.3%	69	24300 ft	14.0 ft	5.2 ft	1.1 ft	4.5 ft
	5%	47			3.5 ft	0.4 ft	2.4 ft
	20%	33			2.4 ft	0.2 ft	1.4 ft
	50%	14			0.9 ft	0.0 ft	0.3 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

8.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 8B is similar to Alternative 1 (no action conditions).

8.6 Performance and Flood Risk

Flood risk to portions of RD17, North Stockton, and Central Stockton would be reduced by Alternative 8B. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). The levee height of the LRTB index point was modified to account for the extension of the RD17 tieback levee. These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate assurance were the same as Alternative 1, the no action condition. The performance of the project at index points throughout the study area is provided in Table 55.

Table 55
FDA Input for San Joaquin River Performance Calculations
Alternative 8B

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	***Raise to 34.9***	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR4	***Raise to 34.9***	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1-1	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	FR1-2	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	***No Fragility***	Scenario B	MS at Bellota	EPR = 52 yrs
	SL2	No Action	***No Fragility***	Scenario B	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	***No Fragility***	Scenario B	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	***No Fragility***	Scenario B	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 8B. The composite floodplains are provided in Plates 79 to 86. Table 56 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

d. Flood Velocities. Flood velocities for a levee beach would be similar to Alternative 1.

Table 56
Performance at Simulated Levee Breach Locations, Alternative 8B
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9898	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9999	0.9987	0.9951	0.9917
French Camp Slough											
FR1-1	0.0001	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9995	0.9990
FR1-2	0.0012	0.1137	0.3037	0.4530	0.9999	0.9938	0.9549	0.8333	0.5886	0.3619	0.2332
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9999	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9998	0.9984	0.9912	0.9828
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0012	0.0019	0.9999	0.9999	0.9999	0.9999	0.9998	0.9997	0.9996

e. Flood Warning Time. Alternative 8B will result in a significant increase in warning time to the population within RD17, North Stockton, and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

8.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

USACE policy allows mitigation for induced flooding to be recommended as a project feature when it is economically justified or there are overriding reasons of safety, economic or social concerns, or a determination of a real estate taking has been made (ER 1105-2-100, para.3-3.b.(5)). Based on the evaluation presented below it was determined that the changes were not significant and no mitigation features would be required.

a. Flood Depth. Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 8B includes fix in place levees, levee raises along the Delta Front, and upstream extension of the RD17 tieback levee. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures.

It is unlikely that improvements along French Camp Slough would increase water levels. For these increases to occur a breach of the San Joaquin levee would have had to already occur and the area would already be flooded. Improvement to the RD17 tieback levee was found to increase stages for events larger than 1% ACE for index points along the San Joaquin River, Old River, Middle River, and Paradise cut. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration. It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises are unlikely to impact flood frequency. However, improvements to the RD17 tieback levee would impact stages for events more rare than 1% ACE. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 57. Changes to AEP and assurance values are presented in Table 58. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 57
2010 Performance at Selected Locations, Alternative 8B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9898	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9990	0.9987	0.9951	0.9917
French Camp Slough											
FR1-1	0.0001	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9995	0.9990
FR1-2	0.0012	0.1137	0.3037	0.4530	0.9999	0.9939	0.9549	0.8333	0.5886	0.3618	0.2332
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0002	0.0023	0.0067	0.0112	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Paradise Cut at I-5 F-PCI5	0.0024	0.0240	0.0703	0.1143	0.9999	0.9995	0.9995	0.9995	0.9986	0.8753	0.5404
Paradise Cut at Paradise Rd. F-PCPR	0.0038	0.0376	0.1085	0.1743	0.9999	0.9995	0.9995	0.9995	0.9993	0.6660	0.1373
SDWSC blw Burns Cutoff F-B95660	0.0001	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.

SDWSC- Stockton Deep Water Ship Channel

Table 58
2010 Change in Performance at Selected Locations, Alternative 8B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	-0.011	-0.1041	-0.2791	-0.417	0	0	0.0042	0.1187	0.4754	0.7416	0.817
LR4	0.0002	0.0027	0.0079	0.0131	0	0	0	-0.0003	-0.0087	-0.0527	-0.1149
LR3	0	0.0004	0.0013	0.0022	0	0	0	0	-0.0016	-0.0094	-0.0201
LR2	0	0	0	0	0	0	0	0	0	0.0003	0.0006
LR1	-0.0005	-0.0047	-0.0138	-0.0228	0	0	0	0	0.0149	0.07	0.1352
French Camp Slough											
FR1-1	-0.0025	-0.0261	-0.0762	-0.1238	0	0	0	0.0029	0.0513	0.2383	0.4489
FR1-2	-0.0136	-0.0244	-0.056	-0.0713	0	0	0.0004	0.0233	0.1338	0.2147	0.2203
FL1	-0.0031	-0.0302	-0.0876	-0.1415	0	0	0	0.0035	0.0591	0.2728	0.5128
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0	0	0	0	0	0	0	0	0	0	0
Calaveras River											
CR2	0	0	0	0	0	0	0	0	0	0	0
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0029	-0.0285	-0.083	-0.1344	0	0.0017	0.0067	0.0182	0.0821	0.2366	0.3784
D4	0	0	0	0	0	0	0	0	0	0	0
D5	0	0	0	0	0	0	0	0	0	0	0
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	-0.0008	0.0013	0.0037	0.0062	0	0	0	0.0001	0.0001	0.0001	0.0001
Paradise Cut at I-5 F-PCI5	0.001	0.01	0.0288	0.0461	0	0	0	0	-0.0009	-0.1199	-0.4375
Paradise Cut at Paradise Rd. F-PCPR	0.0021	0.0209	0.0593	0.0936	0	0	0	0	-0.0002	-0.3207	-0.7268
SDWSC blw Burns Cutoff F-B95660	-0.0001	-0.0006	-0.0019	-0.0031	0	0	0	0	0	0	0

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.
FR1-1 describes performance of levee
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.
SDWSC- Stockton Deep Water Ship Channel

8.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 59. Composite floodplain maps were not developed for 2070 conditions.

Table 59
Performance at Simulated Levee Breach Locations, Alternative 8B
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR4	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR3	0.0000	0.0002	0.0005	0.0008	0.9999	0.9999	0.9999	0.9999	0.9995	0.9983	0.9781
LR2	0.0000	0.0005	0.0014	0.0024	0.9999	0.9999	0.9999	0.9999	0.9996	0.9993	0.9991
LR1	0.0013	0.0128	0.0380	0.0626	0.9999	0.9999	0.9999	0.9958	0.9554	0.8735	0.8231
French Camp Slough											
FR1-1	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9995	0.9990
FR1-2	0.0120	0.1137	0.3037	0.4530	0.9999	0.9938	0.9549	0.8333	0.5886	0.3619	0.2332
FL1	0.0000	0.0001	0.0003	0.0010	0.9999	0.9999	0.9999	0.9999	0.9997	0.9992	0.9987
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9828
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0021	0.0207	0.0608	0.9992	0.9999	0.9968	0.9919	0.9830	0.9824	0.9777	0.6974
D4	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9999	0.9992	0.9952	0.9826	0.9642
D5	0.0005	0.0047	0.0139	0.0231	0.9999	0.9998	0.9992	0.9965	0.9831	0.9402	0.8794
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9996	0.9938

8.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 8B relative to the ULDC requirements for 2070 conditions is provided in Table 60.

Table 60
Alternative 8B Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind-wave Run up (FT)	Minimum ULDC Required Freeboard	0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	38.2	6.3	6.3	31.9	6.3	99%
	LR4	RD17	34.9	2.4	3.0	31.9	3.0	99%
	LR3	RD17	31.0	2.4	3.0	27.0	4.0	99%
	LR2	RD17	27.8	2.4	3.0	22.7	5.1	99%
	LR1	RD17	25.0	2.4	3.0	20.8	4.2	87%
French Camp Slough	FR1-1	CS-02	21.8	<3.0	3.0	16.8	5.0	99%
	FL1	RD17	21.4	<3.0	3.0	16.8	4.6	99%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	11.9	3.0	98%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
FR1-2 is natural channel bank upstream of levee and levee criteria are not applicable.
LRTB assurance based on LR4 index point

9.0 ALTERNATIVE 9A

Alternative 9A provides flood risk reduction benefits to portions of North and Central Stockton economic impact areas. The alternative includes new delta front levee segments, Fix-in-Place levee segments along the Delta front and San Joaquin River, a closure structure at Fourteenmile Slough, and a closure structure at Smith Canal. The alternative also includes a diversion structure to divert floodwaters from the Stockton diverting canal into Old Mormon Slough (Mormon Slough Bypass) and channel improvements to safely convey those flows to the Stockton Deep Water Ship Channel. A summary of the design features associated with Alternative 9A are described below and shown on Plate 87.

9.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 87. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The models used to define the improvements assumed the levees in RD17 also met ULDC requirements. However improvements to the RD17 levees are not included in Alternative 9A and were not included in models used to assess the project performance. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 9A would extend the levee along the right bank of French Camp Slough upstream along Duck Creek to higher ground near the UPRR rail yard. The design height of new levees is described above. It was estimated that extension of the levee further upstream would require extensive modifications to the rail yard and result in a large increase in the cost relative to the benefits. Therefore, the performance of the levee was assumed to optimize at this configuration and further levee extension or height increases were not evaluated.

d. Upstream Reservoir Operation. Alternative 9A does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same as no-action conditions.

e. Interior Drainage Facilities. Alternative 9A does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (Design of I-Walls, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The RD17 and French Camp slough tieback levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

As described above, this alternative would extend the levee along the right bank of French Camp Slough further upstream along Duck Creek. However, the natural ground upstream of the levee would remain lower than the proposed levee extension to maintain levee superiority.

h. Erosion Protection. Rock revetment erosion protection would be placed along the proposed delta front levees with long fetches. The results of wind-wave analysis conducted for Alternative 9A are presented below. The assessment of the existing conditions indicated that the proposed levee improvement reaches did not require additional erosion protection. In some locations, existing erosion protection may be disturbed during construction of the levee improvements. Any disturbed erosion protection will be replaced to the same height and meet current design standards.

i. Diversion structures. The design includes of a diversion structure to divert floodwaters from the Stockton Diverting canal into Old Mormon Slough (Mormon Slough Bypass) and channel improvements to safely convey those flows to the Stockton Deep Water Ship Channel. The diversion structure would consist of an inlet apron, series of 8 radial gates, a box culvert, and outlet apron. A maximum flood flow diversion rate of 1,200cfs was selected based on the ability of downstream channel improvements to pass this flow including additional localized runoff with 90% assurance of not overtopping. The design flow, allowing for localized inflow, is 1,200cfs from the diversion structure to Highway 99, 1,550cfs from Highway 99 to Stanislaus Street, and 1,700 cfs from Stanislaus Street to the Deep Water Ship Channel. The design includes no levees along the bypass. The selected design of the downstream improvements was estimated to maximize economic benefits because a larger size would require a substantial increase in the scale of improvements.

j. Closure Structures.

(1) Smith Canal Closure Structure. A gate type closure structure would be constructed on Smith Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The proposed closure structures would consist of a fixed sheet pile wall structure with an opening gate structure to allow for navigation. The opening portion of the closure structure would be a 50' wide miter gate structure. As needed, a sheet pile floodwall would be constructed adjacent to the control structures to tie the structures into the adjacent levee or high ground areas

The structure would be closed during peak flood events when the stage reached approximately 8.0 feet NAVD88 or in the event of a levee breach along Smith Canal. The closure structures would prevent the extremely large volume of floodwaters in the Delta from flowing to the breach opening. As a result, the volume of floodwaters from a breach would be restricted to only the volume held in the canal.

(2) Fourteenmile Closure Structure. A gate type closure structure would also be constructed on Fourteenmile Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The structure design is similar to the Smith Canal closure structure.

9.2 Hydrology.

The diversion into the Mormon Slough Bypass would change the flood flow frequency for the Stockton Diverting Canal, Lower Calaveras River. The estimated flow diversion is described in Table 61. Inflow to the diversion was based on flow at the SL2 index point for the no action alternative.

Table 61
Estimated Flood Flow Frequency of Mormon Slough Bypass

Parameter	Annual Chance Exceedance						
	50% ACE	**10% ACE**	**4% ACE**	**2% ACE**	**1% ACE**	**0.5% ACE**	**0.2% ACE**
Inflow to Proposed Diversion (CFS)	3740	9650	11920	12720	14810	15200	18240
Flow to Stockton Diverting Canal (CFS)	3740	8450	10720	11510	13610	14000	17240
Flow to Mormon Bypass (CFS)	0	1200	1200	1200	1200	1200	1200
Average Duration of Diversion (Days)	0	5	8	9	11	12	14
Diversion flows obtained from PBI, 2013C							

9.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 9A were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area. It was assumed the upstream levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

9.4 Wind-wave Analysis

The wind-wave analysis performed for Alternative 7A is applicable to Alternative 9A. No additional analysis was required to address the additional Calaveras River and Diverting Canal reaches or Mormon Slough Bypass in Alternative 9A because of the relatively short fetch lengths. The estimated wind-wave runup results are presented in Table 62.

Table 62
Wind-wave Run-Up and Set Up Results, Alternative 9A

Representative Wind-wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Grass Lined)	1.3%	69	24300 ft	14.0 ft	9.5 ft	1.1 ft	7.2 ft
	5%	47			6.4 ft	0.4 ft	4.1 ft
	20%	33			4.4 ft	0.2 ft	2.3 ft
	50%	14			1.7 ft	0.0 ft	0.5 ft
	95%	5			0.5 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

9.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 9A is similar to Alternative 1 (no action conditions) for all locations except the Stockton Deep Water Ship Channel. The proposed project could increase sediment deposition in the Turning Basin of the Stockton Ship Channel. Although the proposed diversion will likely divert negligible bed load, it will divert suspended load. This material size will likely be transported in the higher transport capacity reaches of the proposed bypass without deposition. However, it will likely fall out of suspension in the low transport capacity ship channel turning basin. Without any analysis it should be assumed that about half of the suspended sediment in the diverted flood flows would be deposited in the ship channel turning basin. This estimate could be used to estimate the potential for additional O&M dredging in the turning basin associated with the proposed diversion.

9.6 Performance and Flood Risk

Flood risk to portions of North and Central Stockton would be reduced by Alternative 9A. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height for the FR1-2 breach location was modified to account for the extension of the French Camp Slough levee further upstream along Duck Creek. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate assurance were the same as Alternative 1, the no action condition. The FDA input assumptions are described in Table 63. The performance of the project at index points throughout the study area is provided in Table 64.

Table 63
FDA Input for San Joaquin River Performance Calculations
Alternative 9A

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR4	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1-1	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	FR1-2	***Raise to 18.5 (b)***	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	Scenario A	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
	SL2	No Action	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	No Action	Scenario A	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	Scenario A	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 9A. The composite floodplains are provided in Plates 88 to 96. Table 57provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

c. Flood Velocities. Flood velocities for a levee beach would be identical to Alternative 1.

Table 64
Performance at Simulated Levee Breach Locations, Alternative 9A
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0117	0.0110	0.2973	0.4446	0.9999	0.9984	0.9918	0.8749	0.5090	0.1908	0.0384
LR4	0.0073	0.0706	0.1971	0.3064	0.9999	0.9731	0.9525	0.9241	0.8826	0.8423	0.8095
LR3	0.0095	0.0913	0.2496	0.3803	0.9999	0.9761	0.9394	0.8998	0.7938	0.6627	0.5650
LR2	0.0211	0.1923	0.4731	0.6563	0.9999	0.9289	0.8683	0.7922	0.6831	0.5788	0.5161
LR1	0.0126	0.1188	0.3158	0.4688	0.9999	0.9610	0.9400	0.8830	0.7439	0.5772	0.4620
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.9999	0.9999	0.9999	0.9970	0.9485	0.7612	0.5501
FR1-2	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.3260	0.0673
FL1	0.0132	0.1245	0.3290	0.4857	0.9999	0.9629	0.9460	0.9208	0.8269	0.6032	0.3857
Stockton Diverting Canal											
SL1	0.0089	0.0859	0.2363	0.3619	0.9999	0.9670	0.9661	0.9606	0.9469	0.9262	0.9057
SL2	0.0109	0.1036	0.2797	0.4211	0.9999	0.9700	0.9432	0.9194	0.8897	0.8480	0.8029
Calaveras River											
CR2	0.0051	0.0497	0.1419	0.2251	0.9999	0.9916	0.9619	0.9320	0.8920	0.8444	0.7965
CL2	0.0145	0.1361	0.3552	0.5187	0.9999	0.9577	0.9533	0.9374	0.9110	0.8813	0.8536
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0000	0.0003	0.0008	0.0014	0.9999	0.9999	0.9999	0.9999	0.9995	0.9909	0.9950
D5	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9999	0.9998	0.9986	0.9799	0.9864
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996

Cell shaded if assurance is less than criteria.
FR1-1 describes performance of levee
FR1-2 describes performance associated with overtopping the natural ground upstream of levee.

d. Flood Warning Time. Alternative 9A will result in a significant increase in warning time to the population within North and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

9.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

USACE policy allows mitigation for induced flooding to be recommended as a project feature when it is economically justified or there are overriding reasons of safety, economic or social concerns, or a determination of a real estate taking has been made (ER 1105-2-100, para.3-3.b.(5)). Based on the evaluation presented below it was determined that the changes were not significant and no mitigation features would be required.

a. Flood Depth. Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 9A includes fix in place levees, levee raises along the Delta Front, and diversion of flood flows into Old Mormon Slough. Flood depths in the channel at all index points would be the same as the no action condition except the Stockton Diverting Canal and Lower Calaveras River. Stages in the Stockton Diverting Canal and Lower Calaveras River would be lowered because of the upstream diversion to Old Mormon Slough. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures. For magnitudes greater than 33% (1/3) ACE, stages in Old Mormon Slough would be increased due to the upstream diversion. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration. It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises and extension of French Camp slough levees upstream along Duck Creek are unlikely to have hydraulic impacts that would impact flood frequency. The frequency of flood flows in Old Mormon Slough would be increased due to the upstream diversion. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 65. Changes to AEP and assurance values are presented in Table 66. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 65
2010 Performance at Selected Locations, Alternative 9A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0113	0.1075	0.2892	0.4338	0.9999	0.9999	0.9957	0.8808	0.5134	0.1915	0.0374
LR4	0.0001	0.0007	0.0022	0.0037	0.9999	0.9999	0.9999	0.9998	0.9975	0.9858	0.9693
LR3	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9992	0.9982
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9998	0.9986	0.9972
LR1	0.0005	0.0050	0.0148	0.0245	0.9999	0.9999	0.9999	0.9990	0.9838	0.9251	0.8565
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.9999	0.9999	0.9999	0.9970	0.9485	0.7612	0.5501
FR1-2	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.2432	0.0673
FL1	0.0031	0.0306	0.0889	0.1437	0.9999	0.9999	0.9999	0.9964	0.9407	0.7268	0.4865
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0001	0.0007	0.0021	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Calaveras River											
CR2	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9997	0.9985	0.9963
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9999	0.9998	0.9986	0.9939	0.9864
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0010	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9994	0.9994	0.9994	0.9994
Paradise Cut at I-5 F-PCI5	0.0014	0.0140	0.0415	0.0682	0.9999	0.9995	0.9995	0.9995	0.9995	0.9952	0.9779
Paradise Cut at Paradise Rd. F-PCPR	0.0017	0.0167	0.0492	0.0807	0.9999	0.9995	0.9995	0.9995	0.9995	0.9867	0.8641
SDWSC blw Burns Cutoff F-B95660	0.0002	0.0016	0.0049	0.0081	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.
SDWSC- Stockton Deep Water Ship Channel

Table 66
2010 Change in Performance at Selected Locations, Alternative 9A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0	0	0	0	0	0	0	0	0	0	0
LR4	0	0	0	0	0	0	0	0	0	0	0
LR3	0	0	0	0	0	0	0	0	0	0	0
LR2	0	0	0	0	0	0	0	0	0	0	0
LR1	0	0	0	0	0	0	0	0	0	0	0
French Camp Slough											
FR1-1	0	0	0	0	0	0	0	0	0	0	0
FR1-2	-0.0070	-0.0628	-0.1504	-0.2005	0	0.006	0.0449	0.1579	0.2853	0.0961	0.0544
FL1	0	0	0	0	0	0	0	0	0	0	0
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0.0001	0.0005	0.0015	0.0024	0	0	0	0	1E-04	0.0007	0.0022
Calaveras River											
CR2	-0.0001	-0.0004	-0.001	-0.0016	0	0	0	1E-04	0.0013	0.0061	0.0134
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0026	-0.0263	-0.0763	-0.1232	0	0.0017	0.0067	0.0175	0.0724	0.196	0.3023
D4	0	0	0	0	0	0	0	0	0	0	0
D5	-0.0001	-0.0009	-0.0025	-0.0042	0	0	0	0.0004	0.0035	0.014	0.03
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	0	0	0	0	0	0	0	0	0	0	0
Paradise Cut at I-5 F-PCI5	0	0	0	0	0	0	0	0	0	0	0
Paradise Cut at Paradise Rd. F-PCPR	0	0	0	0	0	0	0	0	0	0	0
SDWSC blw Burns Cutoff F-B95660	0	0	0	0	0	0	0	0	0	0	0
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. SDWSC- Stockton Deep Water Ship Channel											

9.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 67. Composite floodplain maps were not developed for 2070 conditions.

Table 67
Performance at Simulated Levee Breach Locations, Alternative9A
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0118	0.1122	0.3002	0.4483	0.9999	0.9984	0.9912	0.8707	0.5026	0.4440	0.5153
LR4	0.0075	0.0726	0.2023	0.3139	0.9999	0.9725	0.9509	0.9228	0.8819	0.8417	0.8093
LR3	0.0101	0.0968	0.2632	0.3990	0.9999	0.9715	0.9362	0.8962	0.7875	0.6593	0.5652
LR2	0.0257	0.2295	0.5426	0.7285	0.9999	0.9153	0.8415	0.7718	0.6711	0.5788	0.5153
LR1	0.0141	0.1326	0.3475	0.5091	0.9999	0.9567	0.9334	0.8764	0.7412	0.5757	0.4616
French Camp Slough											
FR1-1	0.0026	0.0262	0.0765	0.1243	0.999	0.9999	0.9999	0.9970	0.9485	0.7611	0.5501
FR1-2	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.3260	0.0673
FL1	0.0202	0.1849	0.4586	0.6403	0.9999	0.9443	0.9244	0.9005	0.8055	0.5790	0.3647
Stockton Diverting Canal											
SL1	0.0089	0.0859	0.2363	0.3619	0.9999	0.9670	0.9661	0.9606	0.9469	0.9262	0.9057
SL2	0.0109	0.1036	0.2797	0.4211	0.9999	0.9700	0.9432	0.9194	0.8897	0.8480	0.8029
Calaveras River											
CR2	0.0051		0.1419	0.2251	0.9999	0.9916	0.9619	0.9320	0.8921	0.8444	0.7965
CL2	0.0145	0.1361	0.3552	0.5187	0.9999	0.9577	0.9533	0.9374	0.9110	0.8813	0.8536
Delta Front											
D3	0.0021	0.0207	0.0608	0.9992	0.9999	0.9968	0.9919	0.9830	0.9331	0.8107	0.6974
D4	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9997	0.9983	0.9826	0.9861
D5	0.0002	0.0019	0.0058	0.0096	0.9999	0.9999	0.9997	0.9987	0.9932	0.9753	0.9482
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9969	0.9938

9.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 9A relative to the ULDC requirements for 2070 conditions is provided in Table 68.

Table 68
Alternative 9A Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind-wave Run up (FT)	Minimum ULDC Required Freeboard	0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	33.9	10.6	10.6	30.0	3.9	99%
	LR4	RD17	33.9	2.4	3.0	30.0	3.9	99%
	LR3	RD17	31.0	2.4	3.0	25.6	5.4	99%
	LR2	RD17	27.8	2.4	3.0	23.0	4.8	99%
	LR1	RD17	25.0	2.4	3.0	22.6	2.4	93%
French Camp Slough	FR1-1	CS-02	21.8	<3.0	3.0	20.4	1.4	76%
	FL1	RD17	21.4	<3.0	3.0	20.4	1.0	70%
	FL1	RD17	21.4	<3.0	3.0	20.4	1.0	70%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	29.8	9.4	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.3	5.3	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	25.1	4.6	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.0	5.4	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	13.6	1.3	81%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
FR1-2 is natural channel bank upstream of levee and levee criteria are not applicable.
LRTB assurance based on LR4 index point

10.0 ALTERNATIVE 9B

Alternative 9B is similar to 9A but includes additional levee fixes in RD17. A summary of the design features associated with Alternative 9B are described below and shown on Plate 96.

10.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 96. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 9B would extend and raise the RD17 tieback levee at Walthall Slough. The levee would be extended to where the natural ground elevation was equivalent to the 0.5% (1/200) ACE median water surface. The design height of new levees is described above. The extension of French Camp Slough levees described in Alternative 9A would not be included in this alternative.

d. Upstream Reservoir Operation. Alternative 9B does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same way as the no-action alternative.

e. Interior Drainage Facilities. Alternative 9B does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (Design of I-Walls, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority.

The RD17 and French Camp slough tieback levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

h. Erosion Protection. Erosion protection would be similar to Alternative 9A. However, additional rock revetment erosion protection would be placed along the RD17 tieback levee to address wind-wave erosion. A high erosion potential was identified for several reaches of RD17 during the evaluation of existing conditions. Placement of additional rock revetment within these reaches should be evaluated in greater detail if this alternative is selected as the recommended plan. The results of wind-wave analysis conducted for Alternative 9B are presented below.

i. Diversion structures. Alternative 9B does not include any additional diversion structures beyond the no action alternative.

j. Smith Canal Closure Structure. The Smith Canal Closure Structure is the same as Alternative 9A.

j. Fourteenmile Closure Structure. The Fourteenmile Closure Structure is the same as Alternative 9A.

10.2 Hydrology.

The diversion into the Mormon Slough Bypass would change the flood flow frequency for the Stockton Diverting Canal, Lower Calaveras River. The estimated flow diversion is described in Table 69. Inflow to the diversion was based on flow at the SL2 index point for the no action alternative.

Table 69
Estimated Flood Flow Frequency of Mormon Slough Bypass

Parameter	Annual Chance Exceedance						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
Inflow to Proposed Diversion (CFS)	3740	9650	11920	12720	14810	15200	18240
Flow to Stockton Diverting Canal (CFS)	3740	8450	10720	11510	13610	14000	17240
Flow to Mormon Bypass (CFS)	0	1200	1200	1200	1200	1200	1200
Average Duration of Diversion (Days)	0	5	8	9	11	12	14
Diversion flows obtained from PBI, 2013C							

10.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 9B were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area. Levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

10.4 Wind-wave Analysis

The wind-wave analysis performed for Alternative 7A and 7B is applicable to Alternative 9B. No additional analysis was required to address the additional Calaveras River, Diverting Canal, and Mormon Slough Bypass Reaches in Alternative 9B because of the relatively short fetch lengths. The wind-wave estimates for Alternative 7B are provided in Table 70.

Table 70: Wind-wave Run-Up and Set Up Results, Alternative 9B

Representative Wind-wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Rock Lined)	1.3%	69	24300 ft	14.0 ft	5.2 ft	1.1 ft	4.5 ft
	5%	47			3.5 ft	0.4 ft	2.4 ft
	20%	33			2.4 ft	0.2 ft	1.4 ft
	50%	14			0.9 ft	0.0 ft	0.3 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

10.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 9B is similar to Alternative 1 (no action conditions) for all locations except the Stockton Deep Water Ship Channel. The proposed project could increase sediment deposition in the Turning Basin of the Stockton Ship Channel. Although the proposed diversion will likely divert negligible bed load, it will divert suspended load. This material size will likely be transported in the higher transport capacity reaches of the proposed bypass without deposition. However, it will likely fall out of suspension in the low transport capacity ship channel turning basin. Without any analysis it should be assumed that about half of the suspended sediment in the diverted flood flows would be deposited in the ship channel turning basin. This estimate could be used to estimate the potential for additional O&M dredging in the turning basin associated with the proposed diversion.

10.6 Performance and Flood Risk

Flood risk to portions of RD17, North Stockton, and Central Stockton would be reduced by Alternative 9B. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). The levee height of the LRTB index point was modified to account for the extension of the RD17 tieback levee. These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate assurance were the same as Alternative 1, the no action condition. The FDA input assumptions are described in Table 71. The performance of the project at index points throughout the study area is provided in Table 72.

Table 71
FDA Input for San Joaquin River Performance Calculations
Alternative 9B

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	***Raise to 34.9***	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR4	***Raise to 34.9***	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1-1	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	FR1-2	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	Scenario B	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	No Action	Scenario B	MS at Bellota	EPR = 52 yrs
	SL2	No Action	No Action	Scenario B	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	No Action	Scenario B	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	No Action	Scenario B	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	Scenario B	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 9B. The composite floodplains are provided in Plates 98 to 104. Table 69 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

d. Flood Velocities. Flood velocities for a levee beach would be similar to Alternative 1.

Table 72
Performance at Simulated Levee Breach Locations, Alternative 9B
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9898	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9999	0.9987	0.9951	0.9917
French Camp Slough											
FR1-1	0.0001	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9995	0.9990
FR1-2	0.0012	0.1137	0.3037	0.4530	0.9999	0.9938	0.9549	0.8333	0.5886	0.3618	0.2332
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0089	0.0859	0.2363	0.3619	0.9999	0.9670	0.9661	0.9606	0.9469	0.9262	0.9057
SL2	0.0166	0.1540	0.3945	0.5666	0.9999	0.9700	0.9432	0.9194	0.8897	0.8480	0.8029
Calaveras River											
CR2	0.0051	0.0497	0.1419	0.2251	0.9999	0.9916	0.9619	0.9320	0.8921	0.8349	0.7965
CL2	0.0145	0.1361	0.3552	0.5187	0.9999	0.9577	0.9533	0.9374	0.9110	0.8813	0.8536
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0000	0.0003	0.0008	0.0014	0.9999	0.9999	0.9999	0.9999	0.9995	0.9978	0.9950
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9999	0.9998	0.9986	0.9939	0.9864

e. Flood Warning Time. Alternative 9B will result in a significant increase in warning time to the population within RD17, North Stockton, and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

10.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

USACE policy allows mitigation for induced flooding to be recommended as a project feature when it is economically justified or there are overriding reasons of safety, economic or social concerns, or a determination of a real estate taking has been made (ER 1105-2-100, para.3-3.b.(5)). Based on the evaluation presented below it was determined that the changes were not significant and no mitigation features would be required.

a. Flood Depth. Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 9B includes fix in place levees, levee raises along the Delta Front, upstream extension of the RD17 tieback levee, and diversion of flood flows into Old Mormon Slough. Flood depths in the channel at all index points would be the same as the no action condition except the Stockton Diverting Canal and Lower Calaveras River. Stages in the Stockton Diverting Canal and Lower Calaveras River would be lowered because of the upstream diversion to Old Mormon Slough. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures. For magnitudes greater than 33% (1/3) ACE, stages in Old Mormon Slough would be increased due to the upstream diversion. Stages in Old Mormon Slough would be increased due to the upstream diversion.

It is unlikely that improvements along the delta front levees would increase water levels from delta sources. Improvement to the RD17 tieback levee was found to increase stages for events larger than 1% ACE for index points along the San Joaquin River, Old River, Middle River, and Paradise cut. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration. It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises are unlikely to have hydraulic impacts that would impact flood frequency. However, improvements to the RD17 tieback levee would impact stages for events more rare than 1% ACE. The frequency of flood flows in the Old Mormon Slough would be increased due to the upstream diversion. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 73. Changes to AEP and assurance values are presented in Table 74. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 73
2010 Performance at Selected Locations, Alternative 9B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9898	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9990	0.9987	0.9251	0.9917
French Camp Slough											
0.0001	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9995	0.9990	0.0001
0.0012	0.1137	0.3037	0.4530	0.9999	0.9939	0.9549	0.8333	0.5886	0.3618	0.2332	0.0012
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0001	0.0007	0.0021	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Calaveras River											
CR2	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9997	0.9985	0.9963
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0000	0.0003	0.0008	0.0014	0.9999	0.9999	0.9999	0.9999	0.9995	0.9978	0.9950
D5	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9999	0.9998	0.9986	0.9939	0.9864
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0002	0.0023	0.0067	0.0112	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Paradise Cut at I-5 F-PCI5	0.0024	0.0240	0.0703	0.1143	0.9999	0.9995	0.9995	0.9995	0.9986	0.8753	0.5404
Paradise Cut at Paradise Rd. F-PCPR	0.0038	0.0376	0.1085	0.1743	0.9999	0.9995	0.9995	0.9995	0.9993	0.6660	0.1373
SDWSC blw Burns Cutoff F-B95660	0.0001	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. SDWSC- Stockton Deep Water Ship Channel											

Table 74
2010 Change in Performance at Selected Locations, Alternative 9B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	-0.011	-0.1041	-0.2791	-0.417	0	0	0.0042	0.1187	0.4754	0.7416	0.817
LR4	0.0002	0.0027	0.0079	0.0131	0	0	0	-0.0003	-0.0087	-0.0527	-0.1149
LR3	0	0.0004	0.0013	0.0022	0	0	0	0	-0.0016	-0.0094	-0.0201
LR2	0	0	0	0	0	0	0	0	-1E-04	0.0003	0.0006
LR1	-0.0005	-0.0047	-0.0138	-0.0228	0	0	0	0	0.0149	0	0.1352
French Camp Slough											
FR1-1	-0.0025	-0.0261	-0.0762	-0.1238	0	0	0	0.0029	0.0513	0.2383	0.4489
FR1-2	-0.0136	-0.0244	-0.056	-0.0713	0	0	0.0004	0.0233	0.1338	0.2147	0.2203
FL1	-0.0031	-0.0302	-0.0876	-0.1415	0	0	0	0.0035	0.0591	0.2728	0.5128
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0.0001	0.0005	0.0015	0.0024	0	0	0	0	1E-04	0.0007	0.0022
Calaveras River											
CR2	-0.0001	-0.0004	-0.001	-0.0016	0	0	0	1E-04	0.0013	0.0061	0.0134
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0029	-0.0285	-0.083	-0.1344	0	0.0017	0.0067	0.0182	0.0821	0.2366	0.3784
D4	-0.0001	-0.0004	-0.0012	-0.002	0	0	0	1E-04	0.0015	0.0069	0.0151
D5	-0.0001	-0.0009	-0.0025	-0.0042	0	0	0	0.0004	0.0035	0.014	0.03
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	-0.0008	0.0013	0.0037	0.0062	0	0	0	0.0001	0.0001	0.0001	0.0001
Paradise Cut at I-5 F-PCI5	0.001	0.01	0.0288	0.0461	0	0	0	0	-0.0009	-0.1199	-0.4375
Paradise Cut at Paradise Rd. F-PCPR	0.0021	0.0209	0.0593	0.0936	0	0	0	0	-0.0002	-0.3207	-0.7268
SDWSC blw Burns Cutoff F-B95660	-0.0001	-0.0006	-0.0019	-0.0031	0	0	0	0	0	0	0
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. SDWSC- Stockton Deep Water Ship Channel											

10.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 75. Composite floodplain maps were not developed for 2070 conditions.

Table 75
Performance at Simulated Levee Breach Locations, Alternative 9B
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR4	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR3	0.0000	0.0002	0.0005	0.0008	0.9999	0.9999	0.9999	0.9999	0.9995	0.9983	0.9781
LR2	0.0000	0.0005	0.0014	0.0024	0.9999	0.9999	0.9999	0.9999	0.9996	0.9993	0.9991
LR1	0.0013	0.0128	0.0380	0.0626	0.9999	0.9999	0.9999	0.9958	0.9554	0.8735	0.8231
French Camp Slough											
FR1-1	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9995	0.9990
FR1-2	0.0120	0.1137	0.3037	0.4530	0.9999	0.9938	0.9549	0.8333	0.5886	0.3619	0.2332
FL1	0.0000	0.0001	0.0003	0.0010	0.9999	0.9999	0.9999	0.9999	0.9997	0.9992	0.9987
Stockton Diverting Canal											
SL1	0.0089	0.0859	0.2363	0.3619	0.9999	0.9670	0.9661	0.9606	0.9469	0.9262	0.9057
SL2	0.0109	0.1036	0.2797	0.4211	0.9999	0.9700	0.9432	0.9194	0.8897	0.8480	0.8029
Calaveras River											
CR2	0.0051	0.0497	0.1419	0.2251	0.9999	0.9916	0.9619	0.9320	0.8920	0.8444	0.7965
CL2	0.0145	0.1361	0.3552	0.5187	0.9999	0.9577	0.9533	0.9374	0.9110	0.8813	0.8536
Delta Front											
D3	0.0010	0.0099	0.0294	0.0485	0.9999	0.9967	0.9917	0.9873	0.9824	0.9777	0.9742
D4	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9997	0.9983	0.9934	0.9861
D5	0.0002	0.0019	0.0058	0.0096	0.9999	0.9999	0.9997	0.9987	0.9932	0.9655	0.9482
D-BS	0.0000	0.0004	0.0012	0.0020	0.9999	0.9999	0.9999	0.9998	0.9997	0.9996	0.9996

10.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 9B relative to the ULDC requirements for 2070 conditions is provided in Table 76.

Table 76
Alternative 9B Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind-wave Run up (FT)	Minimum ULDC Required Freeboard	0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	38.2	6.3	6.3	31.9	6.3	99%
	LR4	RD17	34.9	2.4	3.0	31.9	3.0	99%
	LR3	RD17	31.0	2.4	3.0	27.0	4.0	99%
	LR2	RD17	27.8	2.4	3.0	22.7	5.1	99%
	LR1	RD17	25.0	2.4	3.0	20.8	4.2	87%
French Camp Slough	FR1-1	CS-02	21.8	<3.0	3.0	16.8	5.0	99%
	FL1	RD17	21.4	<3.0	3.0	16.8	4.6	99%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	29.8	9.4	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.3	5.3	99%
-Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	25.1	4.6	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.0	5.4	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	11.9	3.0	98%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
FR1-2 is natural channel bank upstream of levee and levee criteria are not applicable.
LRTB assurance based on LR4 index point

11.0 RECOMMENDED PLAN (ALTERNATIVE 7A)

Alternative 7A was selected as the recommended plan. Comparison and selection of the recommended plan is described in the feasibility study report. A description of the recommended plan is described in Section 5 above. The sections below describe refinements made to the Recommended Plan to address feasibility study design requirements and support a Class III cost estimate. Recommendations for further analysis during PED are included.

11.1 Hydraulic Design Summary

a. General Design. The general design of the recommended plan is described in Section 5 of this Addendum. However, a refinement to the extent of the levee improvements was made in the final feasibility design. Reach MC_30_L on Mosher Slough was not included in the recommended plan because it does not meet the minimum flow requirements to establish federal interest. This was also based on additional hydraulic analysis that showed that backwater from the Sacramento San Joaquin Delta would not extend upstream into the MC_30_L reach. Reach CR_70_L was added to the recommended plan to be consistent with the left bank improvements and meet the intended performance. A map of the project features and levee segments is provided in Plate 105. A map of project station lines for each reach is provided in Plate 106.

b. Levee Design Height. Profiles of the proposed levee improvements were developed to describe the recommended plan in more detail. The profiles are provided on Plate 106 and include the existing ground elevation at the levee toe, top of levee, and water surface elevations for a range of flood event magnitudes. As required by USACE guidance for risk and uncertainty based designs, a single design water surface and freeboard value is not provided. The station values shown on the profile refer to Plate 106. These stations were developed for the feasibility study and do not correspond to the project stations shown on as-built plans of existing levee reaches.

It is recommended that refined hydrologic and hydraulic analysis be considered in the determination of the final levee design heights during PED. During the development of this study the California Department of Water Resources (DWR) was in the process of conducting an evaluation of Delta stage frequency estimates using more detailed hydrodynamic modeling. However, the results were not available for this study. It is recommended that the downstream boundary condition be evaluated relative to the final results from the DWR study. DWR was also reevaluating hydrologic frequency analysis and hydraulic models. It is recommended that these models and results also be reviewed. Since the recommended plan is the NED plan, the levee design profile should be modified during PED as necessary to meet the R&U performance values described in this report while maximizing the Net Benefits.

c. New Levees. Additional refined analysis was not deemed necessary for this design assumption. The extent of new levee reaches did not require refinements.

d. Upstream Reservoir Operation. Additional refined analysis was not deemed necessary for this design assumption.

e. Interior Drainage Facilities. Additional analysis was performed for the recommended plan to evaluate the impact of design features on interior drainage.

A more detailed evaluation of the potential impacts of Smith Canal or Fourteenmile Closure Structures on interior drainage was conducted. The coincident probability of having a closed gate structure preventing flow from interior drainage sources was evaluated. The analysis included a comparison of historical peak stages where the gate would have been closed in comparison to coincident local rainfall amounts. The analysis indicated no correlation. It was also found that if the gate was closed during an interior flood event, it could be opened if the interior stages exceeded the exterior stage. An open gate was estimated to have less than 0.1 foot of stage impact on the interior drainage even for extremely rare flood events.

The extension of the French Camp Slough levee upstream along Duck Creek was evaluated relative to interior drainage. It was determined that several additional culverts with flap gates would be necessary to allow floodwaters to drain from the interior into Duck Creek. It was estimated that the cost of these features would be relatively small and economically feasible. Therefore, detailed analysis was not necessary for the feasibility study and was deferred to PED.

f. Operation and Maintenance. Operation and Maintenance considerations related specifically to hydraulic design aspects include measures to address levee crest subsidence and stream gage maintenance.

The design elevation would be maintained by the sponsor through normal operation and maintenance activities over the 100 year project life. As part of Operation and Maintenance the sponsor would be required to verify the crest elevation by conducting a high order survey every 10-years to update the National Levee Database. The sponsor would be required to restore the levee profile if it was found to have subsided more than 0.5 feet. This approach to addressing subsidence related issues is described as the “Managed adaptive approach” in ETL 1100-2-1. It is estimated the crest elevation would need to be restored every 25 years for reaches that subsided at the high rate and 50 years for reaches that subsided at the medium rate. No restoration would be anticipated for reaches that subsided at the low rate. A graphical example of the OMRR&R approach is provided in Plate 108.

It is also recommended that O&M requirements for stream gages necessary for the operation of the project be addressed during PED. O&M requirements are described in the civil design addendum.

g. Levee Superiority. As described in the evaluation of the Final Array, Alternative 7A was designed to include a level of levee superiority that allows overflow upstream of French Camp Slough, providing increased resiliency to floods in excess of design. An additional qualitative assessment of the impact of sea level rise on levee superiority was conducted for the recommended plan. Higher sea level conditions would result in higher stages along the proposed Delta Front Levee segments for smaller more frequent events. However the impacts are likely to be smaller for the larger more infrequent events because the proposed Delta Front Levee will have higher (superior) overtopping performance than the other existing levees in the Delta.

h. Erosion Protection. Additional refined analysis was not deemed necessary for this design assumption because it was found to be a very minor cost consideration relative to the other project costs. Most of the project reach is currently armored with bank protection to address boat wakes. This armor would be replaced upon completion of the levee improvements. It is recommended that the need for additional localized erosion protection be addressed during PED. Examples of minor erosion protection include wing walls, bridge abutments, and the Smith Canal and Fourteen mile closure structures. It is recommended that a full range of potential upstream and downstream stage and flow combinations be considered in the design of erosion protection at the closure structures. Scour could be a major issue through the closure structure(s) during some flow combinations and would need to be adequately designed and maintained.

i. Diversion structures. The recommended plan does not include any diversion structures.

f. Closure structures.

(1) Smith Canal Closure Structure. Additional refined analysis was not deemed necessary for this design assumption. Interior drainage analysis is described above.

(2) Fourteenmile Closure Structure. Additional refined analysis was not deemed necessary for this design assumption. Interior drainage analysis is described above.

11.2 Hydrology.

Additional refined analysis was not deemed necessary for the recommended plan. It is recommended that refined hydrologic and hydraulic analysis be considered in the determination of the final levee design heights during PED. At the time of this study, the State of California in partnership with USACE was conducting a comprehensive study of hydrology. However, this data were not available at the time of this study report. In addition, more detailed evaluations of stage-frequencies in the Sacramento and San Joaquin Delta may be available at the time of PED and these studies should be utilized if practical.

11.3 Hydraulic Models and Results

Additional refined analysis was not deemed necessary for the recommended plan. It is recommended that refined hydraulic analysis be considered in the determination of the final levee design heights during PED. At the time of this study, the State of California was developing extensive hydraulic models in support of the Central Valley Flood Protection Plan. However, this data were not available at the time of this study report.

11.4 Wind-wave Analysis

Additional refined analysis was not deemed necessary for the recommended plan. More refined wind wave analysis is recommended during PED to estimate erosion protection features along the Delta Front levee reaches from wind-induced waves or boat wake.

11.5 Sedimentation and Channel Stability

Additional refined analysis was not deemed necessary for the recommended plan. This assessment was based on an evaluation of the existing conditions which indicated the existing project reaches were relatively stable. More detailed analysis to design localized erosion protection should be considered during PED.

11.6 Performance and Flood Risk

Additional refined analysis was not deemed necessary for the recommended plan. Additional performance analysis will be required during PED to refine the levee profile to meet the performance described in this report.

11.7 Potential Adverse Effects.

Additional refined analysis was not deemed necessary for the recommended plan.

11.8 Climate Change

The sensitivity of the recommended plan to different rates of sea level change (rates described in ER 1110-2-8162) were evaluated over a design life of 100 years. This was accomplished by comparing changes in the 1% (1/100) ACE stage to minimum freeboard requirements that would be necessary for accreditation in the National Flood Insurance Program administered by the Federal Emergency Management Agency. To meet accreditation the NFIP requires three feet of freeboard for the 1% (1/100) ACE flood. This criteria was selected because locals are typically interested if they will be required to purchase flood insurance in floodplain areas defended by levees. The comparison is provided on Plate 108 and is based on the D3 index point because it was found to be the most sensitive to Sea Level Change. Plate 108 shows the project would not exceed the NFIP freeboard requirements over the 100-year design life for the Low Sea Level Rise rate. However the 1% (1/100) ACE event stage would exceed the requirements in year 2057 for the High rate, 2073 for the Curve II rate, and 2110 for the Intermediate rate.

Additional analysis and a qualitative description of the performance of the alternative relative to inland climate change are provided in the hydrology and economic appendices. It is recommended that impacts to project performance due to sea level change and inland climate change be evaluated in PED. Since this is an NED plan, refinements to the design would be to insure it meets the performance values described in this report while maximizing the net benefits.

11.9 California State Urban Levee Design Criteria

Additional analysis was conducted to evaluate the performance of the project relative to this local criterion. It was estimated that levee reaches in the recommended plan would meet the ULDC criteria if additional improvements to address outflanking of the existing RD17 tieback levee were made in the future. Increases in sea level to the year 2070 were considered in the evaluation. However, increases in flood flow frequency related to climate change were not considered. The State of California is currently conducting climate change studies with respect to flood flow frequency and these studies could impact this assessment. Project performance values provided in this report are based on the existing configuration of the RD17 tieback levee. Project performance values are provided for both 2010 and 2070 sea level conditions. Inland climate change was not included because of the high degree of uncertainty.

12.0 SUMMARY

This report describes hydraulic, sedimentation, and operations and maintenance analyses performed for the final alternatives and recommended plan of the Lower San Joaquin Interim Feasibility Study. Analyses were performed for without-project and six project alternative conditions.

The study is focused on Lower San Joaquin Interim Feasibility Study area. Composite floodplain delineations are provided for 50% (1/2) ACE, 10% (1/10) ACE, 4% (1/25) ACE, 2% (1/50) ACE, 1% (1/100) ACE, 0.5% (1/200) ACE, and 0.2% (1/500) ACE events for the existing and alternative conditions.

12.0 REFERENCES

1. Chow, 1959, Open Channel Hydraulics, McGraw Hill, 1959.
2. CNRFC, 2013, Weather Forecast Office (WFO) Hydrologic Products, California Nevada River Forecast Center, http://www.cnrfc.noaa.gov/wfo_hydro.php, 27 Feb, 2013
3. DAS, 2011. Potential Increase in Flood in California's Sierra Nevada under Future Climate Projections, June 2011.
4. DWR, 2010. State Plan of Flood Control Descriptive Document, California Department of Water Resources, November 2010.
5. DWR,2012. Urban Levee Design Criteria, California Department of Water Resources, May 2012.
6. DWR, 2014. Erosion Screening Process Report, Reclamation District 404 Study Area, Prepared by URS for California Department of Water Resources, April 2014.
7. DWR, 2014. Erosion Screening Process Report, Shima Tract Study Area, Prepared by URS for California Department of Water Resources, April 2014.
8. DWR, 2014. Erosion Screening Process Report, Stockton Diverting Canal/Mormon Slough Study Area, Prepared by URS for California Department of Water Resources, April 2014.
9. DWR, 2014. Erosion Screening Process Report, Reclamation District 17 Study Area, Prepared by URS for California Department of Water Resources, April 2014.
10. DWR, 2014. Erosion Screening Process Report, Smith Canal Study Area, Prepared by URS for California Department of Water Resources, April 2014.
11. FEMA, 2011, Floodplain Management Requirements, A Study Guide and Desk Reference for Local Officials http://www.fema.gov/plan/prevent/floodplain/fm_sg.shtm.
12. FEMA, 2012, Levee Certification vs. Accreditation, Federal Emergancy Management Agency, October 2012.
13. FEMA, 2009, Flood Insurance Study, San Joaquin County California and incorporated Areas, Study Number 06077CV001A, Federal Emergency Management Agency, 16 October 2009.
14. FLO-2D Software Inc. Flo-2D Flood Routing Model, Version 2006.01, 2004
15. HEC, 2008. Hydrologic Engineering Center, HEC-RAS River Analysis Program Version 4.0.0, March, 2008.
16. HEC, 2010. Hydrologic Engineering Center, HEC-FDA Flood Damage Assessment Program Version 1.2.5a, November, 2010.
17. PBI, 2013A. Technical Memorandum, San Joaquin River Main Stem HEC-RAS Model Setup, September 2013.
18. PBI, 2013B. Technical Memorandum, Review and Update of the CVFED Calaveras River HEC-RAS Model, September 2013.

19. PBI, 2013C. Technical Memorandum, Mormon Channel Bypass Cost Estimate, September 2013.

20. SJAFCA, 2012. Lower San Joaquin River Feasibility Study Hydrology Appendix, July 2012.

21. SJAFCA, 2013. Lower San Joaquin and Delta South Regional Flood Management Plan, November 2013.

22. USACE, 1952. Operation and Maintenance Manual for Duck Creek Diversion, A Unit of Farmington Reservoir Project, December 1952.

23. USACE, 1955. San Joaquin River Levees General Design, December 1955.

24. USACE, 1974. Civil Works Project Maps, River and Harbor, Flood Control, and California Debris Commission, U.S. Army Engineer District, Sacramento, Corps of Engineers. 1974.

25. USACE, 1975. Southwest Stream Group, December 1975.

26. USACE, 1989. Expected Annual Flood Damage Computation, Users Manual, CPD-30 US Army Corps of Engineers Hydrologic Engineering Center, March 1989.

27. USACE 1993. San Joaquin River Mainstem, California, January 1993

28. USACE, 1996. Engineering and Design, Risk-Based Analysis for Flood Damage Reduction Studies, EM110-2-1619, 1 August 1996

29. USACE, 1999. Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies, May 1999.

30. USACE, 2001. Mormon Channel 1135 Restoration, Stockton, California, 90% Final Alternatives Report, Prepared by HDR Engineering Inc for San Joaquin Area Flood Control Agency and USACE, August 2001.

31. USACE, 2002. Sacramento San Joaquin Comprehensive Study. United States Army Corps of Engineers, December 2002.

32. USACE, 2008. Memorandum, Clarification on the Policy and Procedural Guidance for the Approval of Modifications and Alterations of Corps of Engineers Projects, Director of Civil Works, 17 November 2008.

33. USACE, 2009, Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums, ER 1110-2-8160, 1 March 2009.

34. USACE, 2010a, Operation and Maintenance Manual for Mormon Slough Project, San Joaquin County, California, Part 1 - Levees and Channels San Joaquin River to Bellota, Department of the Army, Sacramento District, Corps of Engineers, Sacramento Ca, 6 April 2010.

35. USACE, 2010b, Operation and Maintenance Manual Duck Creeek Project, San Joaquin County, California, French Camp Road to One-Half Mile Upstream from Escalon-Bellota Road, Department of the Army, Sacramento District, Corps of Engineers, Sacramento Ca, 26 March 2010.

36. USACE, 2011a, Memorandum, Corps of Engineer Civil Works Cost Definitions and Applicability, Director of Civil Works, 25 August 2011

37. USACE, 2011b. Supplement to Standard Operation and Maintenance Manual, Lower San Joaquin River and Tributaries Project, California, Unit No. 1 Right Bank Levee of San Joaquin River and French Camp Slough within Reclamation District No. 404, Department of the Army, Sacramento District, Corps of Engineers, Sacramento Ca, March 1963, Revised 24 May 2011.

38. USACE, 2011c. Supplement to Standard Operation and Maintenance Manual, Lower San Joaquin River and Tributaries Project, California, Unit No. 2 Right Bank Levee of San Joaquin River and Left Bank of French Camp Slough within Reclamation District No. 17, Department of the Army, Sacramento District, Corps of Engineers, Sacramento Ca, May 1963, Revised 24 May 2011.

39. USACE, 2011d. Supplement to Standard Operation and Maintenance Manual, Lower San Joaquin River and Tributaries Project, California, Unit No. 7 West Levee of San Joaquin River and North Levee of Old River Reclamation Districts No.524 and 544, Department of the Army, Sacramento District, Corps of Engineers, Sacramento Ca, April 1964, Revised 24 May 2011.

40. USACE, 2012. Operations and Maintenance Manual, Bear Creek Project, San Joaquin County, CA. Part 1 from Disappointment Slough Upstream to U.S. Highway No. 99. June 1965, Revised 31 December 2012

41. USACE, 2012 Engineering and Design: USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation, EC 1110-2-6067, 31 August 2010

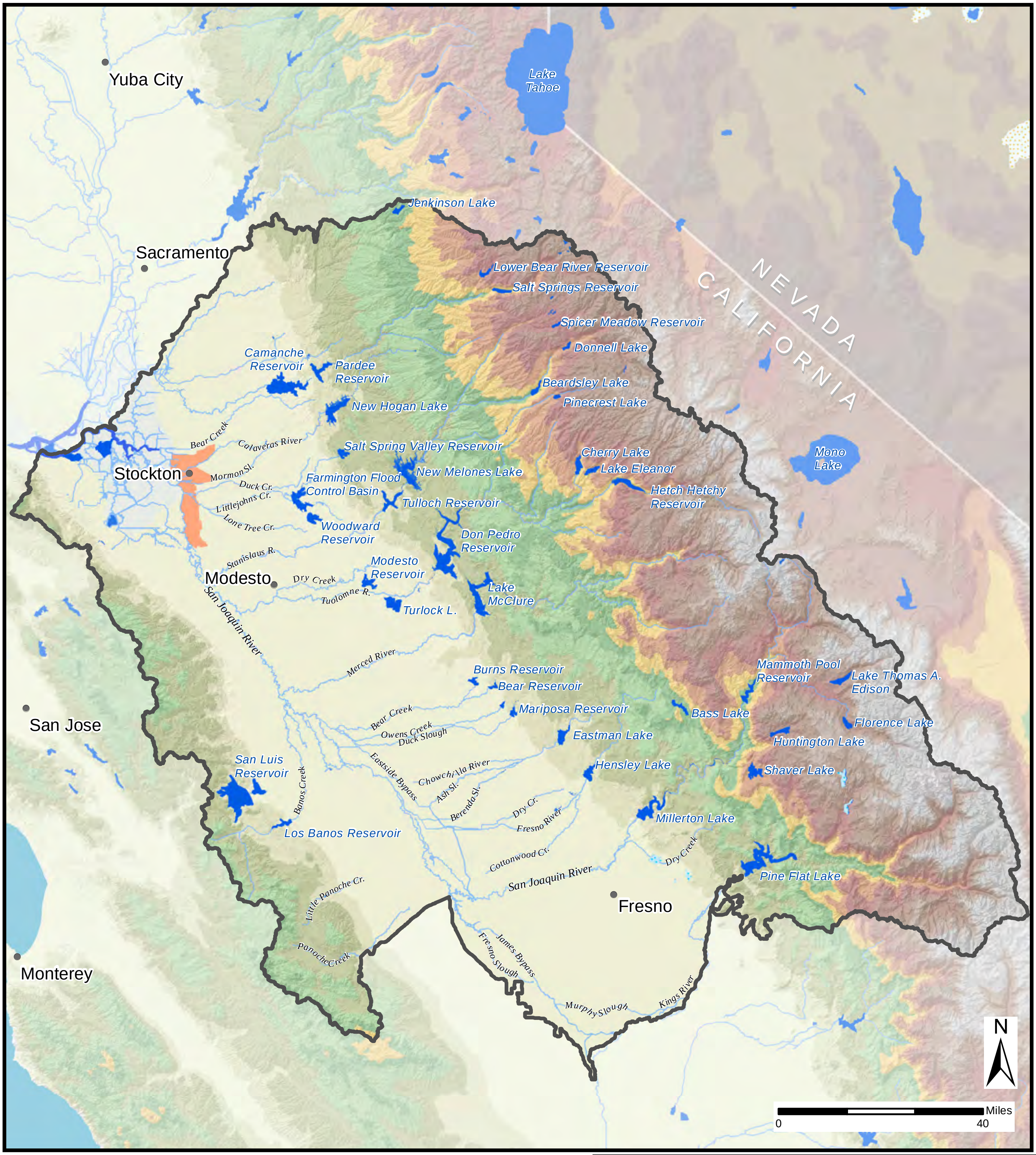
42. USACE, 2014, Engineering and Design: Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures, ETL 1110-2-583, 30 April 2014

43. USACE, 2015, Letter to the Central Valley Flood Protection Board from Operations and Readiness Branch of Sacramento District U.S. Army Corps of Engineers, 23 July, 2015.

44. U.S. Census Bureau, 2010. 2010 Census Block-Derived Housing and Population Density, Tiger/Line Shapefile: census2010den_11_1.shp, U.S. Department of Commerce, U.S. Census Bureau, Geography Division, 11 April 2011, Retrieved from http://www.census.gov/cgi-bin/geo/shapefiles2010/main

45. USDOT, 2012. United States Department of Transportation Federal Highway Administration, Hydraulic Design of Highway Culverts, Third Edition, Hydraulic Design Series Number 5, Publication No. FHWA-HIF-12-026, April 2012.

46. Jonkman, 2008. Methods for the estimation of loss of life due to floods: a literature review and a proposal for a new method.



Legend

- Cities
- LSJ Watershed Boundary
- Rivers or Streams
- Lake or Reservoir
- Study Extent

NOTE: Analysis Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

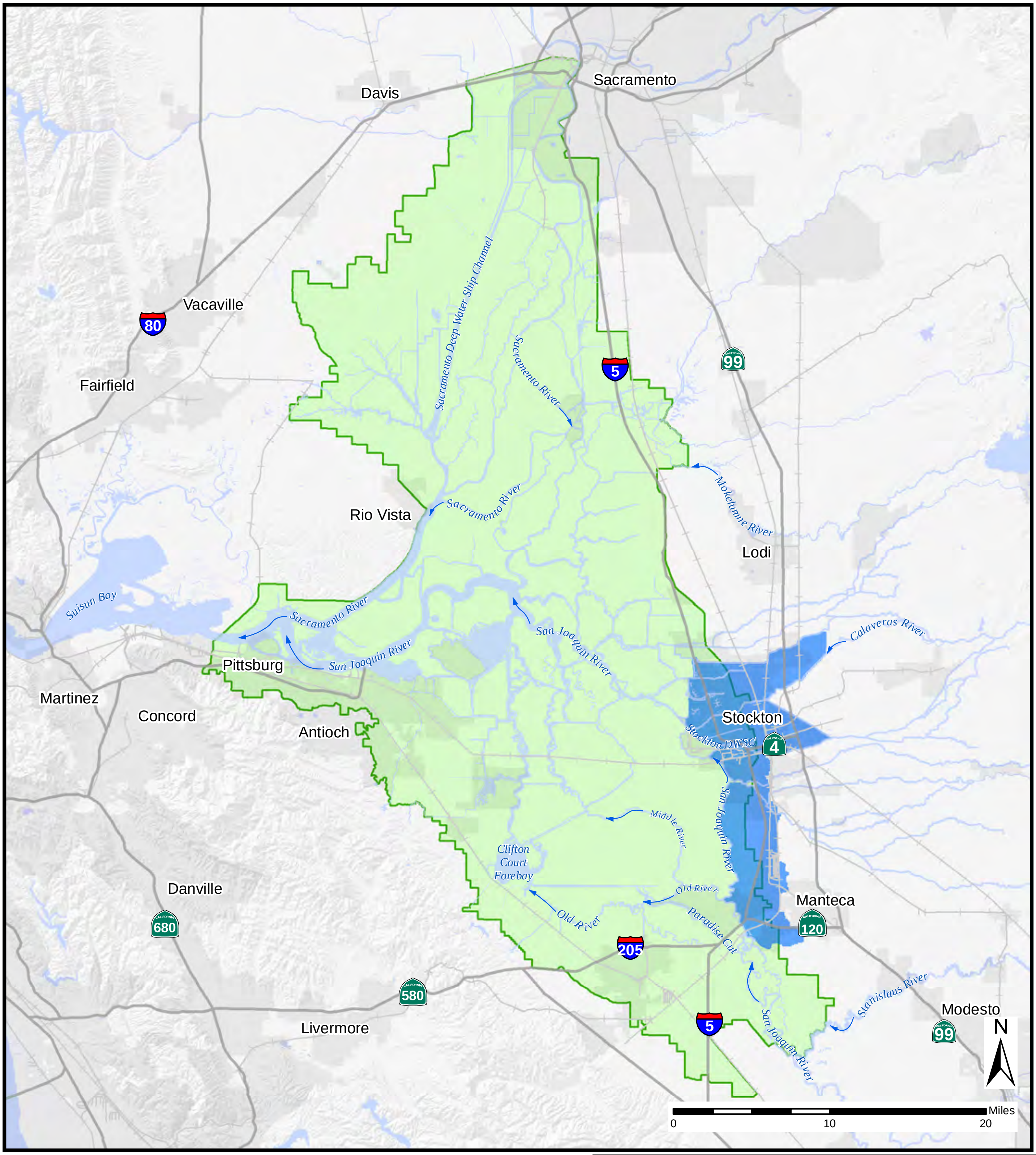
SAN JOAQUIN WATERSHED BOUNDARY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 1



Legend

- Highway
- Railroads
- Delta Legal Boundary
- Study Extent

Note:
Downstream of Rio Vista, the Sacramento River is maintained as part of the Sacramento Deep Water Ship Channel
Downstream of Stockton, the San Joaquin River is maintained as part of the Stockton Deep Water Ship Channel.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

SACRAMENTO-SAN JOAQUIN DELTA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

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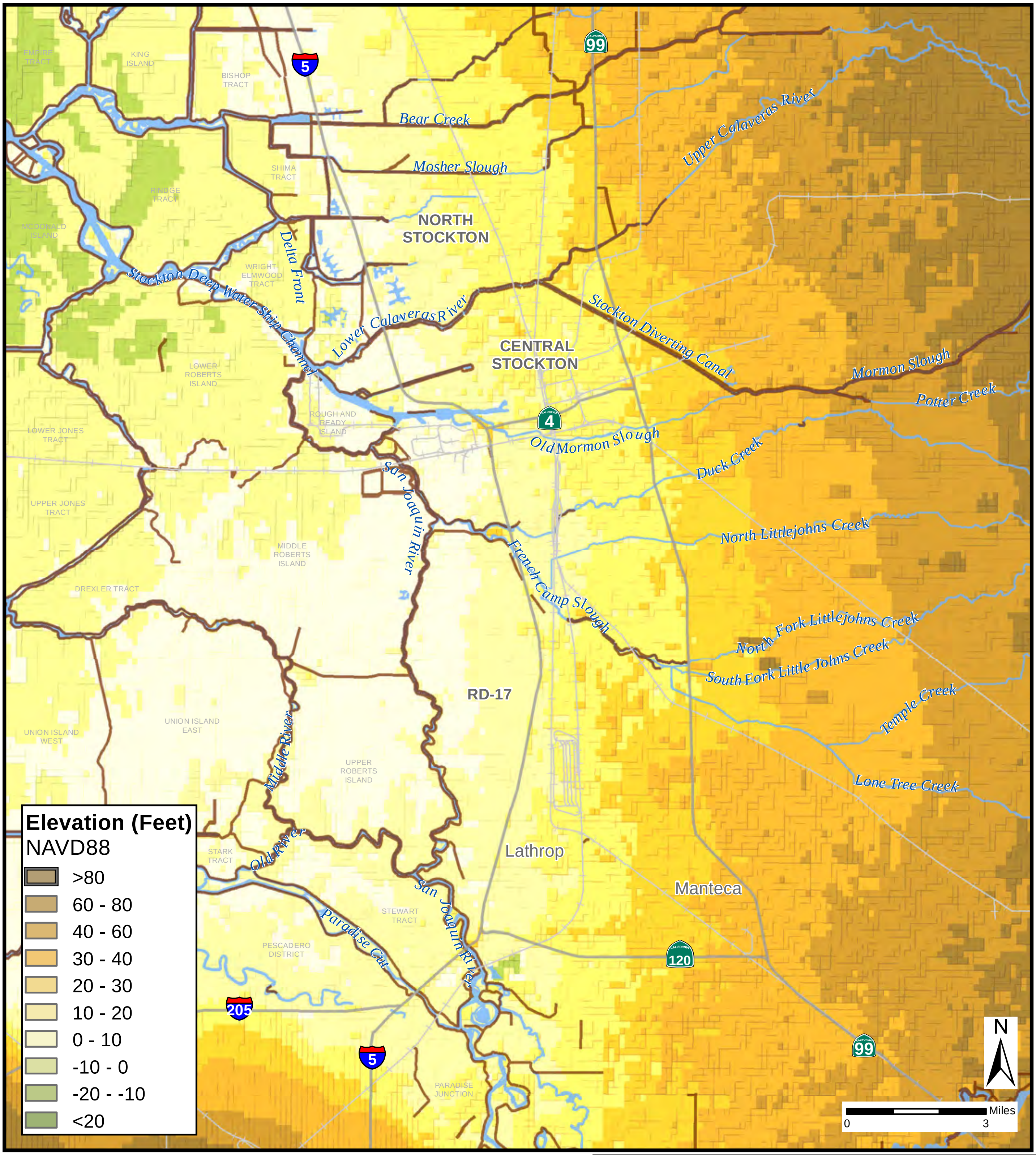


Legend

Highway

Railroads

Levees (Fed/Non-Fed)

NOTE: Elevation Based on IFSAR 2.0 (feet)

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

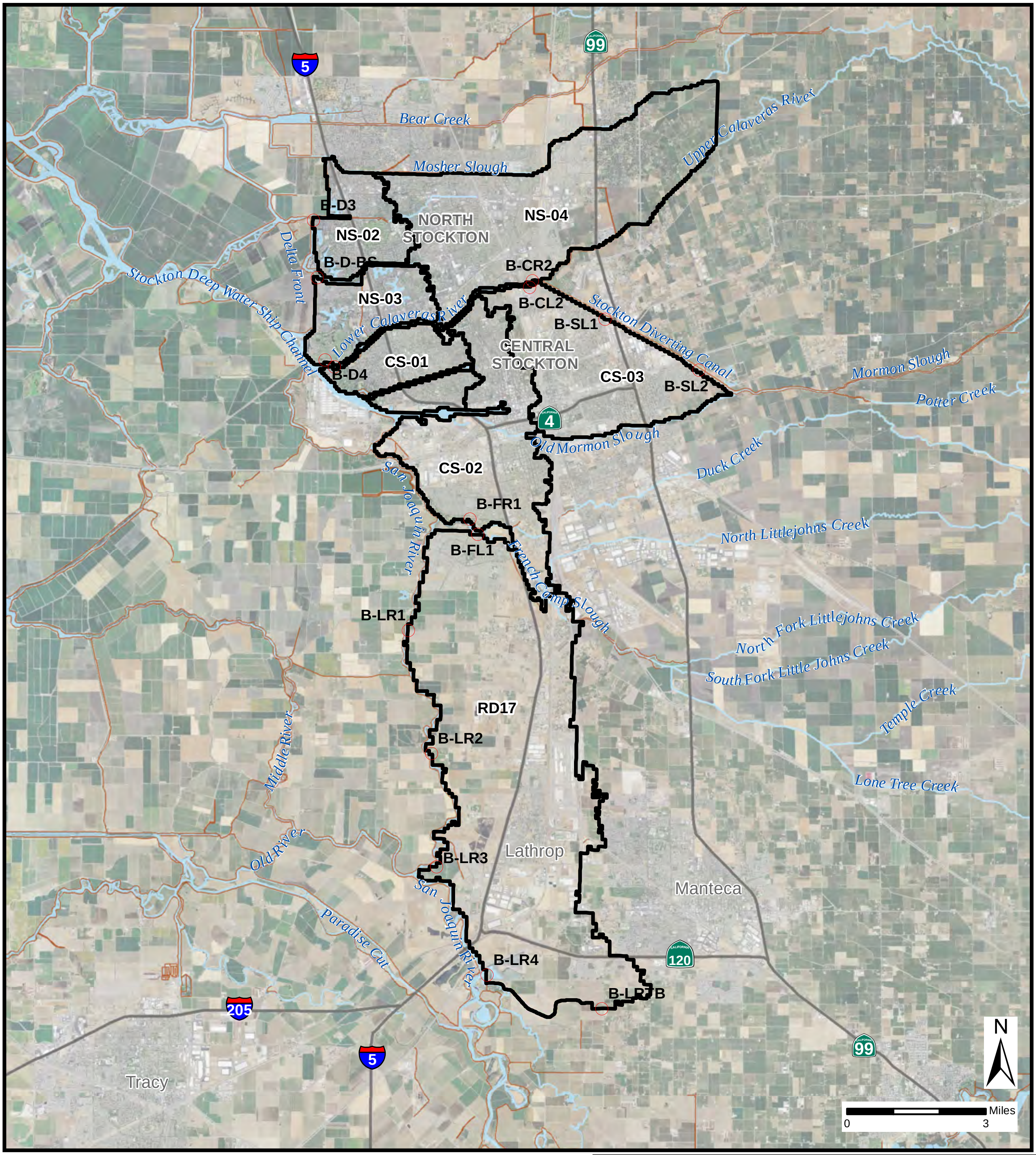
REGIONAL TOPOGRAPHY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 3



Legend

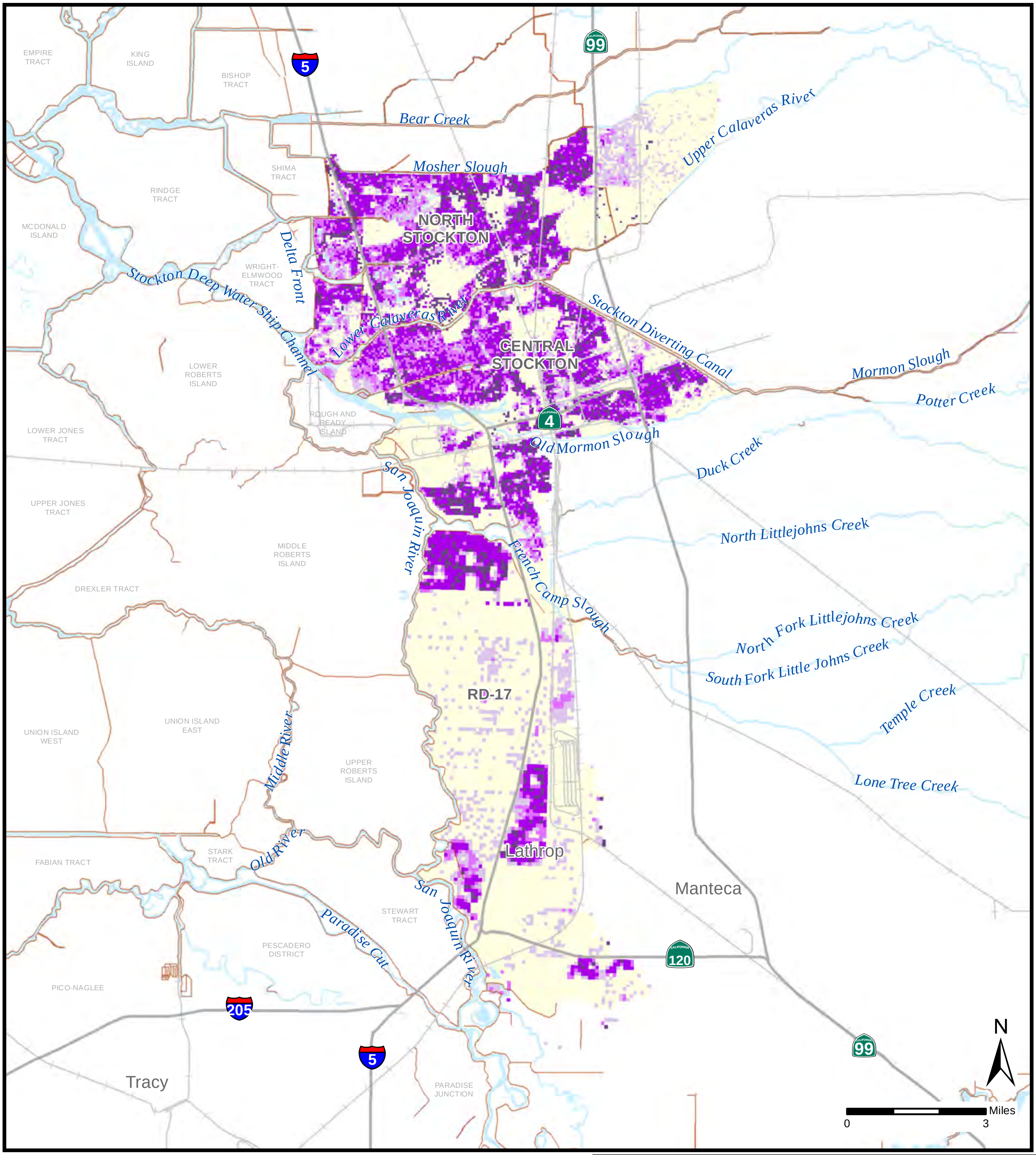
- Economic Index Point
- Highway
- Railroads
- Levees (Fed/Non-Fed)

NOTE: Economic Impact Areas Limited to Study Extent.
Aerial Imagery: NAIP 2012, 1 m.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

ECONOMIC IMPACT AREAS AND AERIAL IMAGERY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



Legend

- Highway
- Railroads
- Levees
- Study Extent

Population Per Acre

- Sparse (0 - 5)
- Low Density (5 - 10)
- Medium Density (10 - 20)
- High Density (Greater than 20)

NOTE: Population Only Shown within Study Extent

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

POPULATION STUDY AREA DENSITY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

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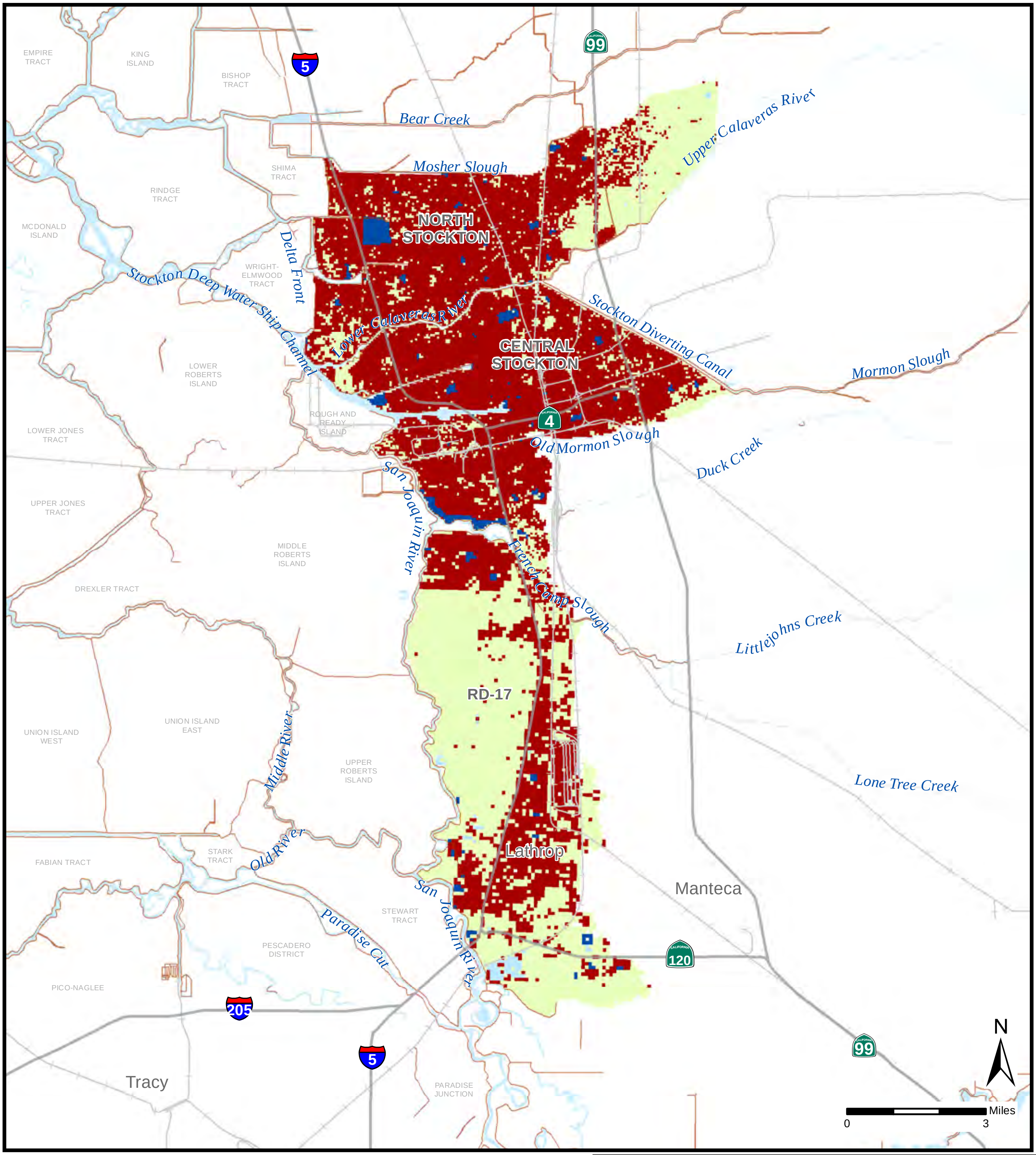

Legend

- Highway
- Railroads
- Levees (Fed/Non-Fed)
- LSJ Study Extent

Landuse

- Protected Area
- Agriculture
- Developed Land
- Open Water

NOTE: Land Use Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

EXISTING LANDUSE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

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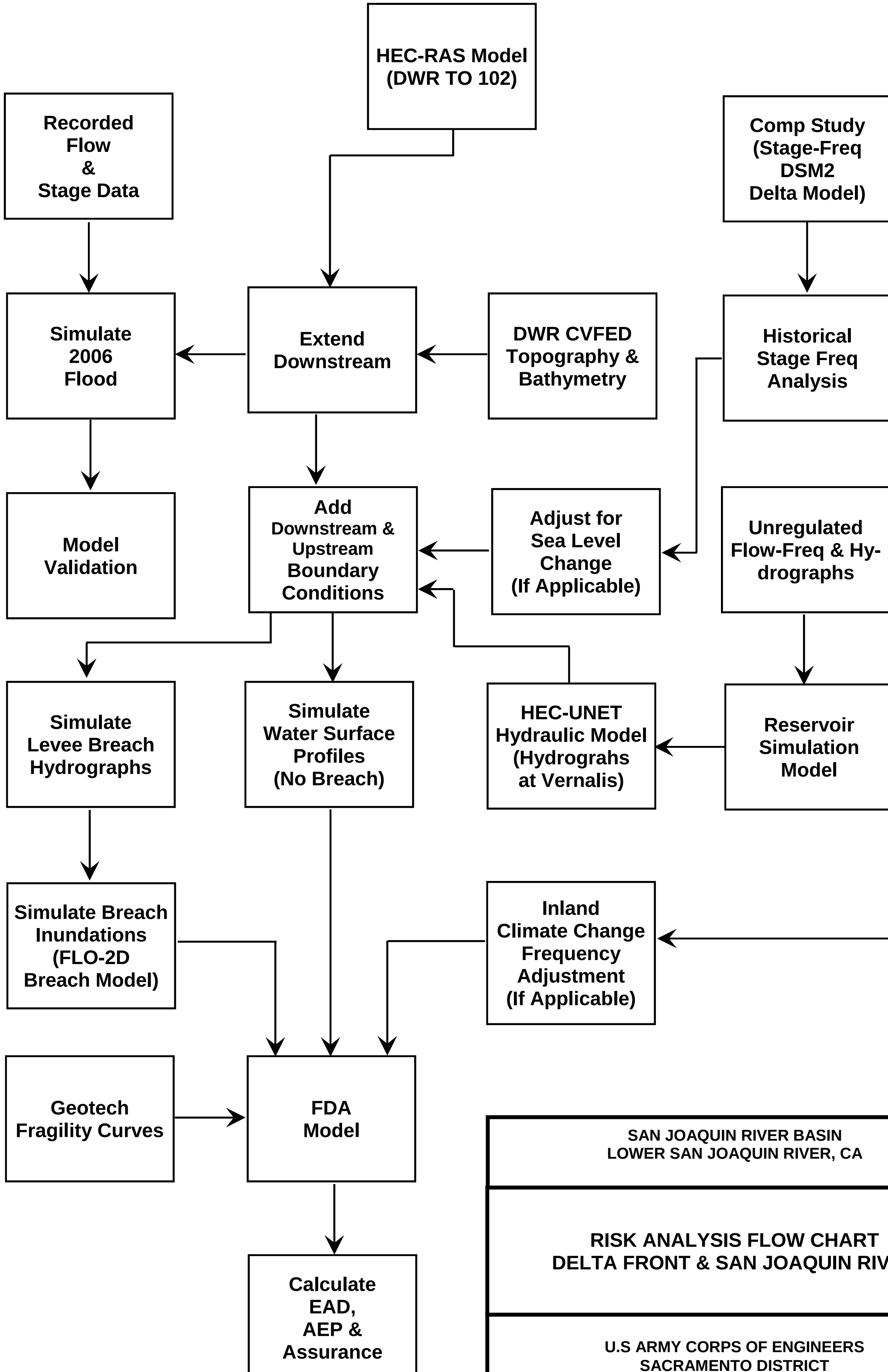
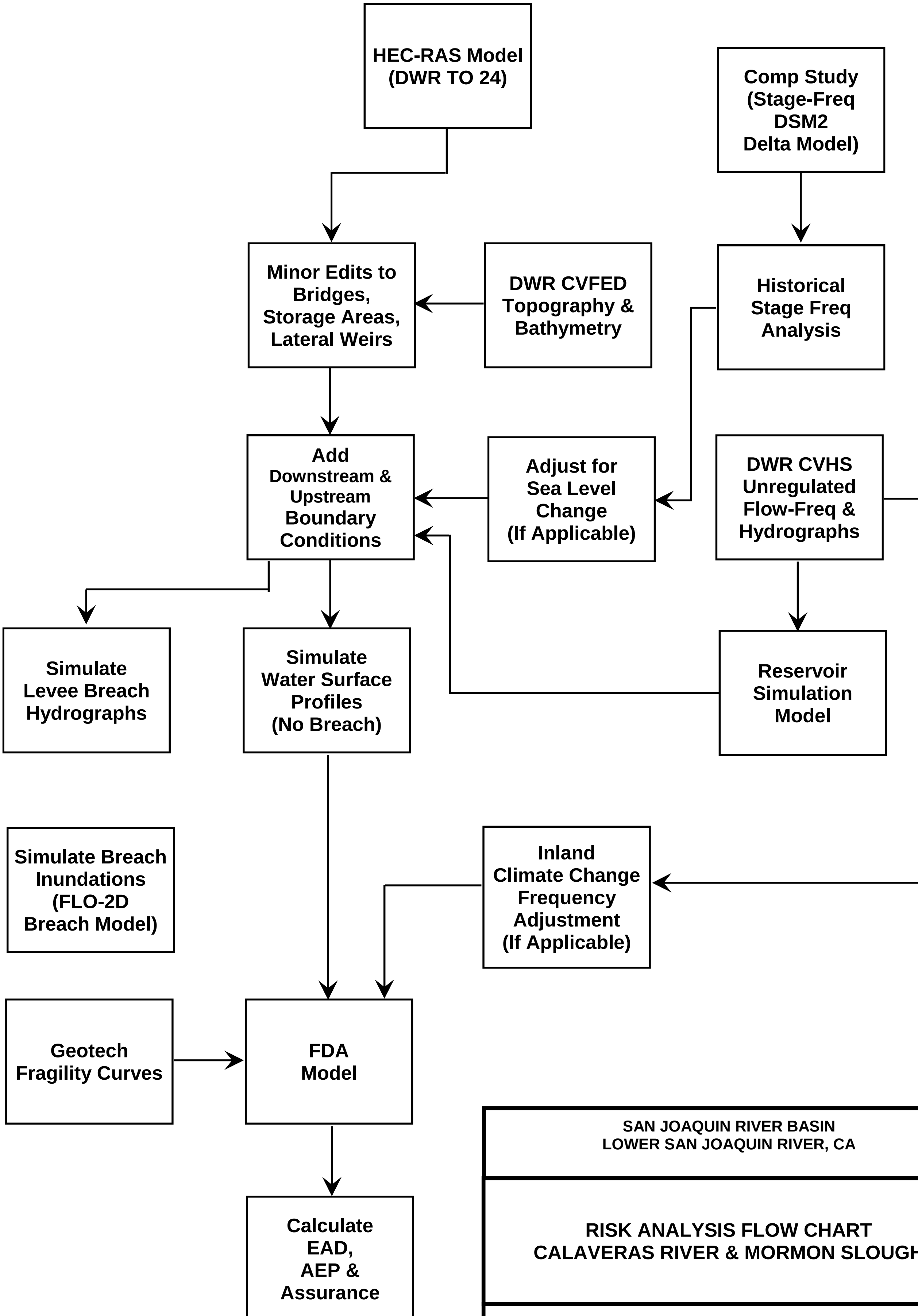


SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA

RISK ANALYSIS FLOW CHART
DELTA FRONT & SAN JOAQUIN RIVER

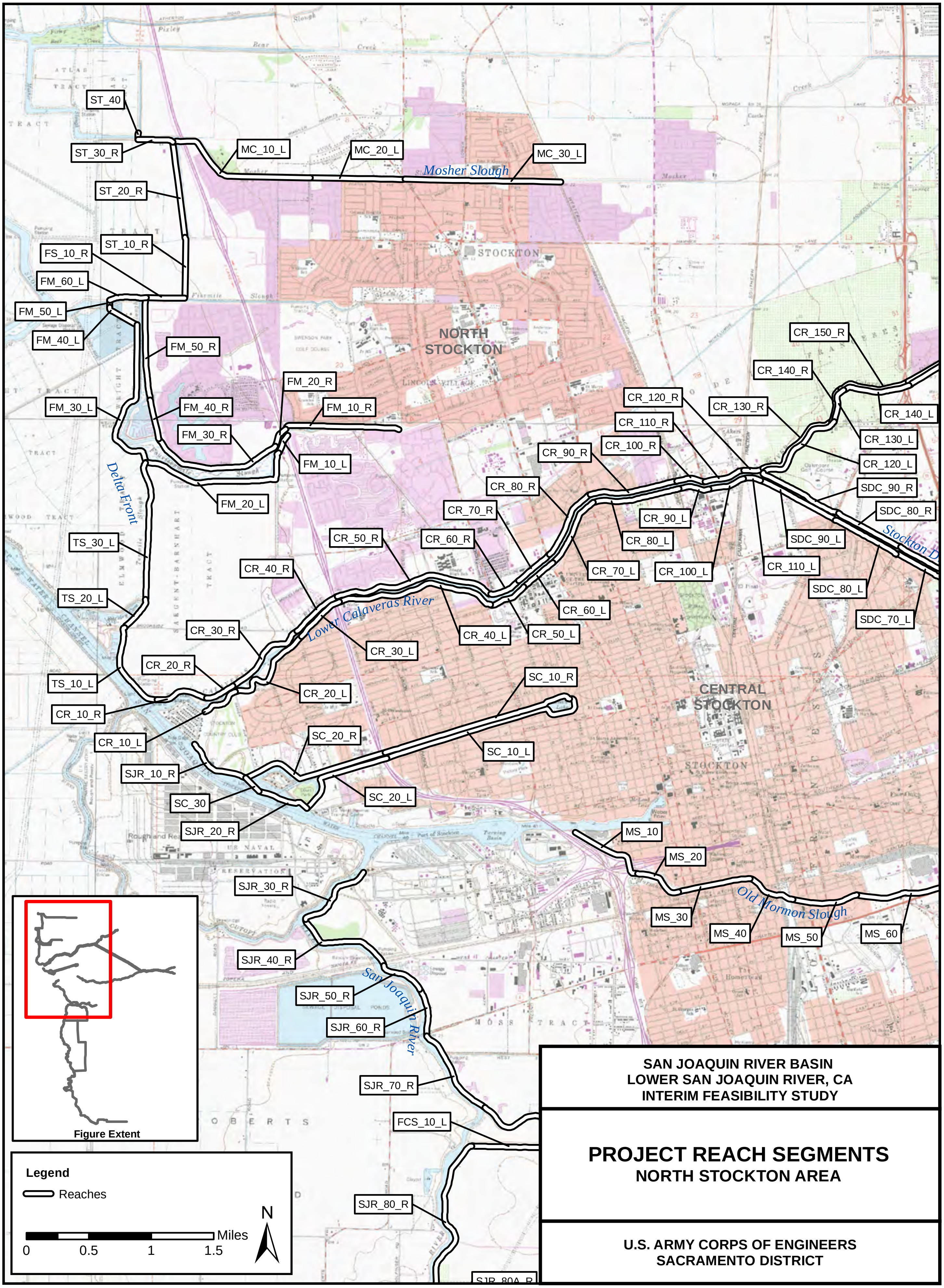
U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA

RISK ANALYSIS FLOW CHART
CALAVERAS RIVER & MORMON SLOUGH

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



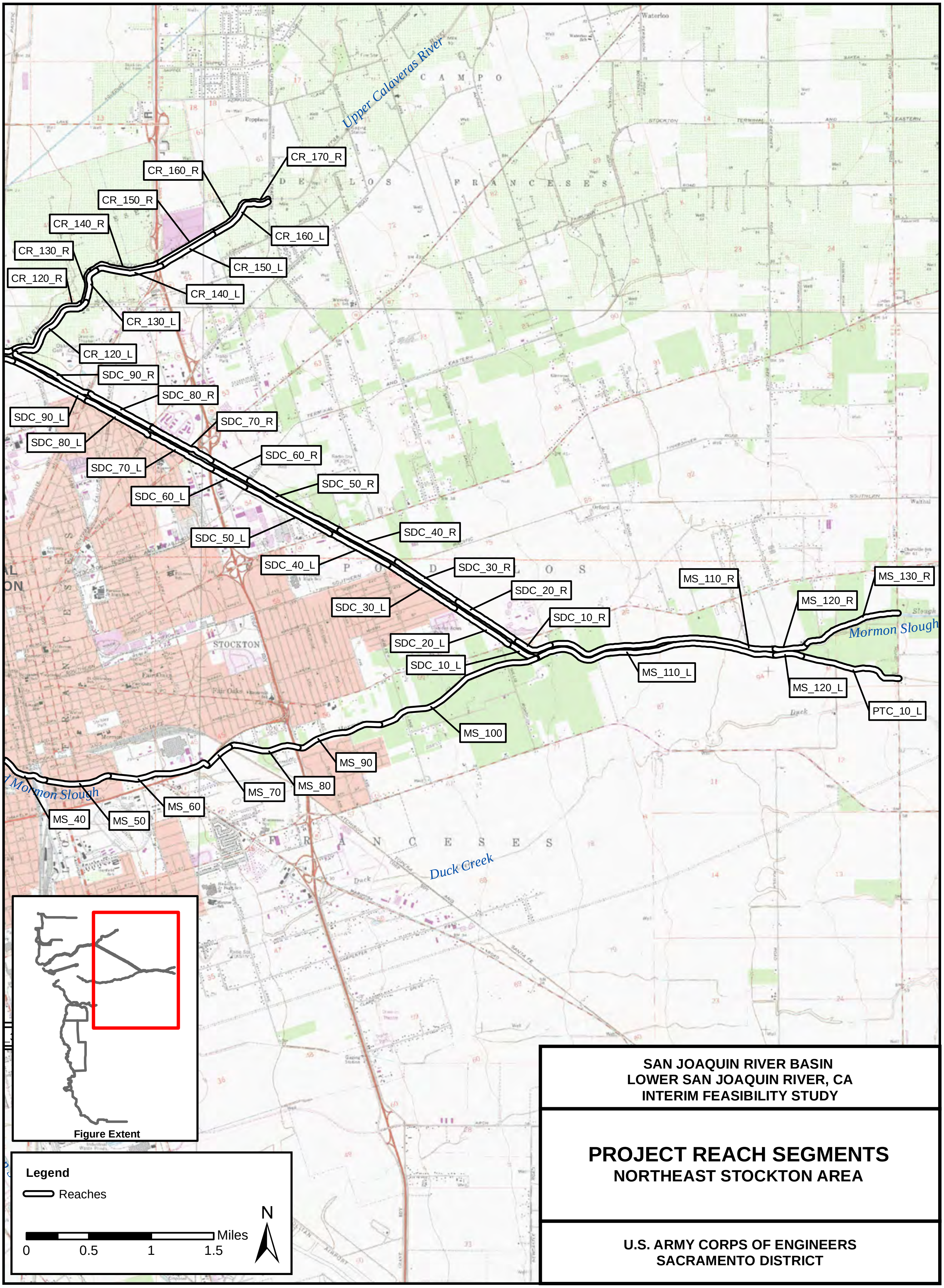
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS
NORTH STOCKTON AREA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

PLATE 9A

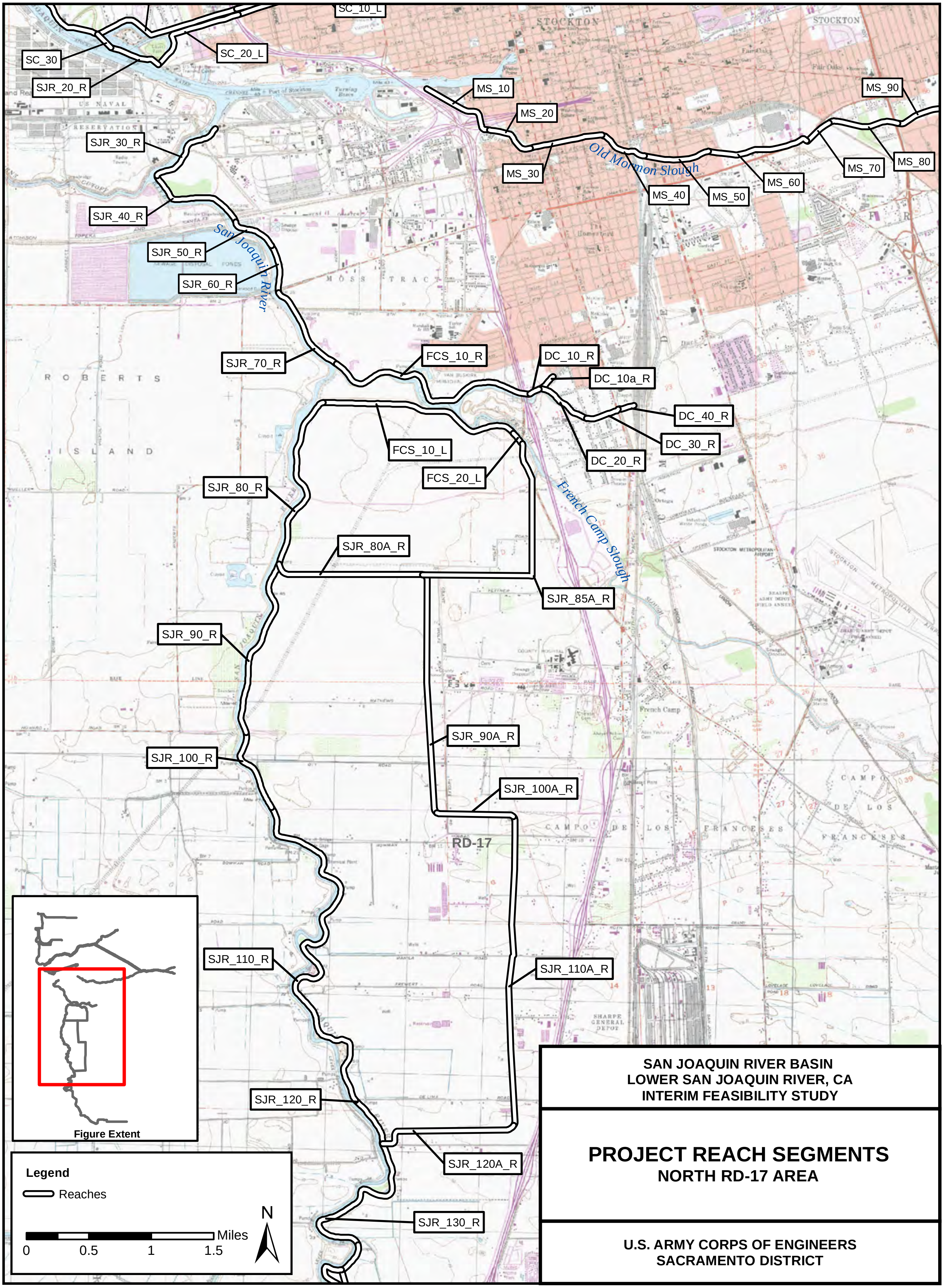


SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS
NORTHEAST STOCKTON AREA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014 PLATE 9B



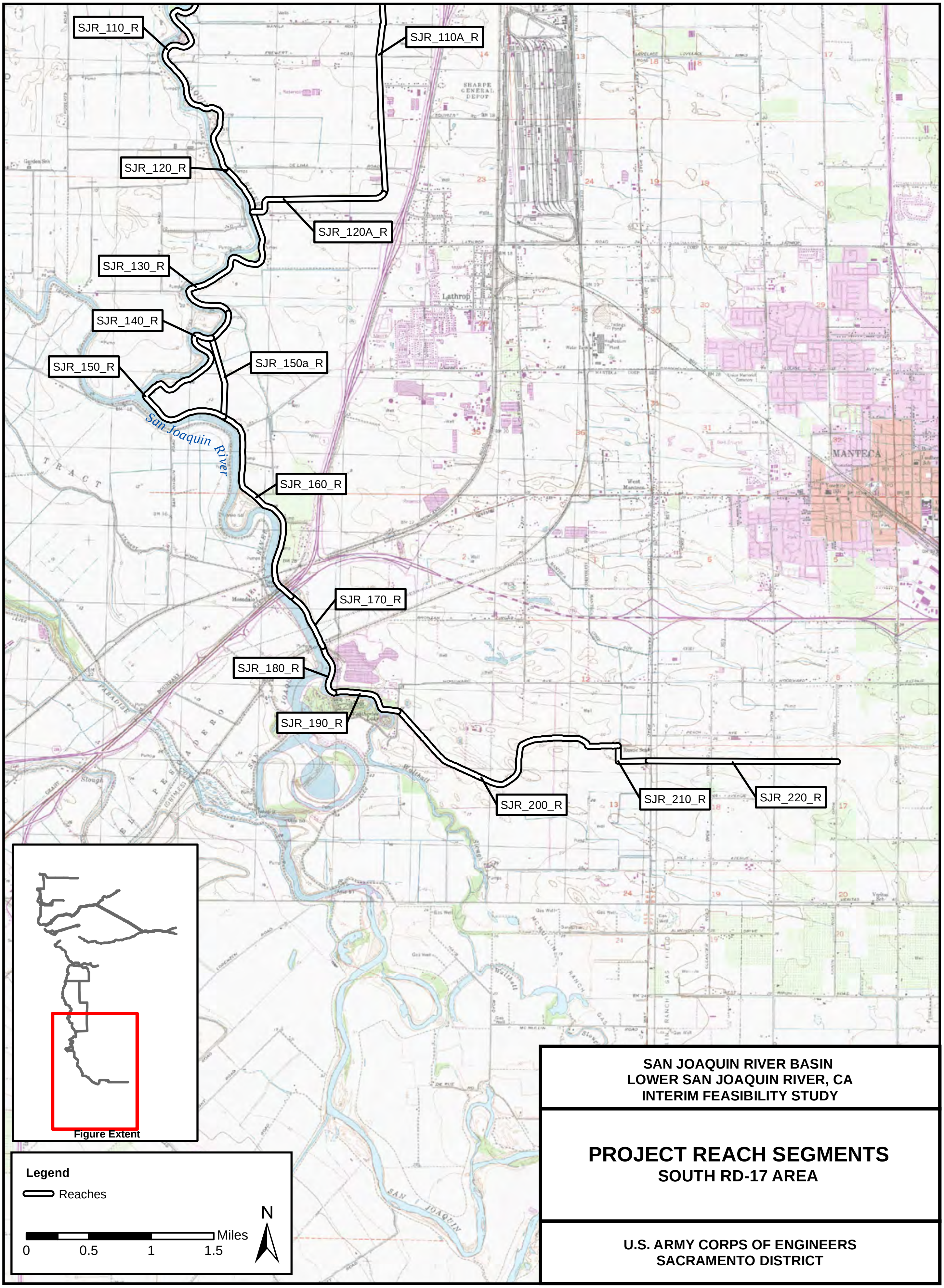
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS
NORTH RD-17 AREA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

PLATE 9C



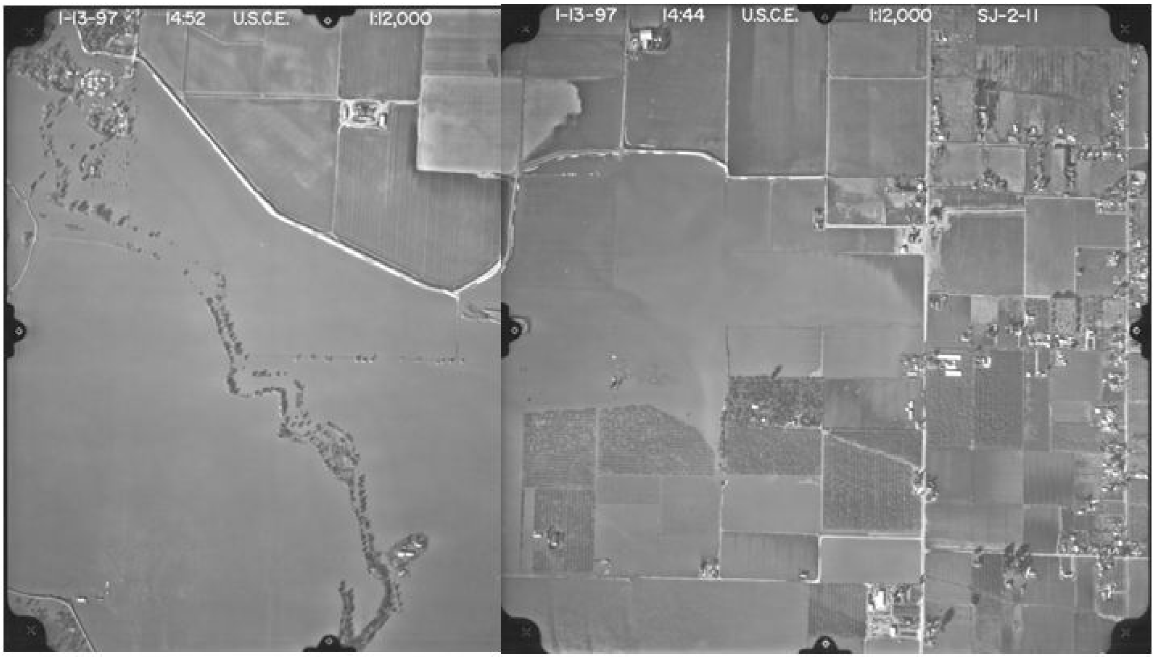
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS

SOUTH RD-17 AREA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014 PLATE 9D



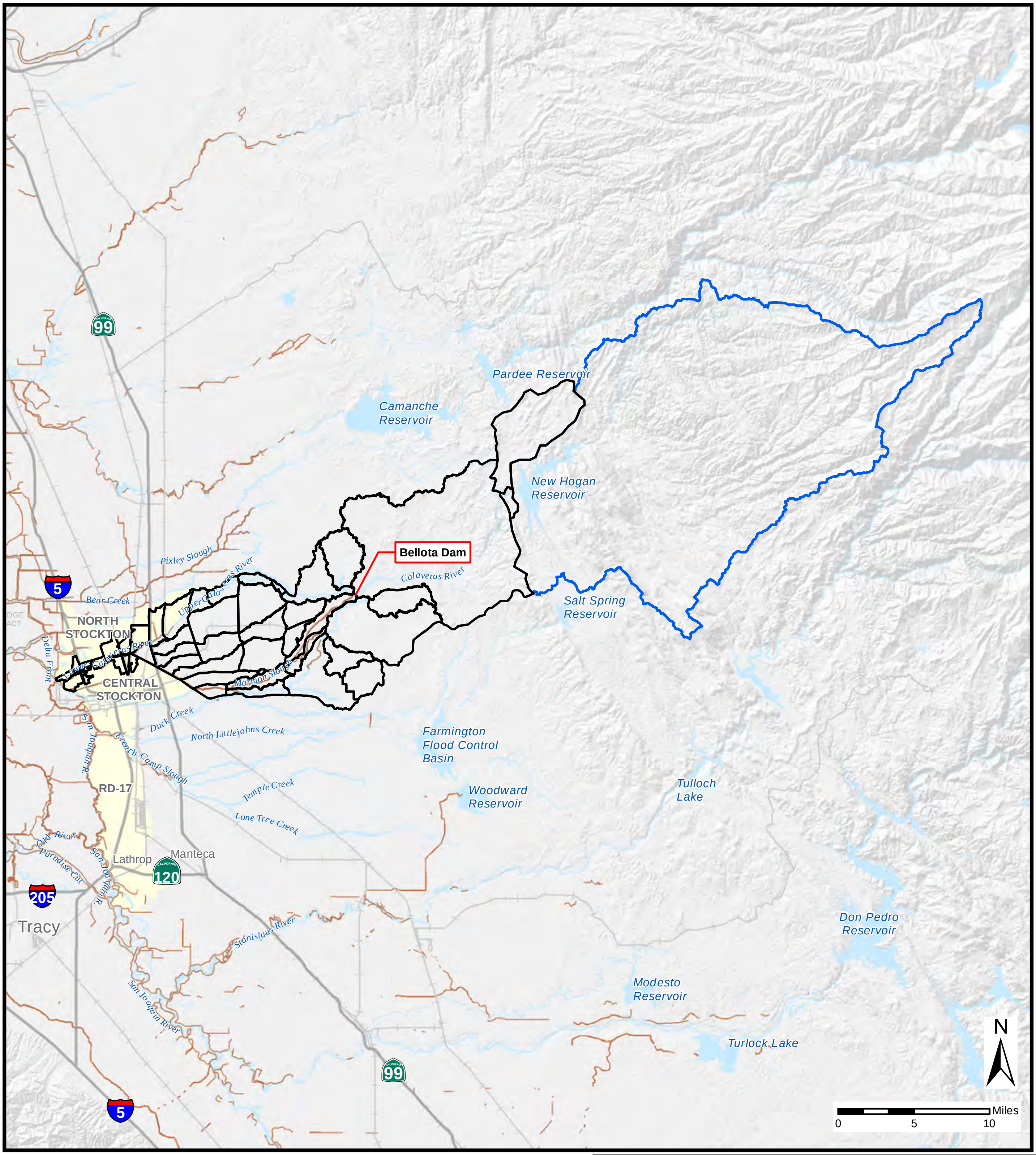
Tieback Levee at Upstream end of RD17, 13 January 1997

Floodwaters within Wetherbee Lake (Walthal Slough)
upstream of RD17 Tieback Levee Looking East

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

1997 FLOOD
WETHERBEE LAKE AND
RD17 TIEBACK LEVEE

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



Legend

- Calaveras Subbasins
- New Hogan Reservoir Wathershed Boundary
- Highway
- Railroads
- Levees (Fed/Non-Fed)
- Study Extent

NOTE: Analysis Limited to Study Extent.

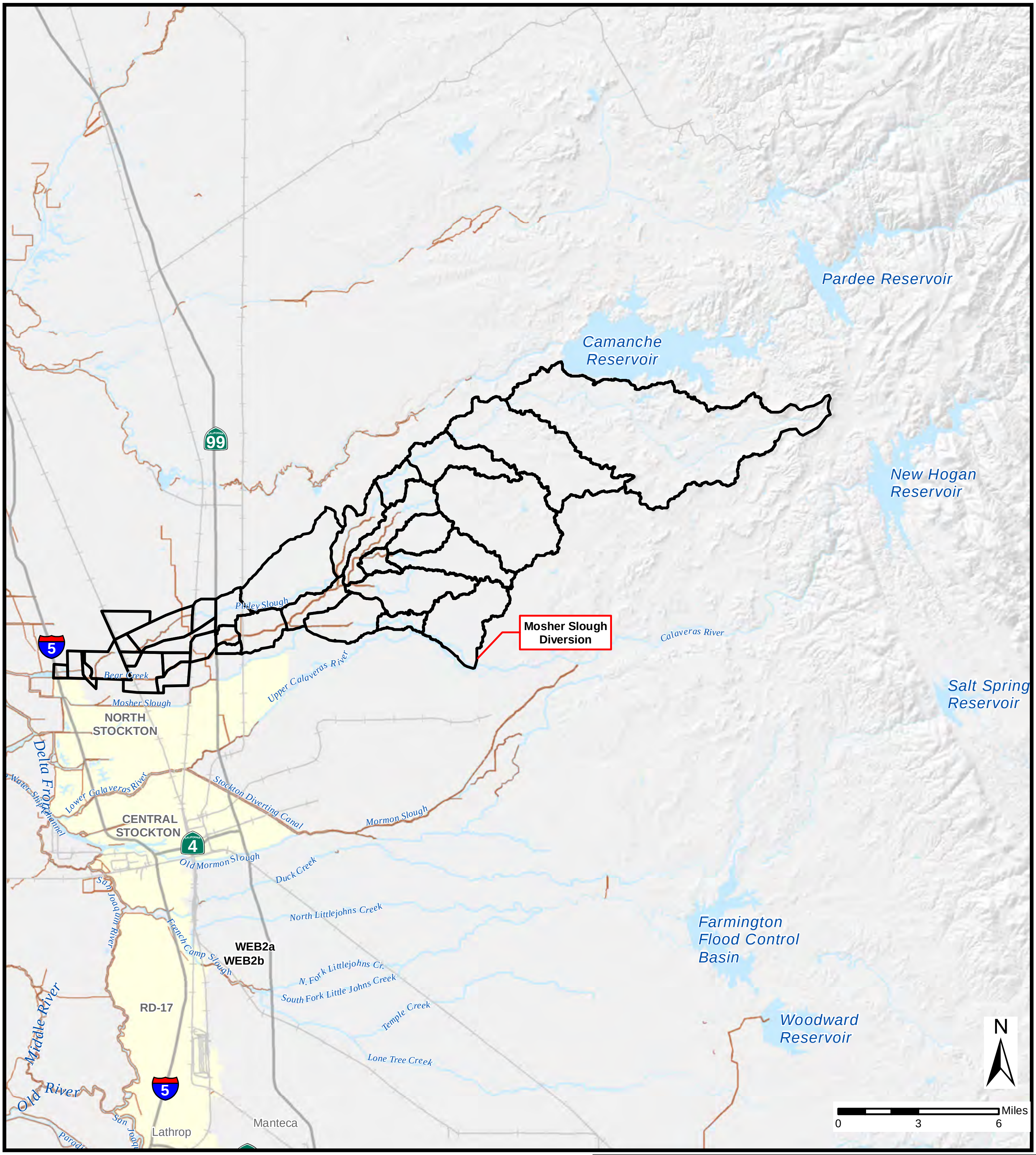
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

CALAVERAS RIVER WATERSHED BOUNDARY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014



Legend

- Highway
- Railroads
- Levees (Fed/Non-Fed)
- Bear Creek Subbasins
- Study Extent

NOTE: Analysis Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BEAR CREEK WATERSHED BOUNDARY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

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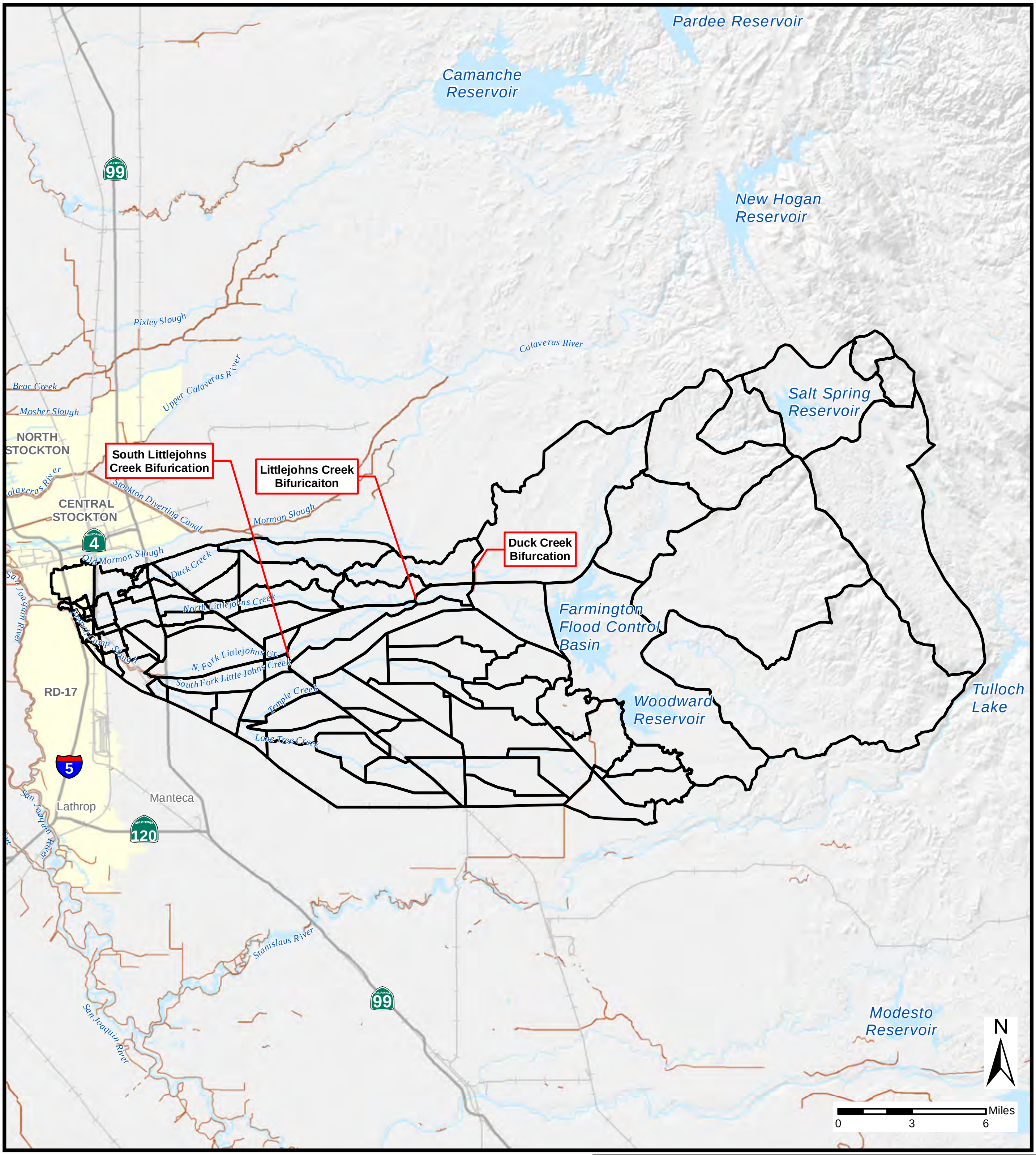
PLATE 12



Legend

- Highway
- Railroads
- Levees (Fed/Non-Fed)
- FCS Subbasins
- Study Extent

NOTE: Analysis Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

FRENCH CAMP SLOUGH WATERSHED BOUNDARY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

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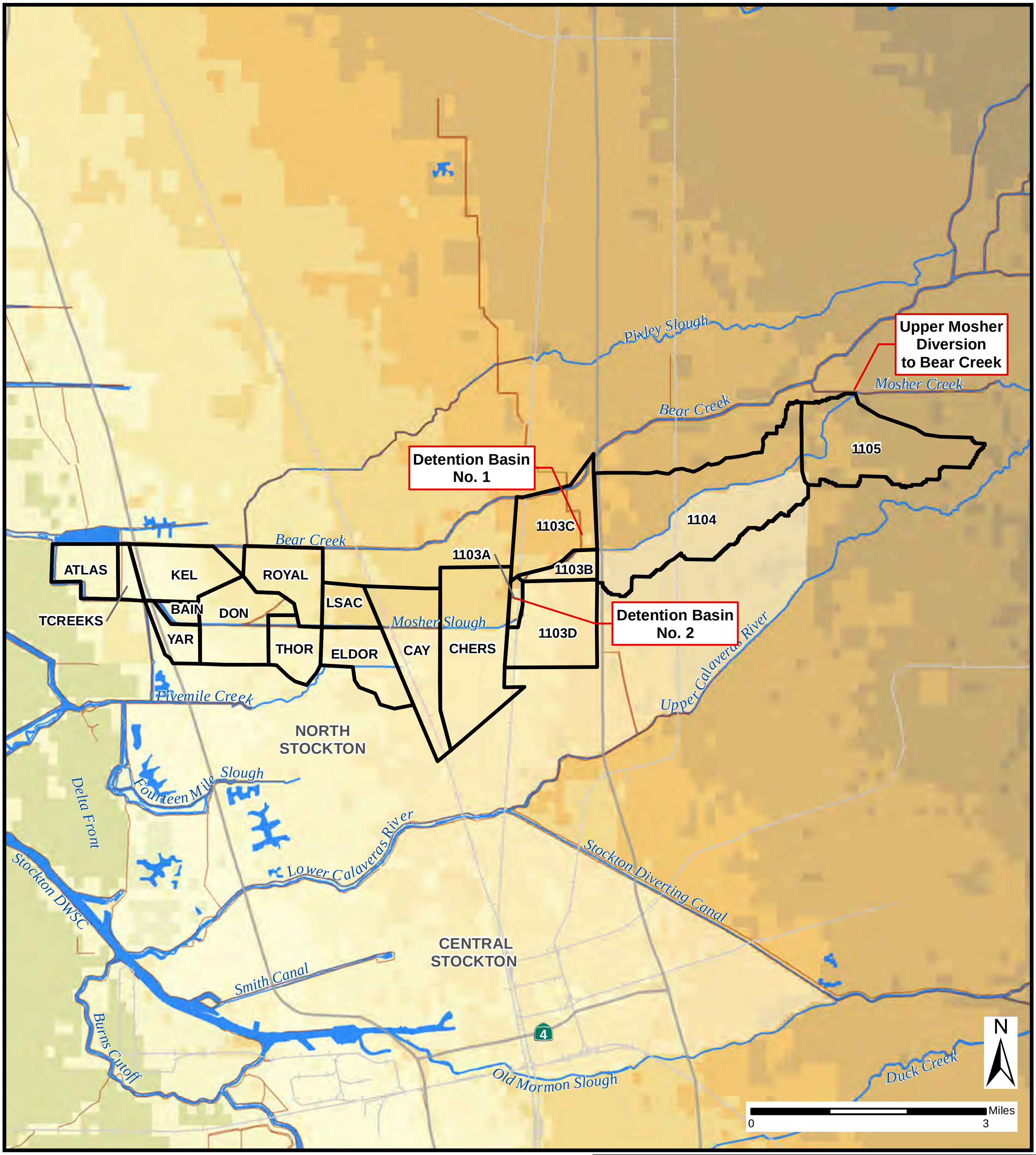
PLATE 13



Legend

- Highway
- Railroads
- Levees (Fed/Non-Fed)
- Mosher Subbasins
- Study Extent

NOTE: Elevation Based on IFSAR 2.0 (NAVD 88, feet)

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

MOSHER SLOUGH WATERSHED BOUNDARY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

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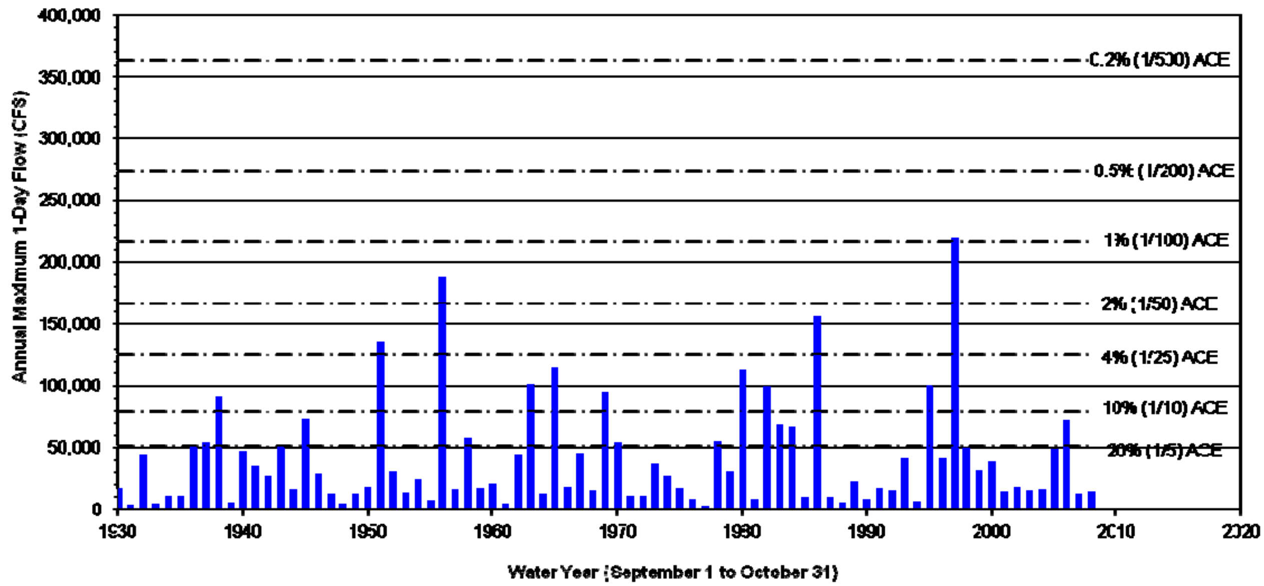
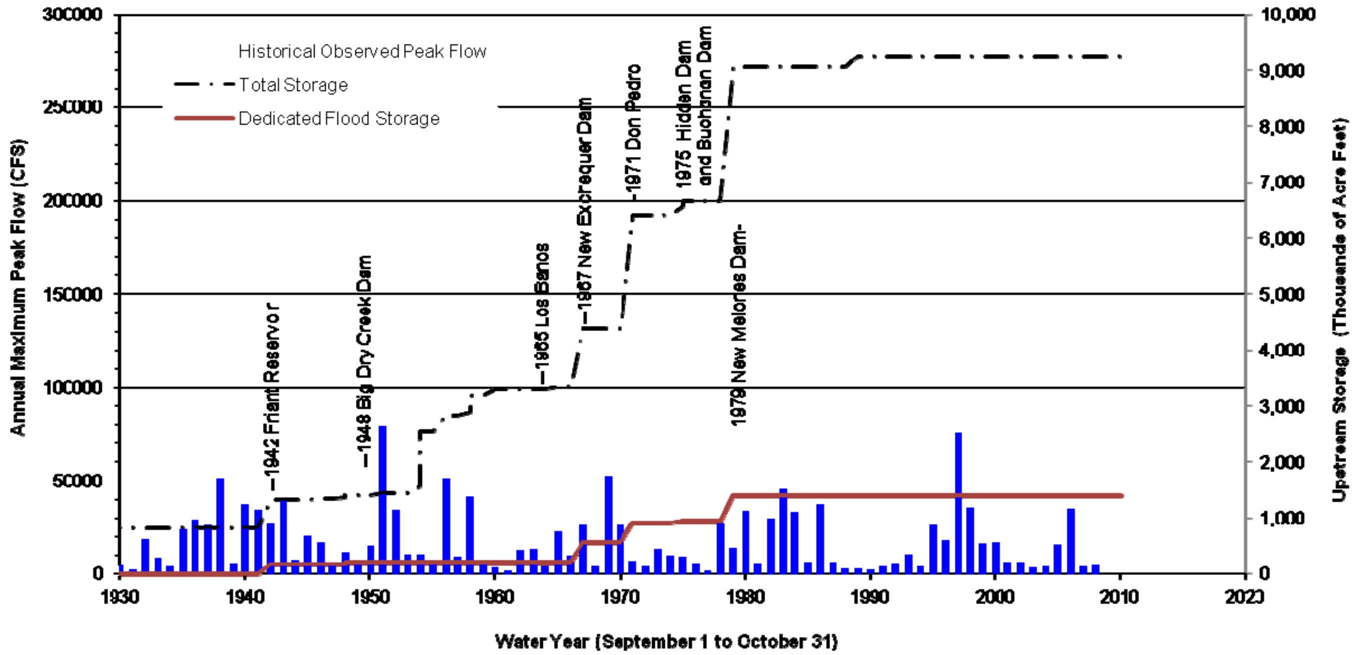
PLATE 14



Unregulated Conditions



Regulated Conditions

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASBILITY STUDY

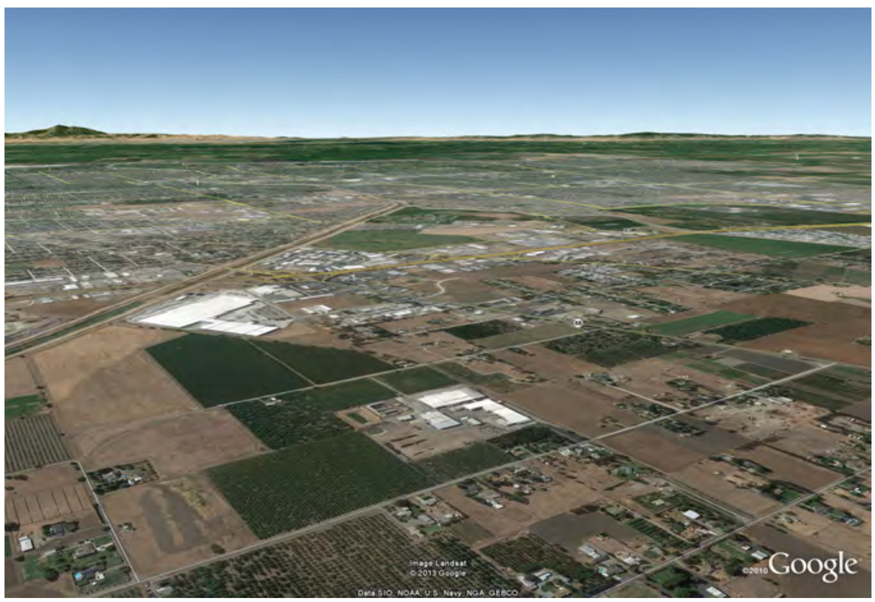
ANNUAL MAXIMUM 1-DAY FLOW
SAN JOAQUIN RIVER AT VERNALIS
UNREGULATED AND REGULATED CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by PJB

JUN 2014

1955 Flood



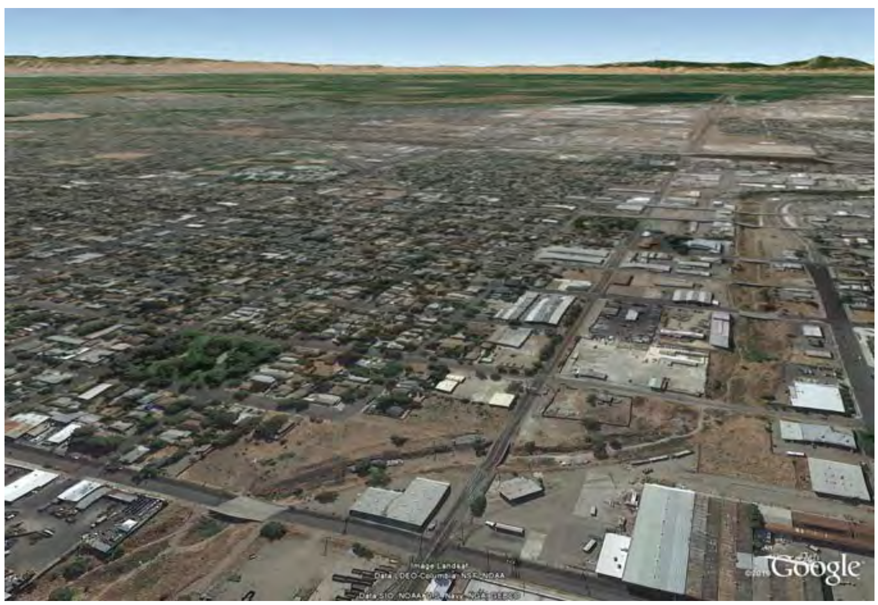
2013 Conditions, Source: Google

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

MORMAN DIVERTING CANAL
1955 FLOOD COMPARED TO 2013 CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Looking downstream (west) towards San Joaquin River, 1955 Flood

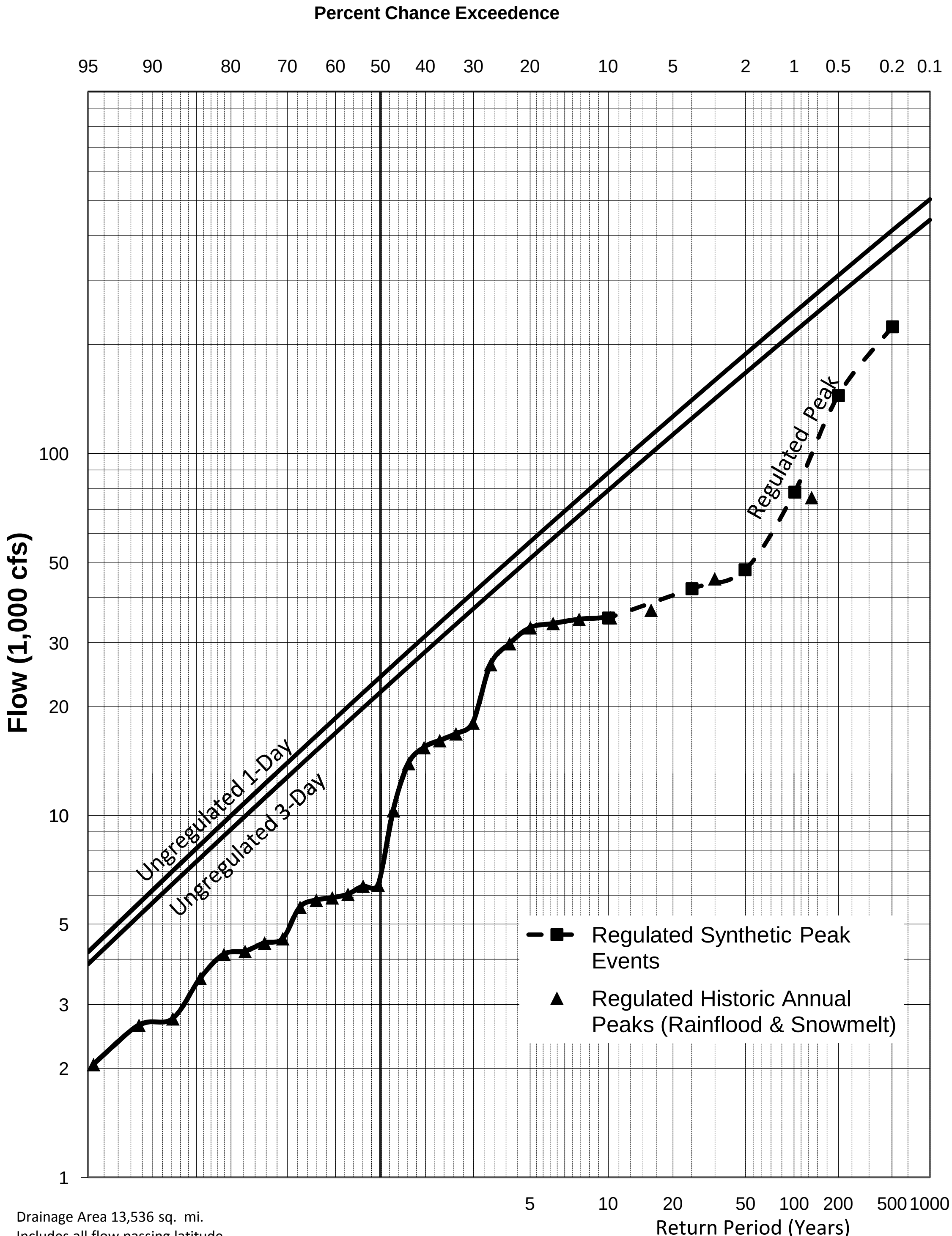


2013 Conditions, Source: Google Earth

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

MORMAN SLOUGH
1955 FLOOD COMPARED TO 2013 CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



Drainage Area 13,536 sq. mi.
Includes all flow passing latitude
Median Plotting Positions

UNREGULATED FLOW
Log Statistics

	Mean	Std Dev	Skew
1 - Day	4.375	0.450	-0.1
3 - Day	4.333	0.445	-0.1

Period of Record 1917-1998
Source: Sacramento-San Joaquin Basin Comprehensive Study, March 2002

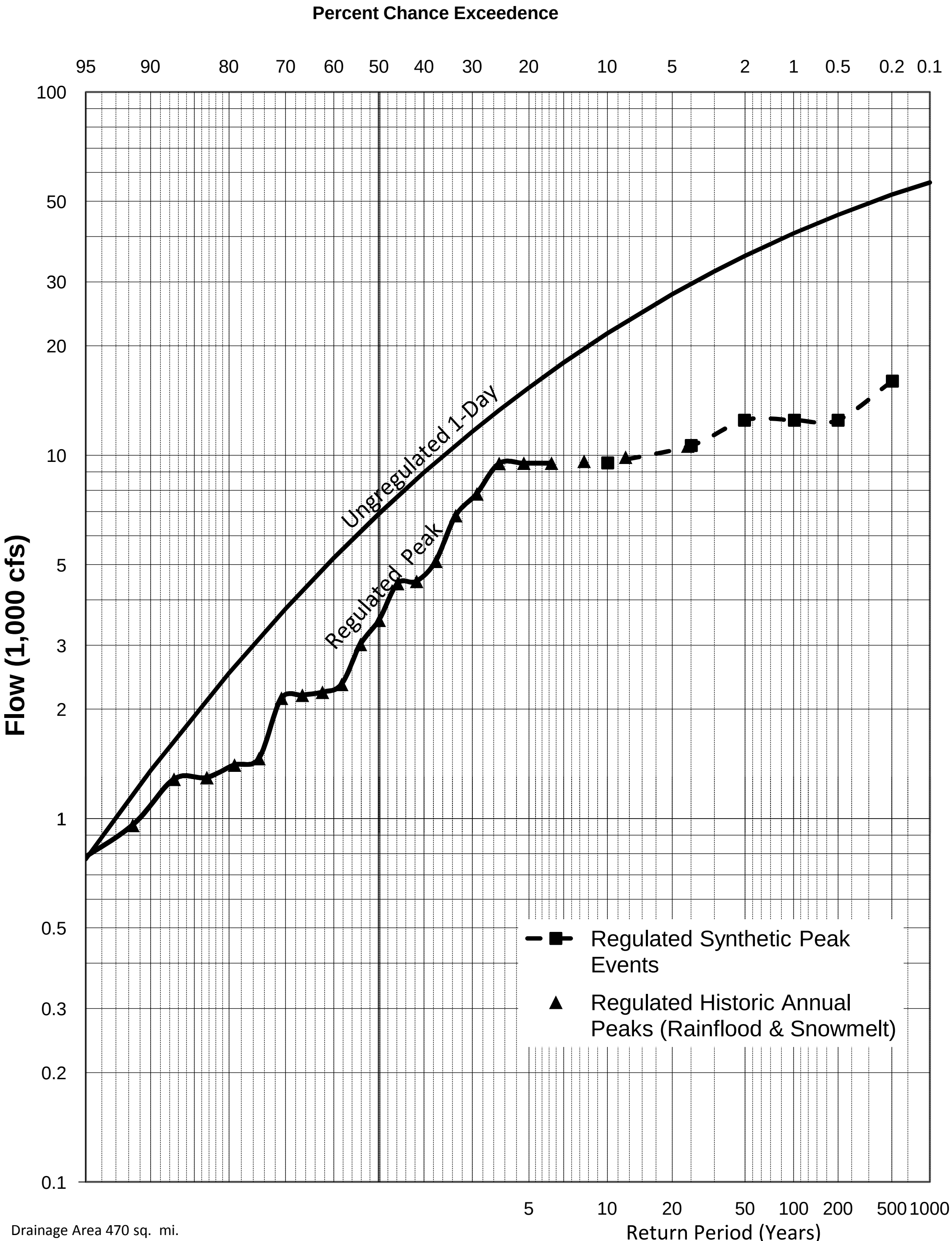
REGULATED PEAK FLOW
Beard Plotting Positions
Graphical Plot
Period of Record 1979 -2006

Regulated Hypothetical Events based on UNET modeling conducted for Sacramento-San Joaquin Basin Comprehensive Study, March 2002
1997 Peak Flow estimate did not account for overbank flow, USGS, 2013

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

SAN JOAQUIN RIVER NEAR VERNALIS FLOOD FLOW FREQUENCY

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



Drainage Area 470 sq. mi.
Includes all flow passing gage
Median Plotting Positions

UNREGULATED FLOW
Log Statistics

	Mean	Std Dev	Skew
1 - Day	3.775	0.482	-0.81

Period of Record 1907-2010
Source: Lower San Joaquin Feasibility Study
Hydraulic Appendix, May 2014

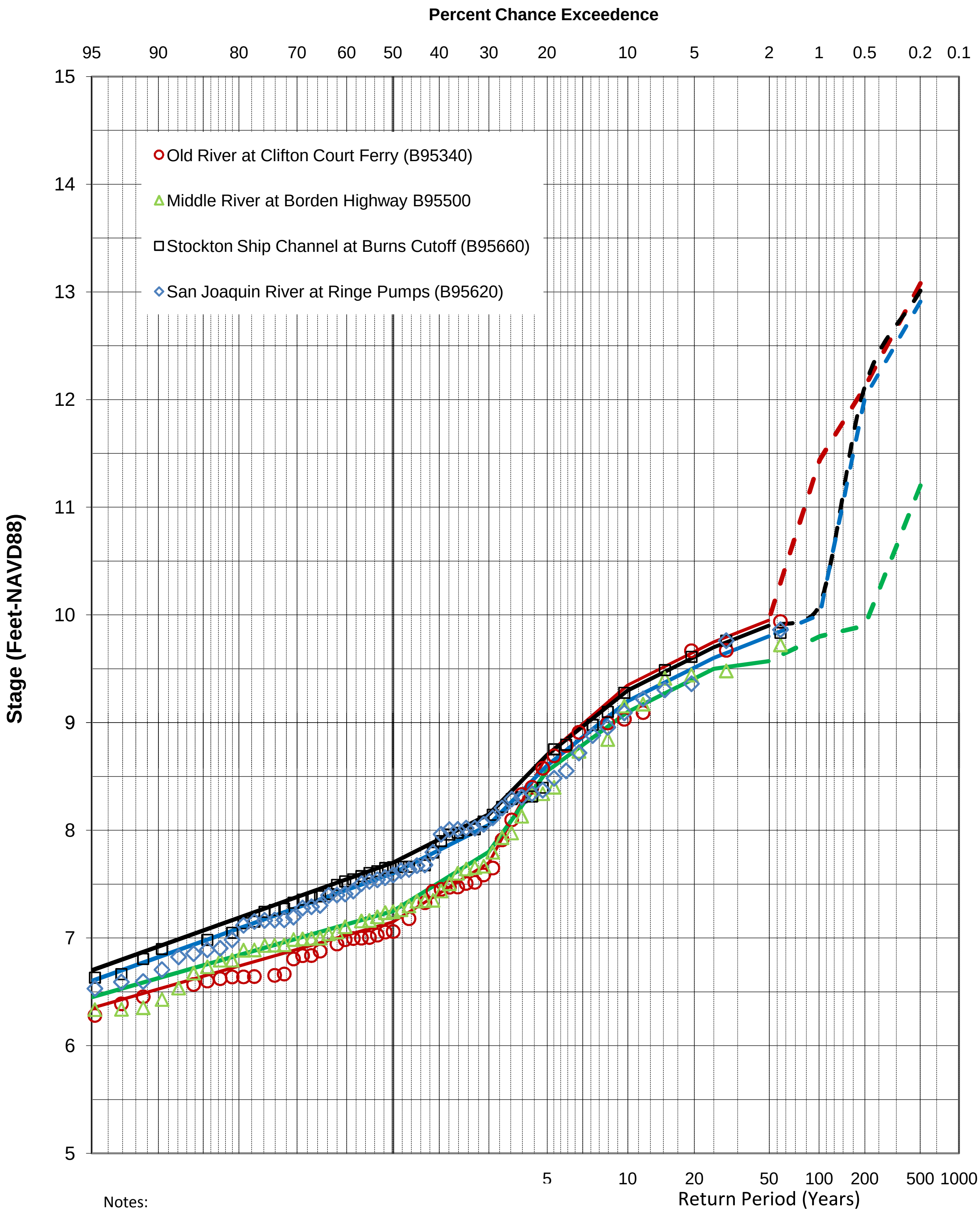
REGULATED PEAK FLOW
Weibull Plotting Positions
Graphical Plot
Period of Record 1988 -2011

Regulated Hypothetical Events based on Reservoir Simulation Model

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

MORMON SLOUGH AT BELLOTA
FLOOD FLOW FREQUENCY

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

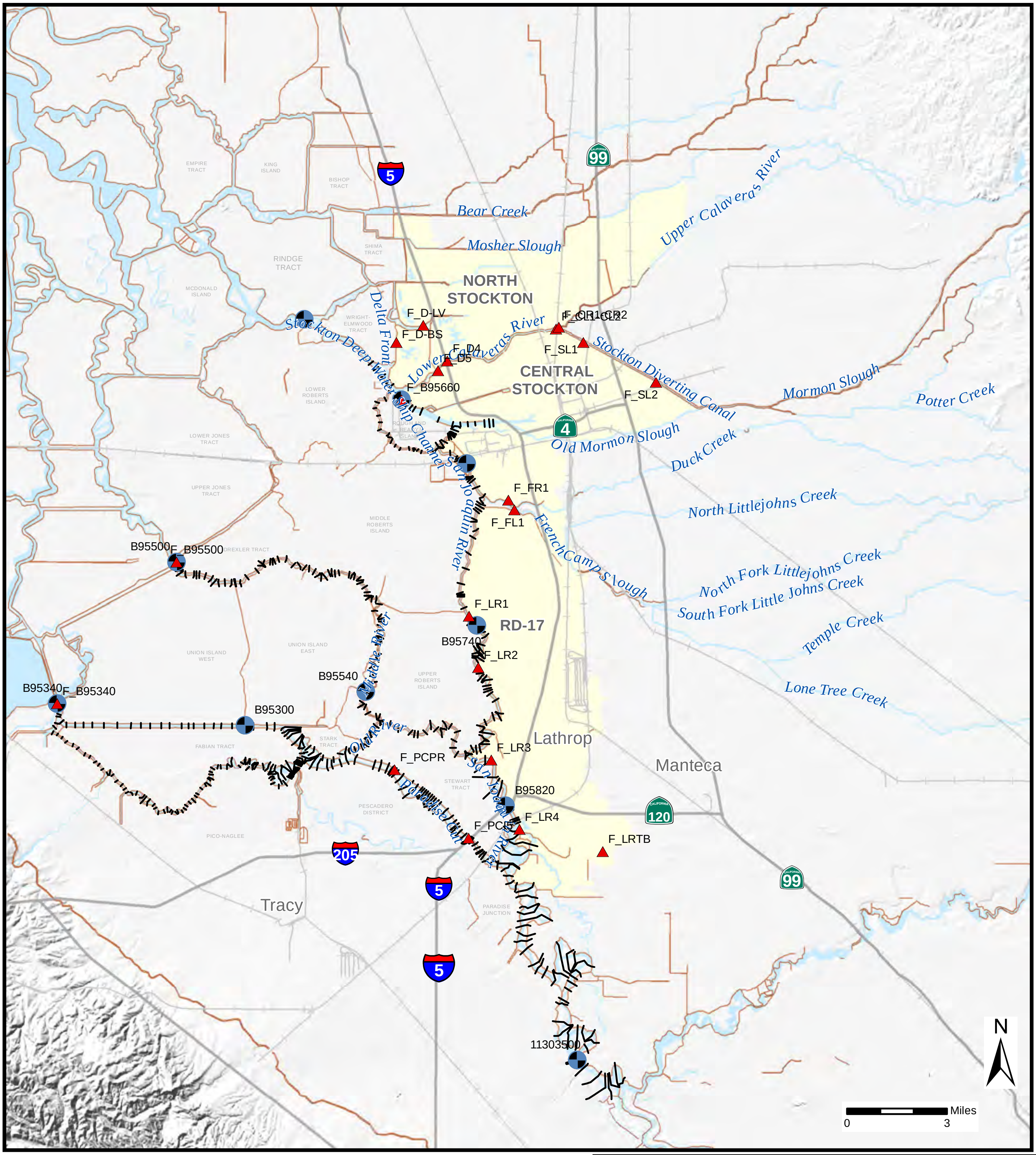


Notes:
1. Period of Record 1953 to 2009
2. Missing Records estimated by correlation:
   B95340: 1953-1957,1971,1987,1997
   B95500: 1958,1973,1989
   B95620: No missing data
   B95660: 1953-1958
3. Historic stages adjusted to 2010 Sea Level using historical 1.7mm/yr eustatic sea level rise
4. Extrapolation to from 1% to 0.2% (dashed) based on HEC-RAS Model results. While suitable for economic analysis, estimates should be refined for design purposes.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

STAGE-FREQUENCY CURVES
HEC-RAS DOWNSTREAM BOUNDARIES
2010 CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



Legend

- Stream Gages
- Model Cross-Sections
- Highway
- Railroads
- Levees (Fed/Non-Fed)
- Study Extent
- Index Point

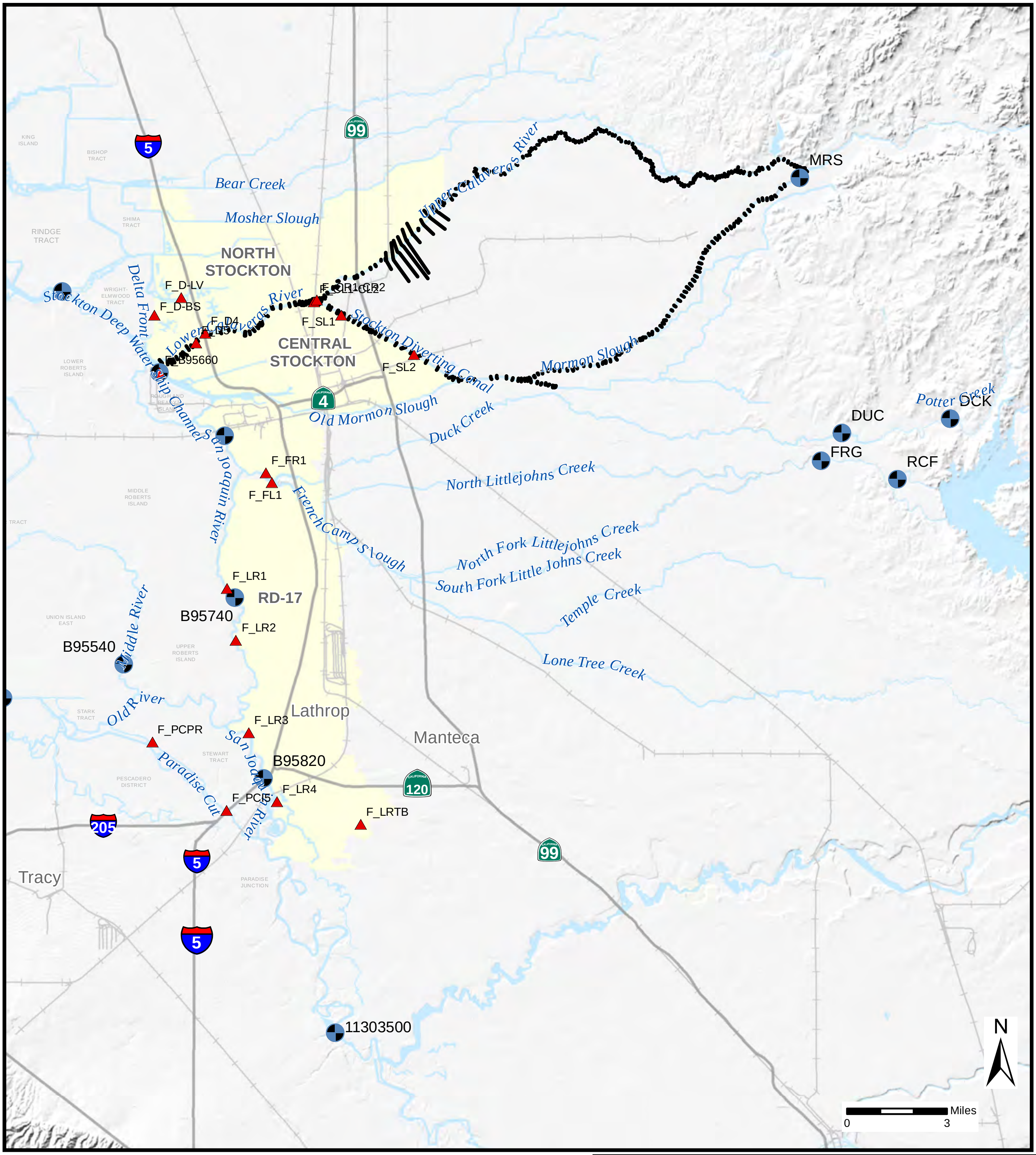
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

SAN JOAQUIN RIVER HEC-RAS MODEL EXTENT

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUG 2014



Legend

- Stream Gages
- Model Cross Sections
- Highway
- Railroads
- Levees (Fed/Non-Fed)
- Study Extent
- Index Point

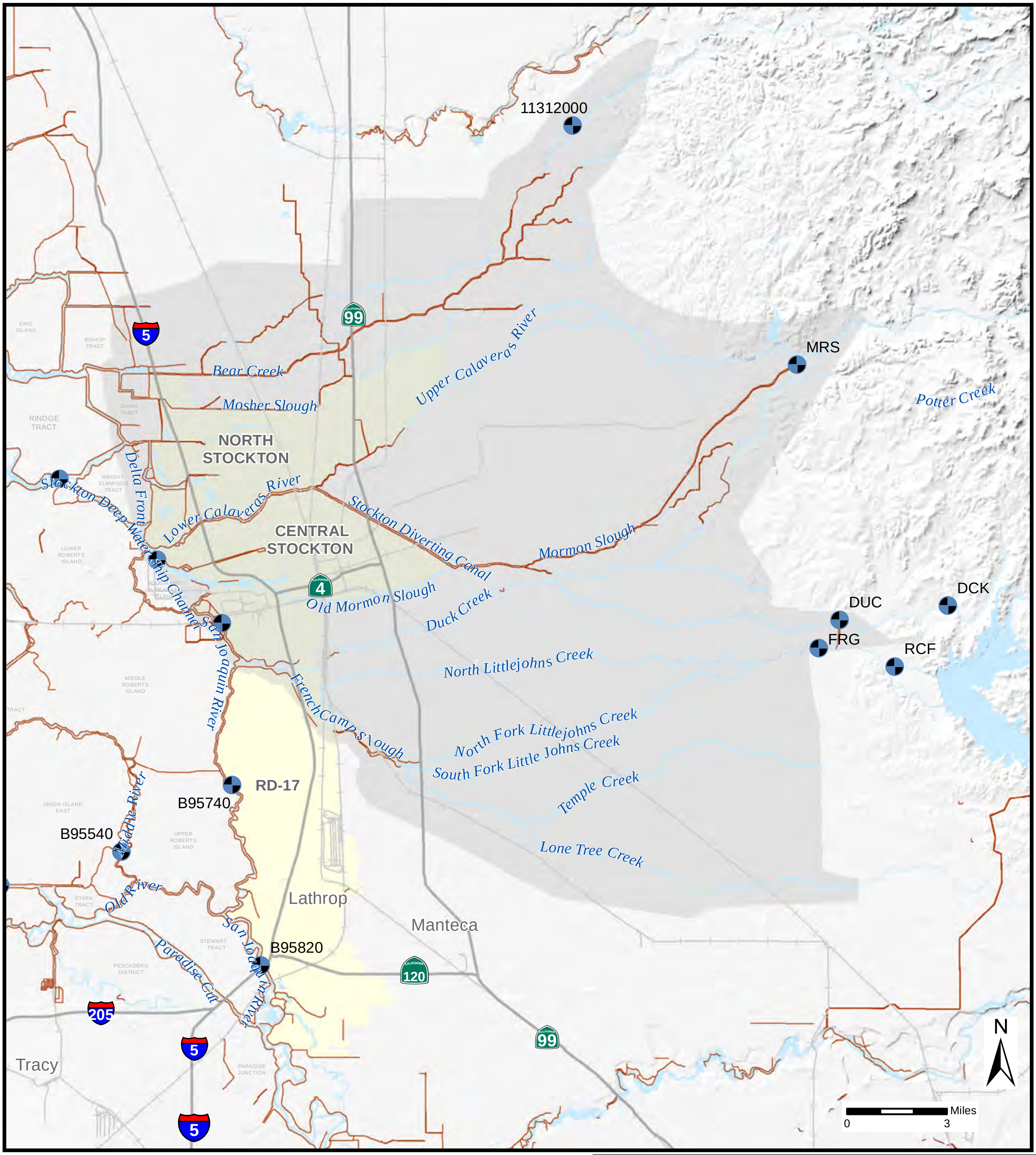
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

CALAVERAS RIVER
HEC-RAS MODEL

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUG 2014



Legend

- Stream Gages
- Highway
- Railroads
- Study Extent
- North FLO2D Model Extent

NOTE: Analysis Limited to Study Extent.

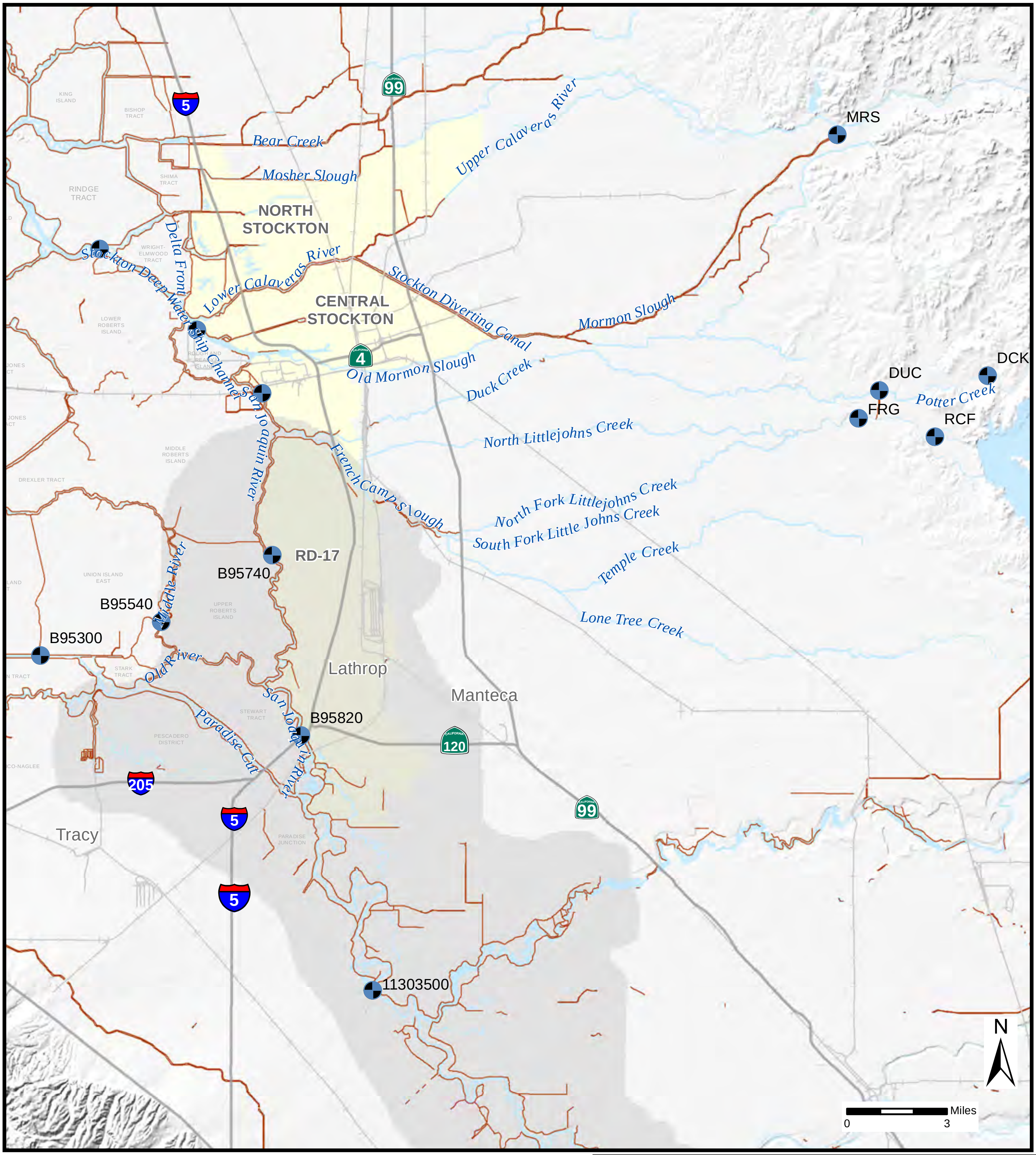
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NORTH FLO-2D MODEL DOMAIN

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014



Legend

- Stream Gages
- Highway
- Railroads
- Study Extent
- South FLO2D Model Extents

NOTE: Analysis Limited to Study Extent.

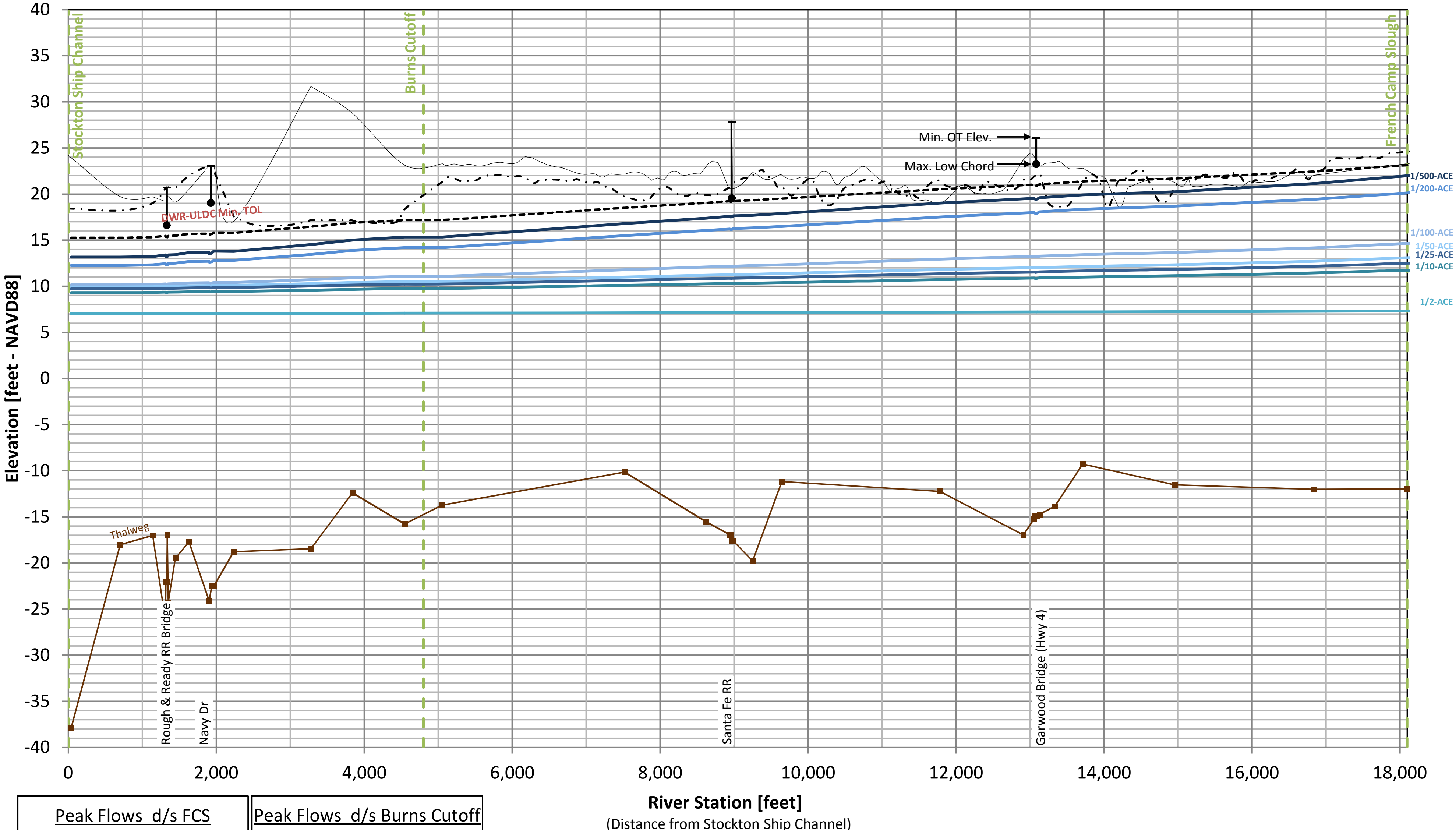
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

SOUTH FLO-2D MODEL DOMAIN

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014



Peak Flows d/s FCS
1/2-ACE - 2,450 cfs
1/10-ACE - 9,870 cfs
1/25-ACE - 11,150 cfs
1/50-ACE - 12,300 cfs
1/100-ACE - 15,900 cfs
1/200-ACE - 28,700 cfs
1/500-ACE - 35,200 cfs

Peak Flows d/s Burns Cutoff
1/2-ACE - 2,260 cfs
1/10-ACE - 8,930 cfs
1/25-ACE - 10,040 cfs
1/50-ACE - 11,040 cfs
1/100-ACE - 14,200 cfs
1/200-ACE - 25,000 cfs
1/500-ACE - 28,500 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

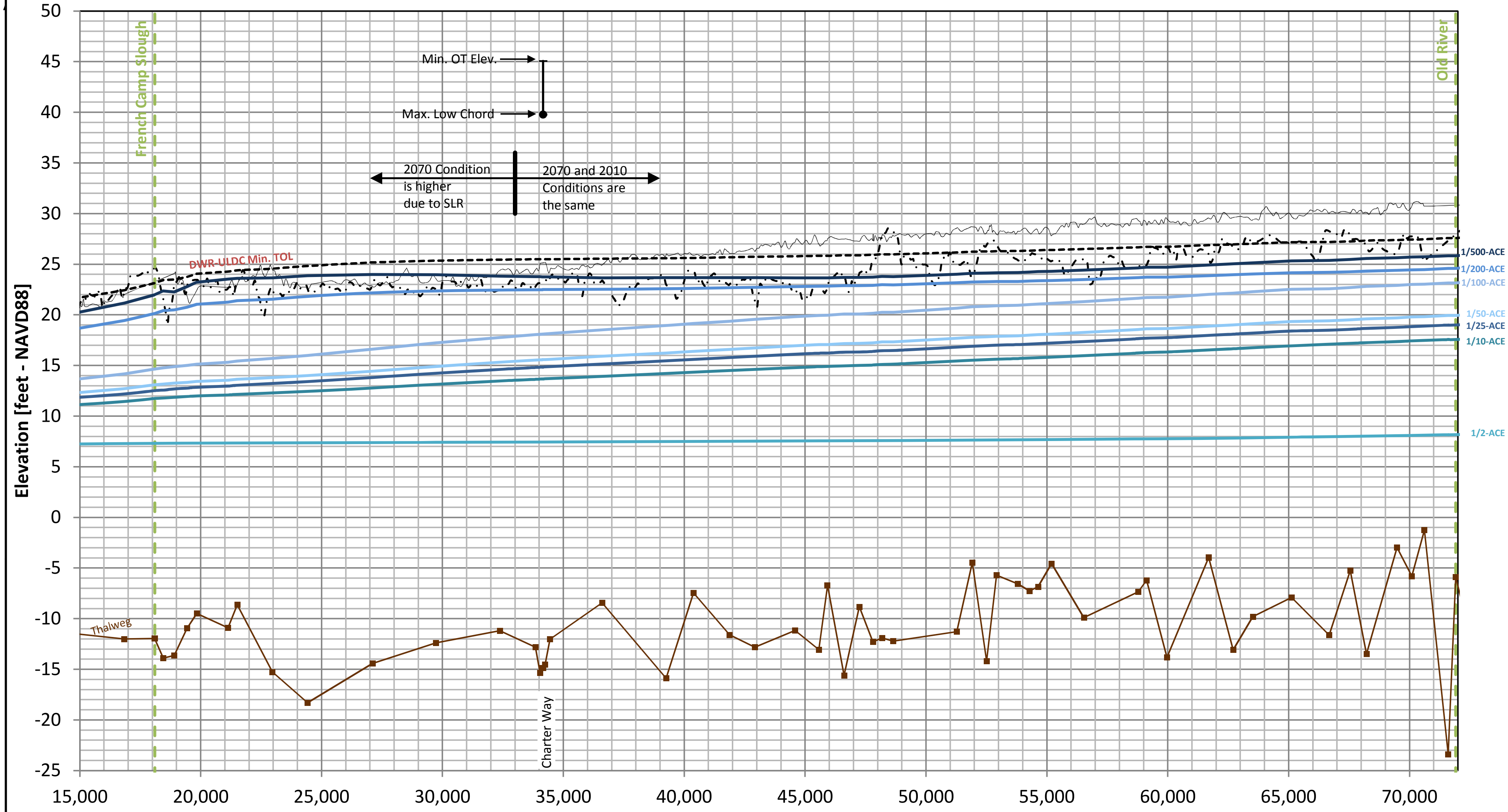
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
Stockton Ship Channel to French Camp Slough
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 25A



Peak Flows d/s Old River
1/2-ACE - 1,770 cfs
1/10-ACE - 7,800 cfs
1/25-ACE - 9,100 cfs
1/50-ACE - 10,100 cfs
1/100-ACE - 13,900 cfs
1/200-ACE - 15,800 cfs
1/500-ACE - 17,000 cfs

Peak Flows d/s FCS
1/2-ACE - 2,450 cfs
1/10-ACE - 9,870 cfs
1/25-ACE - 11,150 cfs
1/50-ACE - 12,300 cfs
1/100-ACE - 15,900 cfs
1/200-ACE - 28,700 cfs
1/500-ACE - 35,200 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

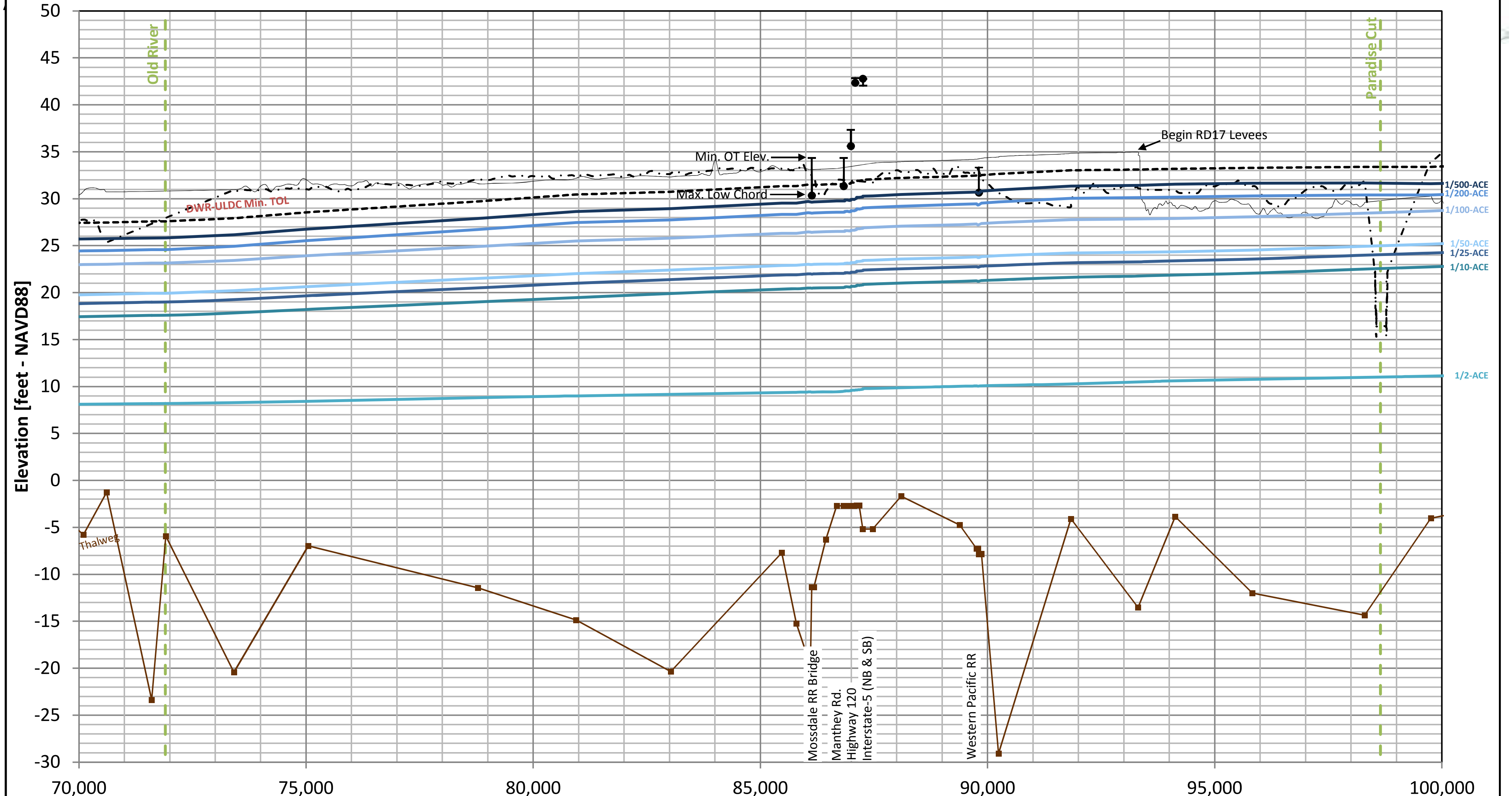
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
French Camp Slough to Old River
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 25B



Peak Flows d/s Paradise Cut
1/2-ACE - 6,390 cfs
1/10-ACE -25,200cfs
1/25-ACE - 28,900 cfs
1/50-ACE - 31,600 cfs
1/100-ACE - 43,000 cfs
1/200-ACE - 47,300 cfs
1/500-ACE - 49,400 cfs

Peak Flows d/s Old River
1/2-ACE - 1,770 cfs
1/10-ACE - 7,800 cfs
1/25-ACE - 9,100 cfs
1/50-ACE - 10,100 cfs
1/100-ACE - 13,900 cfs
1/200-ACE - 15,800 cfs
1/500-ACE - 17,000 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

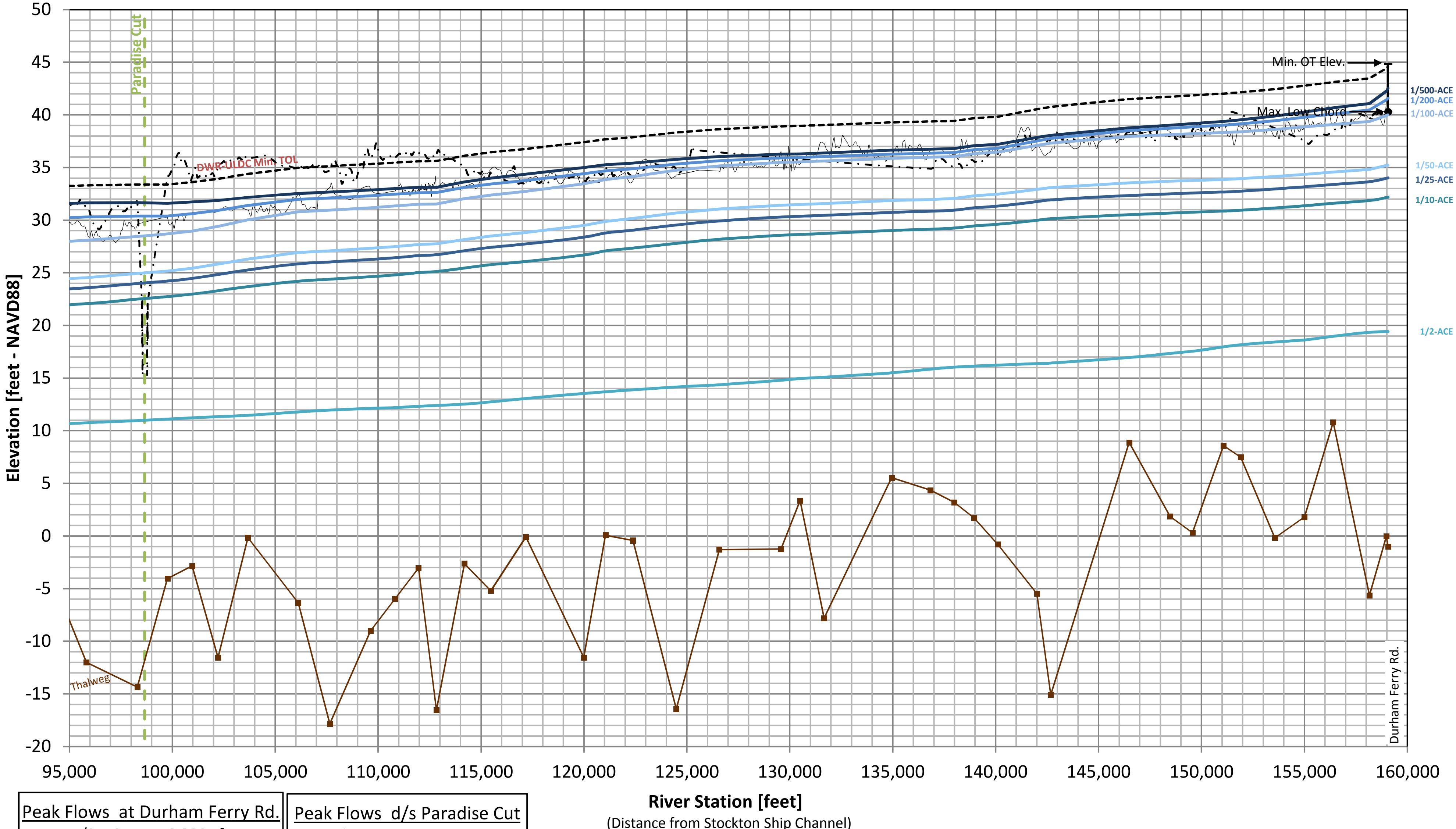
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
Old River to Paradise Cut
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 25C



Peak Flows at Durham Ferry Rd.
1/2-ACE - 6,390 cfs
1/10-ACE - 35,100 cfs
1/25-ACE - 42,300 cfs
1/50-ACE - 47,600 cfs
1/100-ACE - 78,200 cfs
1/200-ACE - 124,500 cfs
1/500-ACE - 165,100 cfs

Peak Flows d/s Paradise Cut
1/2-ACE - 6,390 cfs
1/10-ACE - 25,200cfs
1/25-ACE - 28,900 cfs
1/50-ACE - 31,600 cfs
1/100-ACE - 43,000 cfs
1/200-ACE - 47,300 cfs
1/500-ACE - 49,400 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

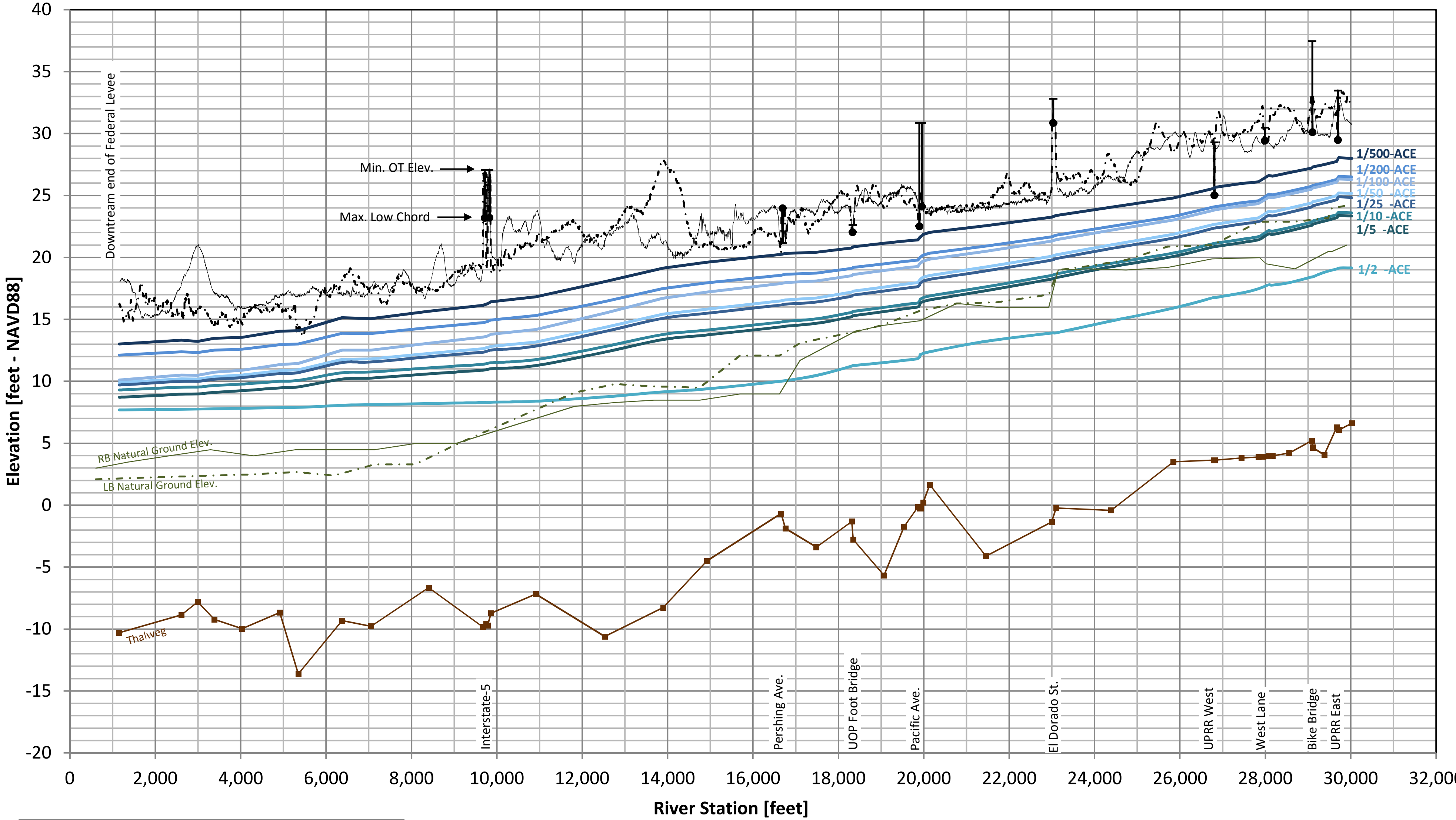
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
Paradise Cut to Durham Ferry Road
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 25D



Peak Flows in Reach

1/2-ACE - 3,850 cfs	1/50-ACE - 12,850 cfs
1/5-ACE - 9,500 cfs	1/100-ACE - 15,360 cfs
1/10-ACE - 9,860 cfs	1/200-ACE - 15,750 cfs
1/25-ACE - 12,280 cfs	1/500-ACE - 19,130 cfs

Top of Levee - RB

Top of Levee - LB

Bridge (Max. Low Chord; Min. OT Elev.)

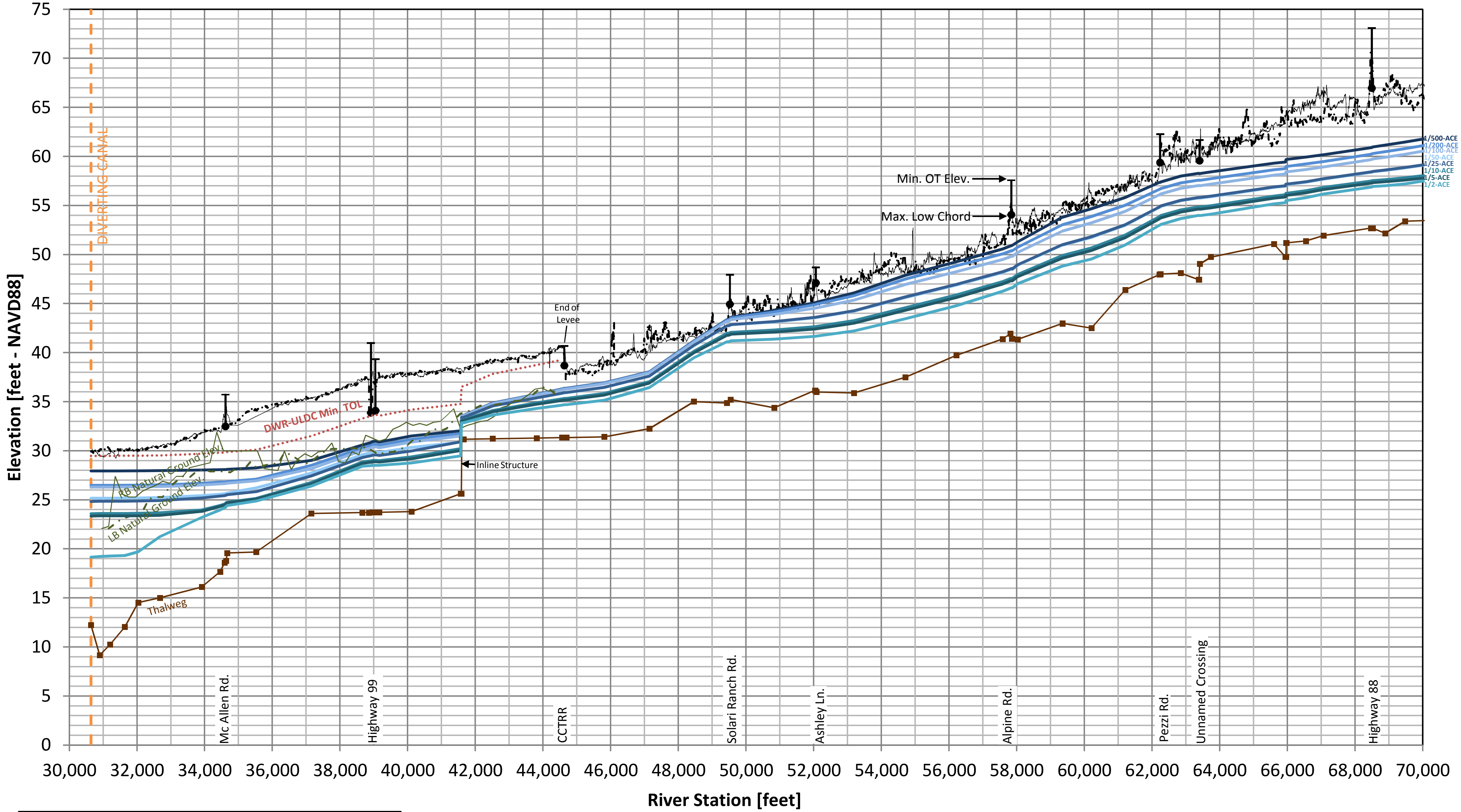
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Downstream of Diverting Canal
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

DEC 2015

PLATE 26



<u>Peak Flows at Highway 88</u>

1/2-ACE - 170 cfs	1/50-ACE - 340 cfs
1/5-ACE - 220 cfs	1/100-ACE - 490 cfs
1/10-ACE - 240 cfs	1/200-ACE - 570 cfs
1/25-ACE - 340 cfs	1/500-ACE - 680 cfs

Top of Levee/Bank - RB

Top of Levee/Bank - LB

Stream Junction

Bridge (Max. Low Chord; Min. OT Elev.)

September 18, 2013

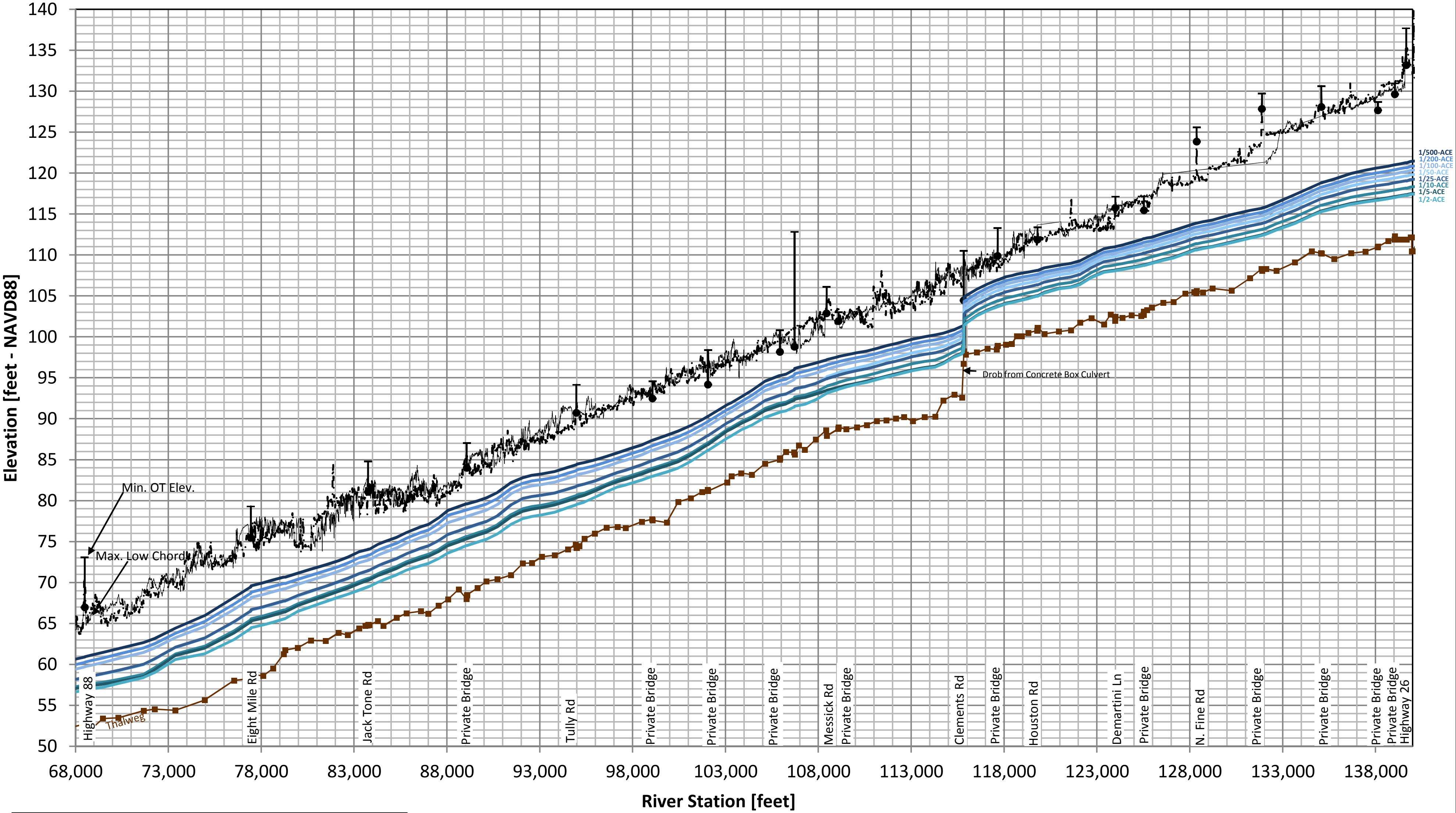
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Stockton Diverting Canal to Hwy 88
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 27A



Peak Flows at Fine Rd (d/s of Podesta Reservoir)

1/2-ACE - 150 cfs	1/50-ACE - 320 cfs
1/5-ACE - 160 cfs	1/100-ACE - 370 cfs
1/10-ACE - 210 cfs	1/200-ACE - 420 cfs
1/25-ACE - 270 cfs	1/500-ACE - 480 cfs

Top of Levee/Bank - RB

Top of Levee/Bank - LB

Stream Junction

Bridge (Max. Low Chord; Min. OT Elev.)

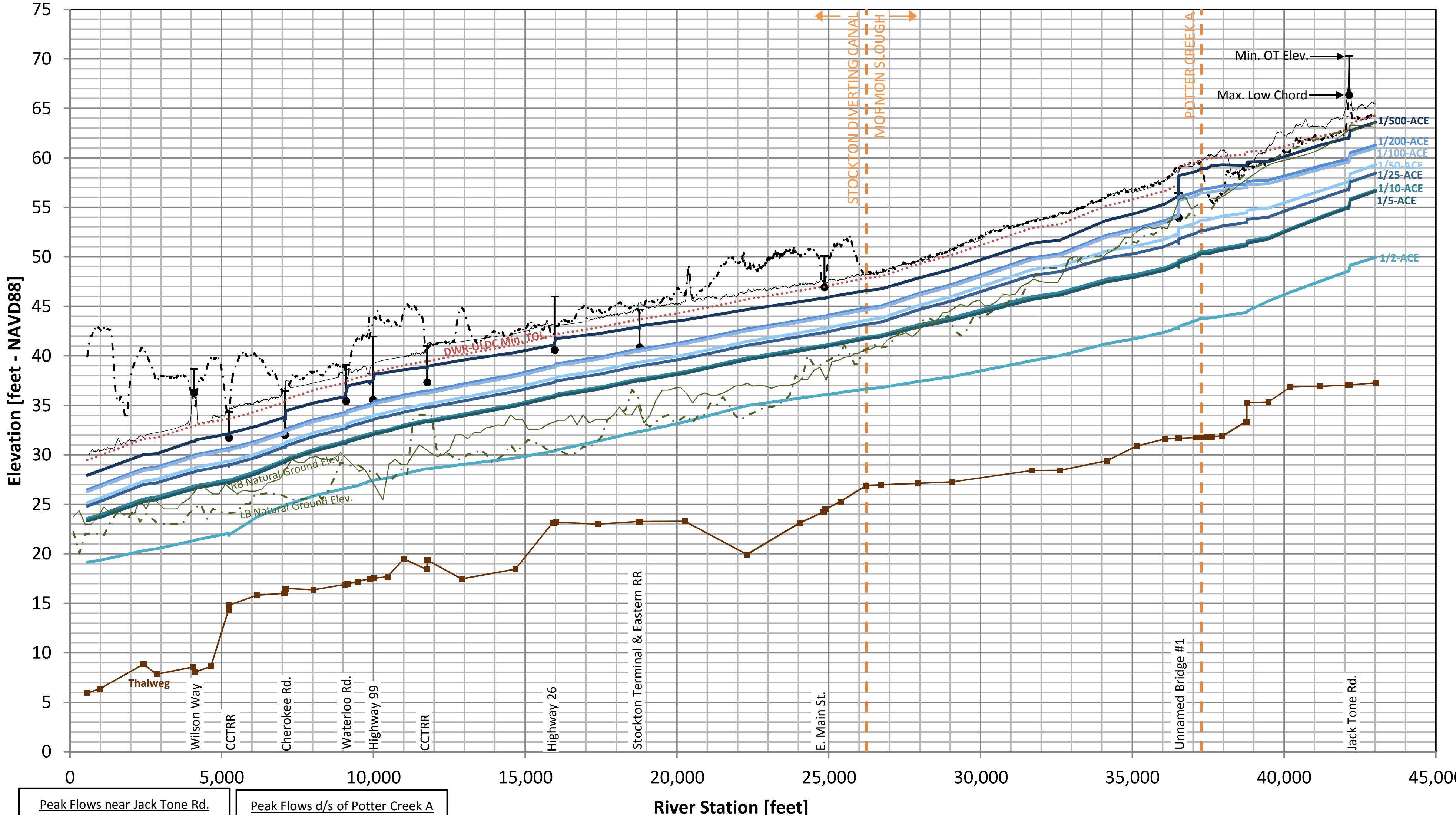
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Hwy 88 to Bellota
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 27B



Peak Flows near Jack Tone Rd.
1/2-ACE - 3,500 cfs
1/5-ACE - 9,400 cfs
1/10-ACE - 9,500 cfs
1/25-ACE - 11,300 cfs
1/50-ACE - 12,500 cfs
1/100-ACE - 13,800 cfs
1/200-ACE - 14,050 cfs
1/500-ACE - 17,450 cfs

Peak Flows d/s of Potter Creek A
1/2-ACE - 3,750 cfs
1/5-ACE - 9,400 cfs
1/10-ACE - 9,650 cfs
1/25-ACE - 11,950 cfs
1/50-ACE - 12,650 cfs
1/100-ACE - 14,950 cfs
1/200-ACE - 15,200 cfs
1-500-ACE - 18,400 cfs

Top of Levee - RB

Top of Levee - LB

Stream Junction

Bridge (Max. Low Chord; Min. OT Elev.)

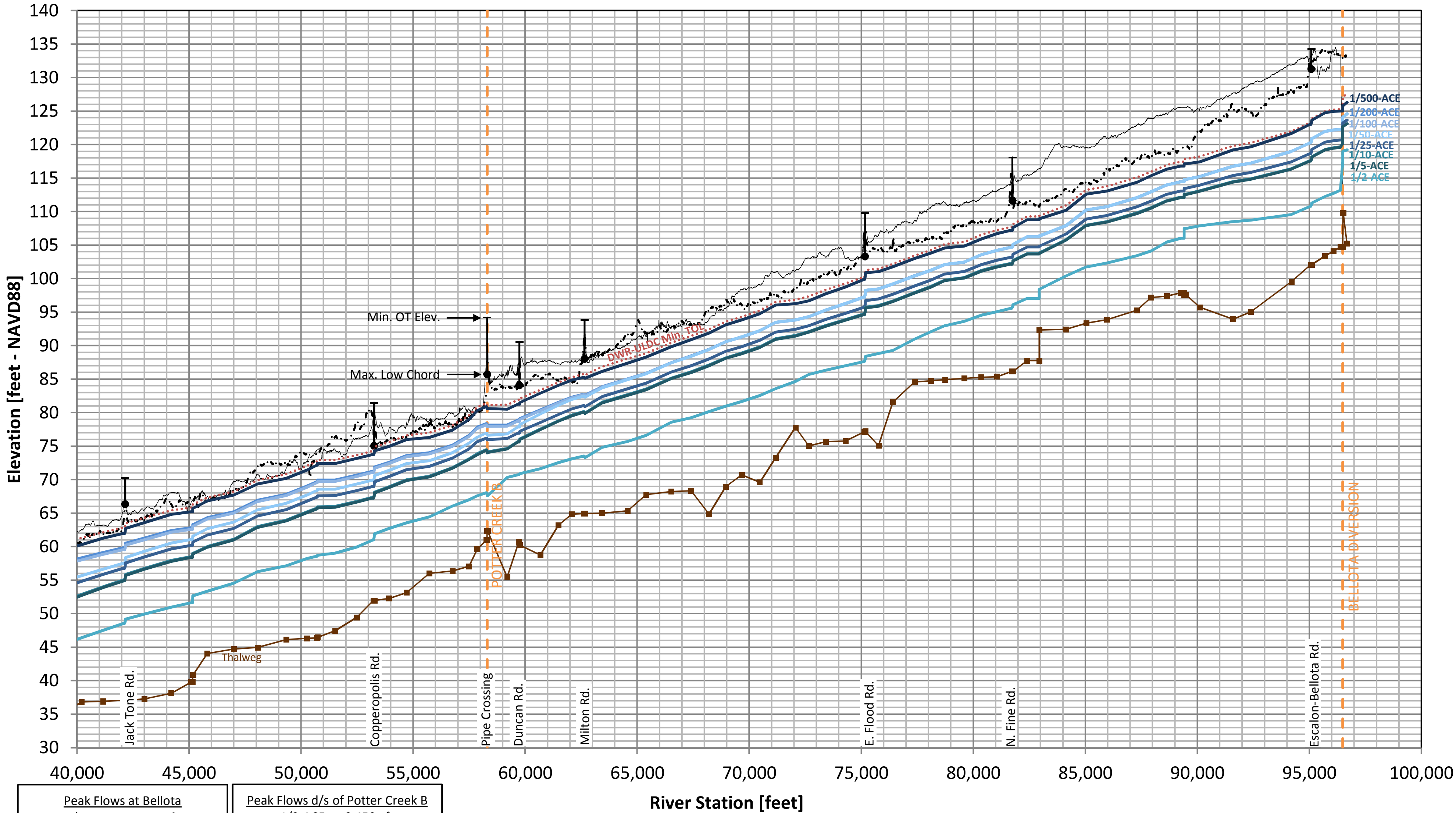
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STOCKTON DIVERTING CANAL /
MORMON SLOUGH
Downstream of Jack Tone Rd.
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 28A



Peak Flows at Bellota
1/2-ACE - 3,500 cfs
1/5-ACE - 9,500 cfs
1/10-ACE - 9,550 cfs
1/25-ACE - 10,650 cfs
1/50-ACE - 12,500 cfs
1/100-ACE - 12,500 cfs
1/200-ACE - 12,500 cfs
1/500-ACE - 16,000 cfs

Peak Flows d/s of Potter Creek B
1/2-ACE - 3,450 cfs
1/5-ACE - 9,350 cfs
1/10-ACE - 9,500 cfs
1/25-ACE - 11,350 cfs
1/50-ACE - 12,500 cfs
1/100-ACE - 13,800 cfs
1/200-ACE - 14,050 cfs
1-500-ACE - 17,650 cfs

Top of Levee/Bank - RB
Top of Levee/Bank - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

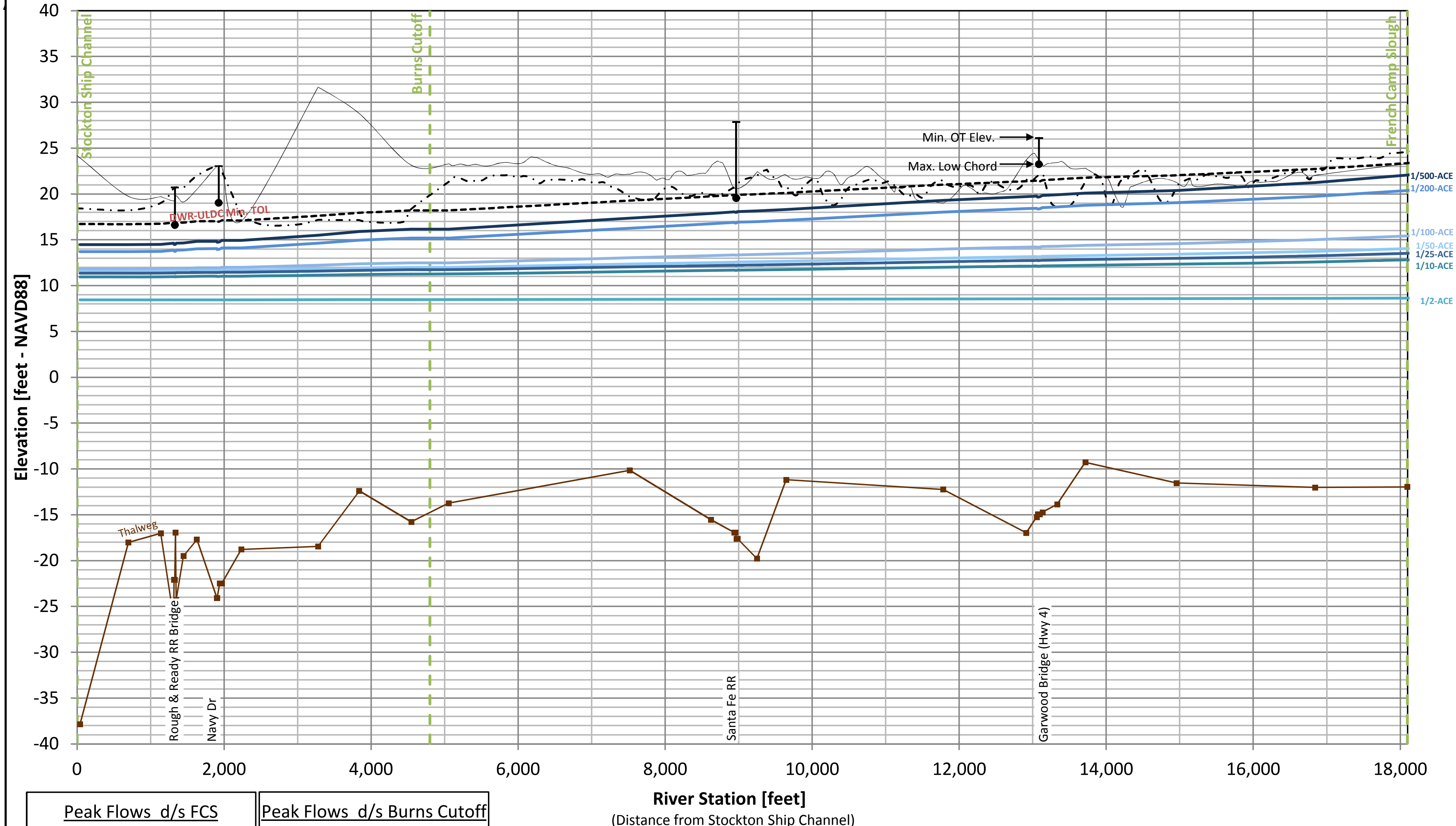
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STOCKTON DIVERTING CANAL /
MORMON SLOUGH
Upstream of Jack Tone Rd.
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 28B



Peak Flows d/s FCS
1/2-ACE - 2,270 cfs
1/10-ACE - 9,780 cfs
1/25-ACE - 11,040 cfs
1/50-ACE - 12,200 cfs
1/100-ACE - 15,800 cfs
1/200-ACE - 27,900 cfs
1/500-ACE - 34,600 cfs

Peak Flows d/s Burns Cutoff
1/2-ACE - 2,070 cfs
1/10-ACE - 8,720 cfs
1/25-ACE - 9,800 cfs
1/50-ACE - 10,800 cfs
1/100-ACE - 13,900 cfs
1/200-ACE - 24,100 cfs
1/500-ACE - 27,300 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

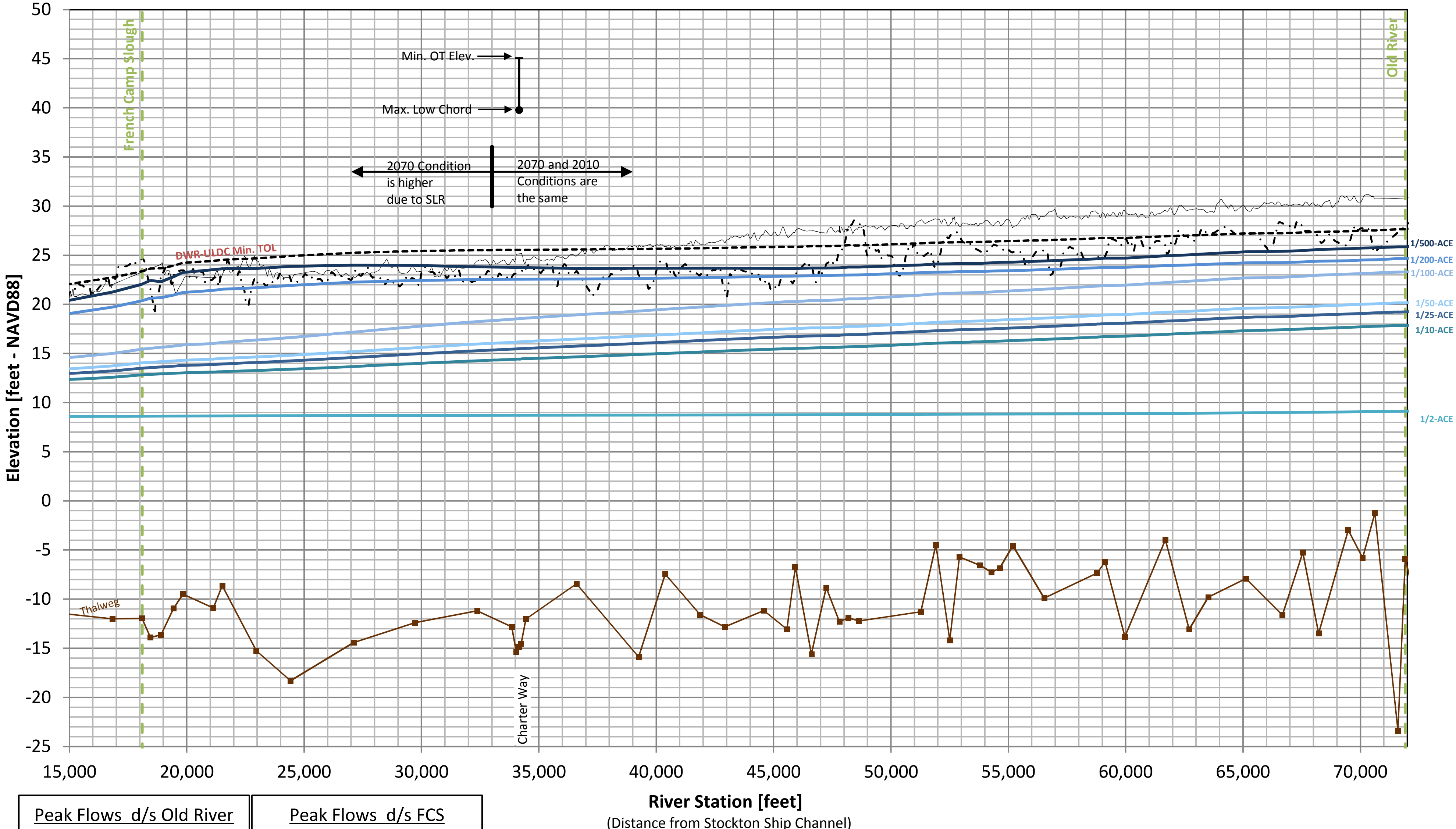
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
Stockton Ship Channel to French Camp Slough
2070 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 29A



Peak Flows d/s Old River
1/2-ACE - 1,700 cfs
1/10-ACE - 7,700 cfs
1/25-ACE - 9,000 cfs
1/50-ACE - 10,000 cfs
1/100-ACE - 13,800 cfs
1/200-ACE - 15,700 cfs
1/500-ACE - 16,900 cfs

Peak Flows d/s FCS
1/2-ACE - 2,270 cfs
1/10-ACE - 9,780 cfs
1/25-ACE - 11,040 cfs
1/50-ACE - 12,200 cfs
1/100-ACE - 15,800 cfs
1/200-ACE - 27,900 cfs
1/500-ACE - 34,600 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

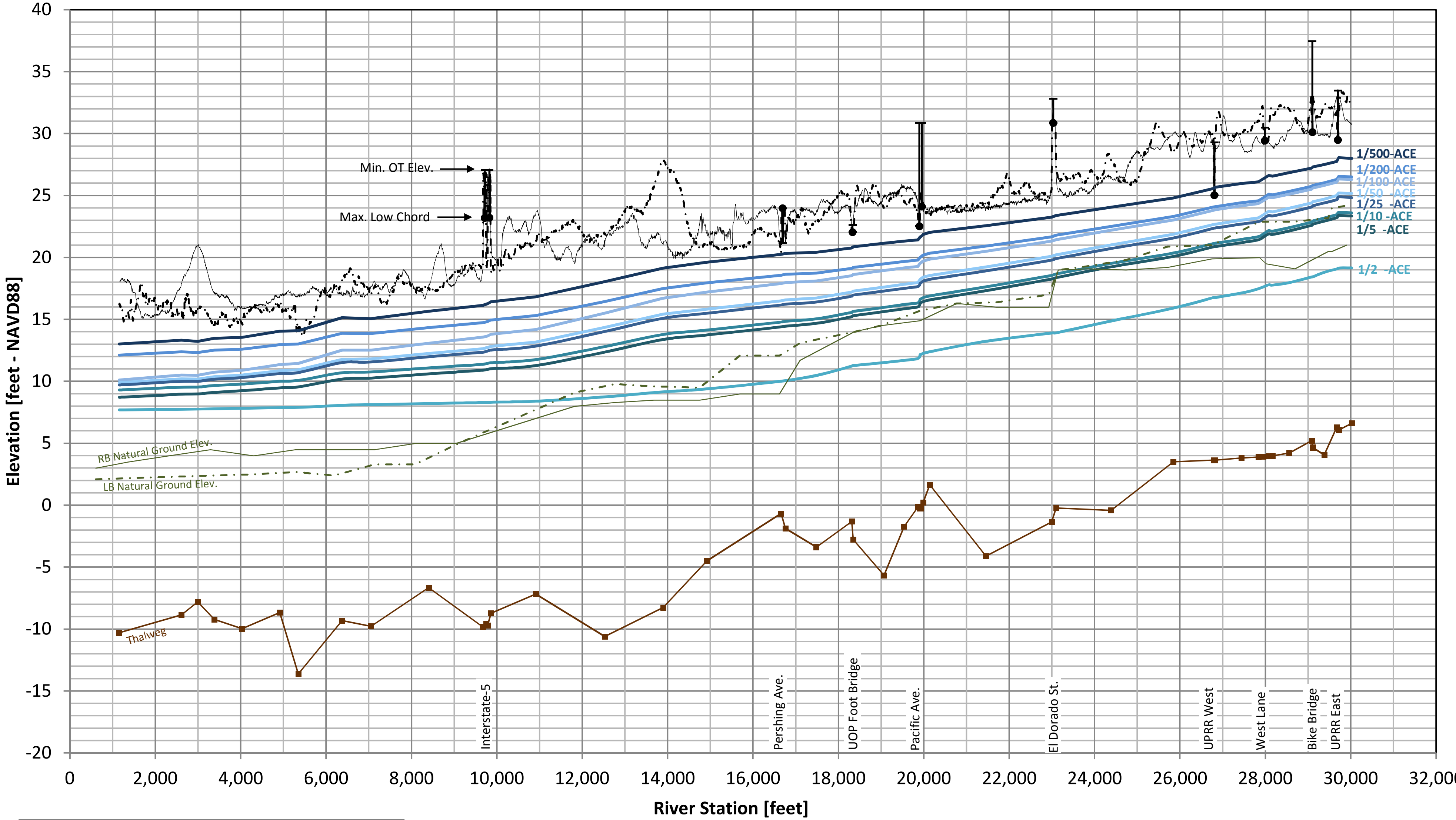
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
French Camp Slough to Old River
2070 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

FIGURE 29B



Peak Flows in Reach

1/2-ACE - 3,850 cfs   1/50-ACE - 12,850 cfs
1/5-ACE - 9,500 cfs   1/100-ACE - 15,360 cfs
1/10-ACE - 9,860 cfs   1/200-ACE - 15,750 cfs
1/25-ACE - 12,280 cfs   1/500-ACE - 19,130 cfs

Top of Levee - RB
Top of Levee - LB
Bridge (Max. Low Chord; Min. OT Elev.)

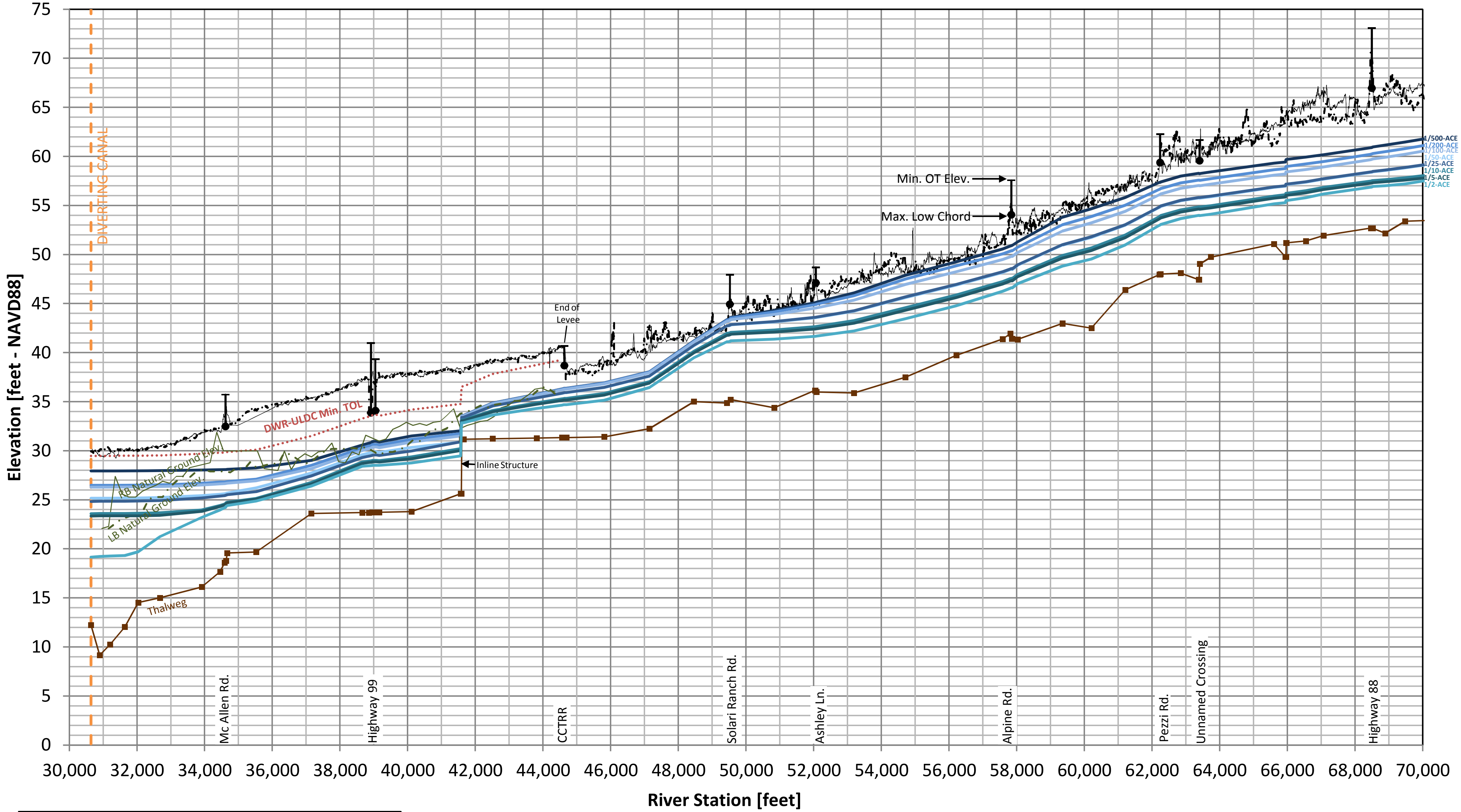
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Downstream of Diverting Canal
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 26



Peak Flows at Highway 88

1/2-ACE - 170 cfs	1/50-ACE - 340 cfs
1/5-ACE - 220 cfs	1/100-ACE - 490 cfs
1/10-ACE - 240 cfs	1/200-ACE - 570 cfs
1/25-ACE - 340 cfs	1/500-ACE - 680 cfs

Top of Levee/Bank - RB
Top of Levee/Bank - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

September 18, 2013

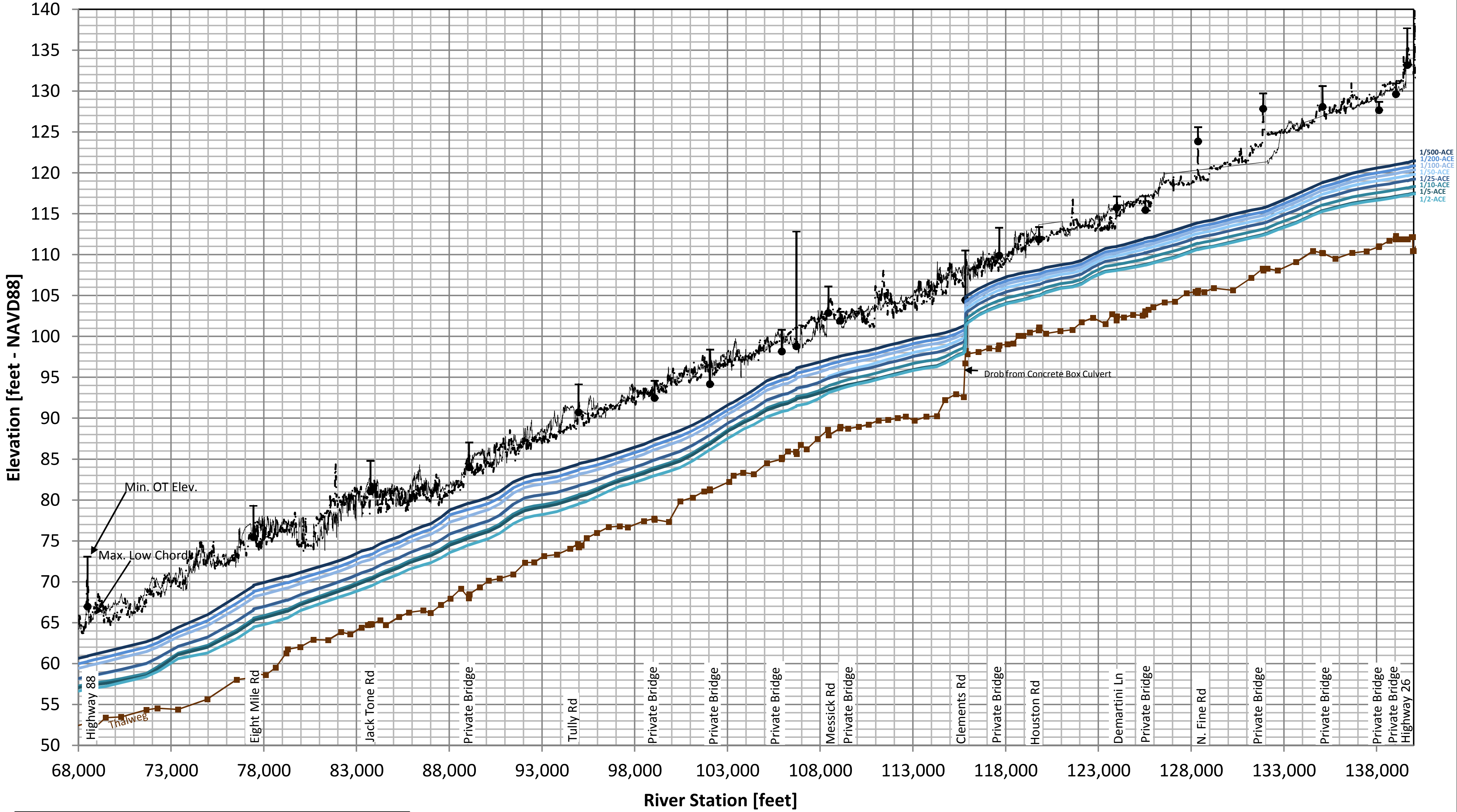
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Stockton Diverting Canal to Hwy 88
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 27A



Peak Flows at Fine Rd (d/s of Podesta Reservoir)

1/2-ACE - 150 cfs	1/50-ACE - 320 cfs
1/5-ACE - 160 cfs	1/100-ACE - 370 cfs
1/10-ACE - 210 cfs	1/200-ACE - 420 cfs
1/25-ACE - 270 cfs	1/500-ACE - 480 cfs

Top of Levee/Bank - RB
Top of Levee/Bank - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

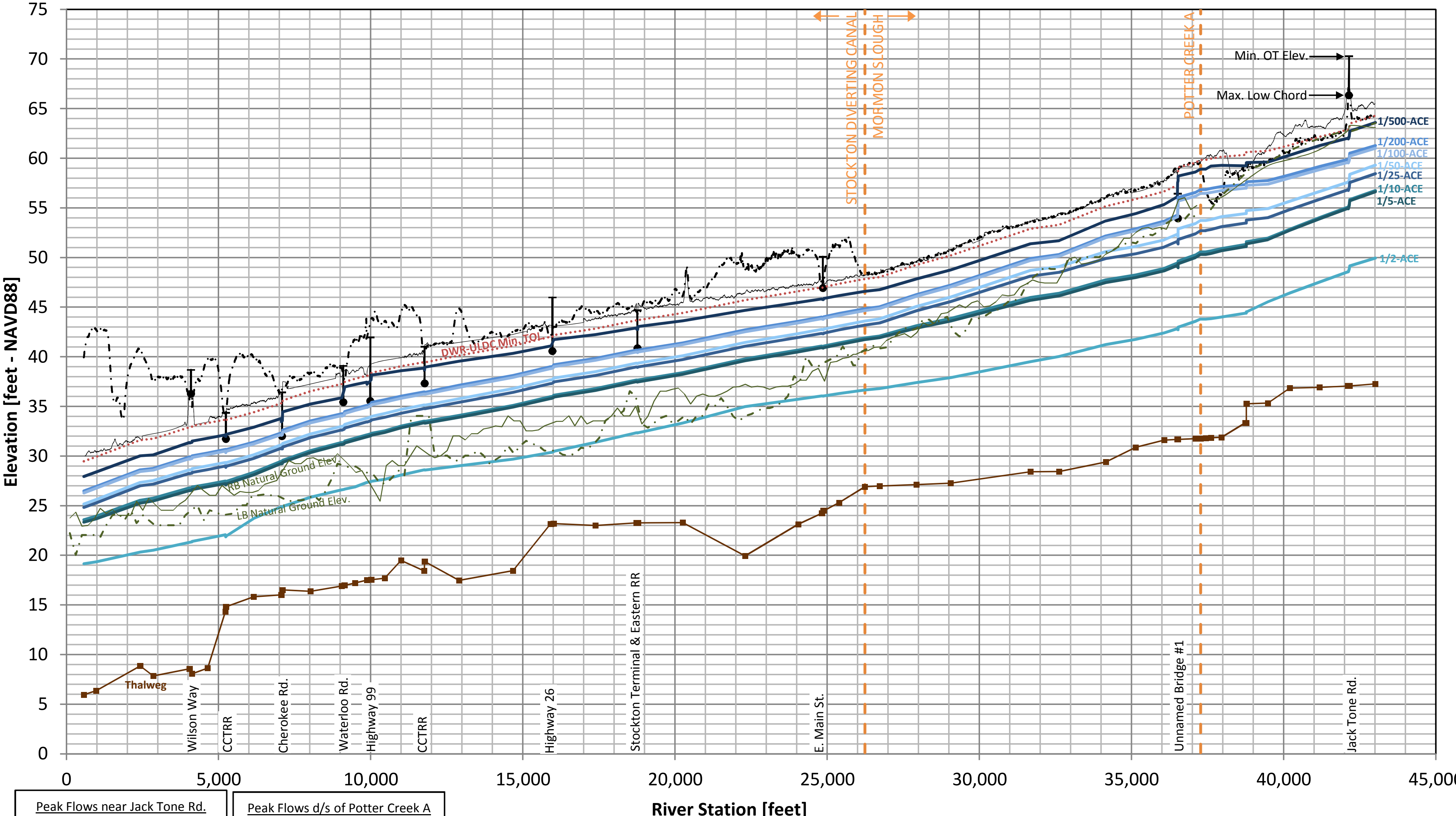
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Hwy 88 to Bellota
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 27B



Peak Flows near Jack Tone Rd.
1/2-ACE - 3,500 cfs
1/5-ACE - 9,400 cfs
1/10-ACE - 9,500 cfs
1/25-ACE - 11,300 cfs
1/50-ACE - 12,500 cfs
1/100-ACE - 13,800 cfs
1/200-ACE - 14,050 cfs
1/500-ACE - 17,450 cfs

Peak Flows d/s of Potter Creek A
1/2-ACE - 3,750 cfs
1/5-ACE - 9,400 cfs
1/10-ACE - 9,650 cfs
1/25-ACE - 11,950 cfs
1/50-ACE - 12,650 cfs
1/100-ACE - 14,950 cfs
1/200-ACE - 15,200 cfs
1-500-ACE - 18,400 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

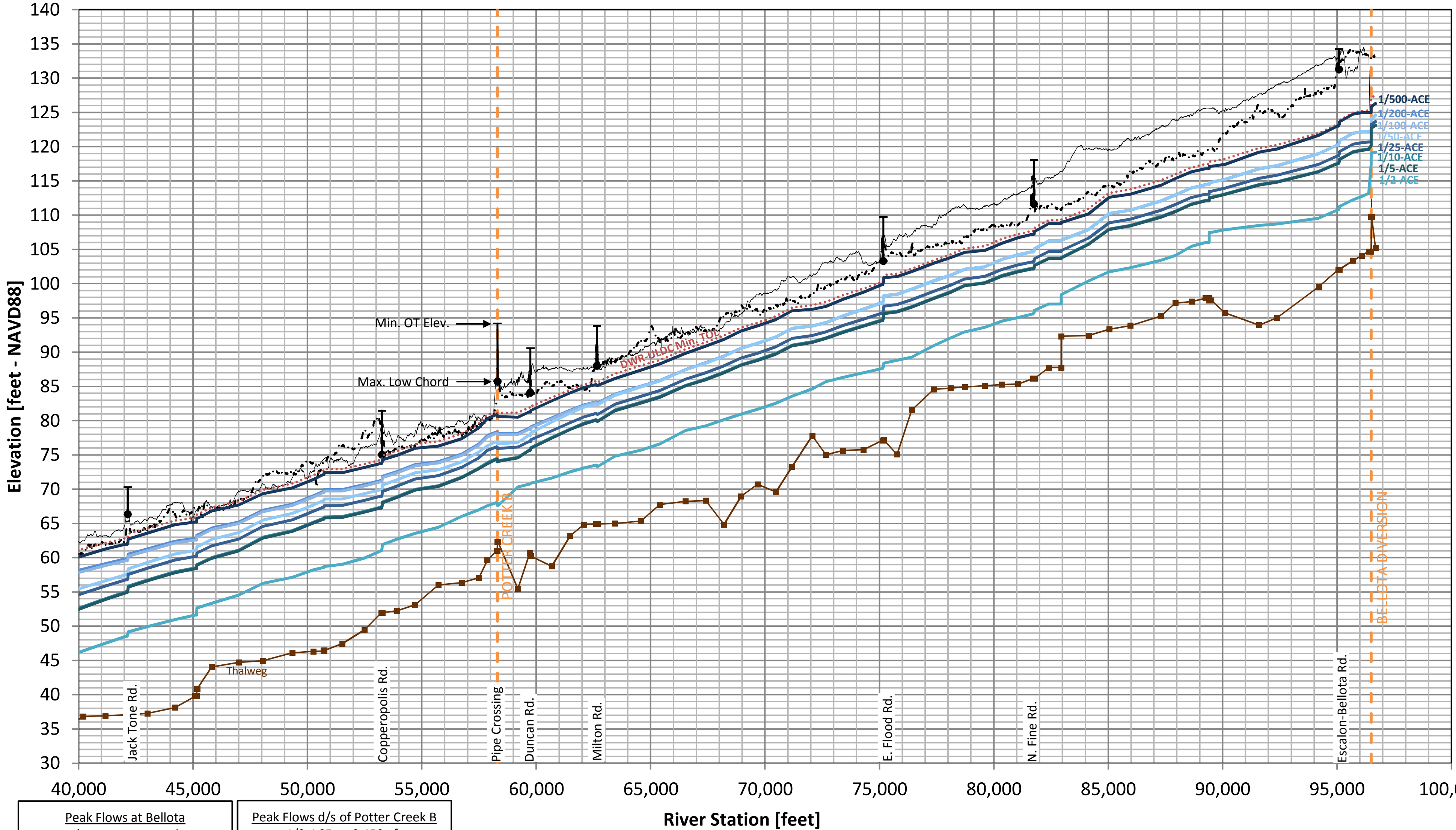
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STOCKTON DIVERTING CANAL /
MORMON SLOUGH
Downstream of Jack Tone Rd.
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 28A



Peak Flows at Bellota
1/2-ACE - 3,500 cfs
1/5-ACE - 9,500 cfs
1/10-ACE - 9,550 cfs
1/25-ACE - 10,650 cfs
1/50-ACE - 12,500 cfs
1/100-ACE - 12,500 cfs
1/200-ACE - 12,500 cfs
1/500-ACE - 16,000 cfs

Peak Flows d/s of Potter Creek B
1/2-ACE - 3,450 cfs
1/5-ACE - 9,350 cfs
1/10-ACE - 9,500 cfs
1/25-ACE - 11,350 cfs
1/50-ACE - 12,500 cfs
1/100-ACE - 13,800 cfs
1/200-ACE - 14,050 cfs
1-500-ACE - 17,650 cfs

Top of Levee/Bank - RB
Top of Levee/Bank - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

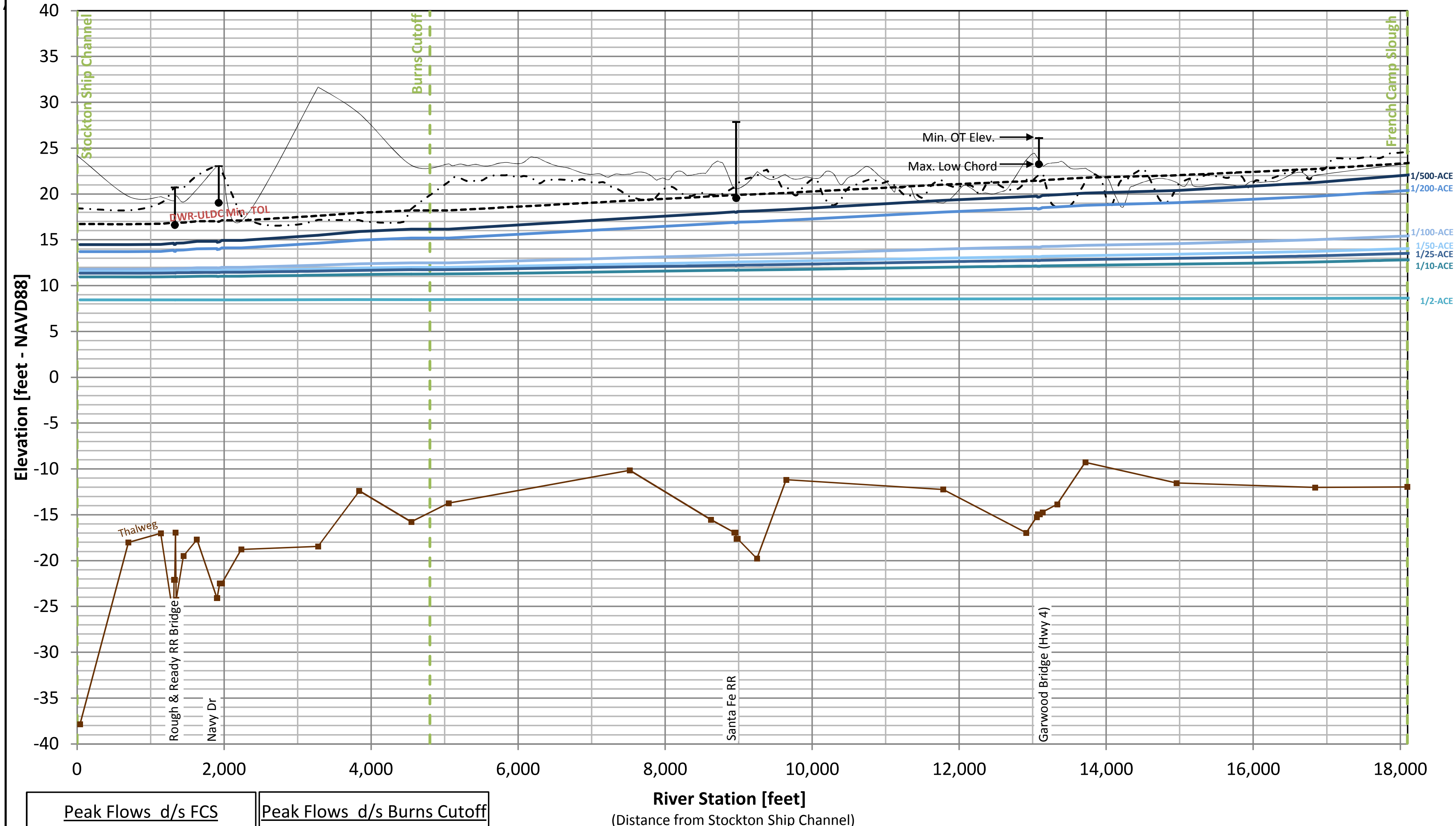
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STOCKTON DIVERTING CANAL /
MORMON SLOUGH
Upstream of Jack Tone Rd.
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 28B



Peak Flows d/s FCS
1/2-ACE - 2,270 cfs
1/10-ACE - 9,780 cfs
1/25-ACE - 11,040 cfs
1/50-ACE - 12,200 cfs
1/100-ACE - 15,800 cfs
1/200-ACE - 27,900 cfs
1/500-ACE - 34,600 cfs

Peak Flows d/s Burns Cutoff
1/2-ACE - 2,070 cfs
1/10-ACE - 8,720 cfs
1/25-ACE - 9,800 cfs
1/50-ACE - 10,800 cfs
1/100-ACE - 13,900 cfs
1/200-ACE - 24,100 cfs
1/500-ACE - 27,300 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

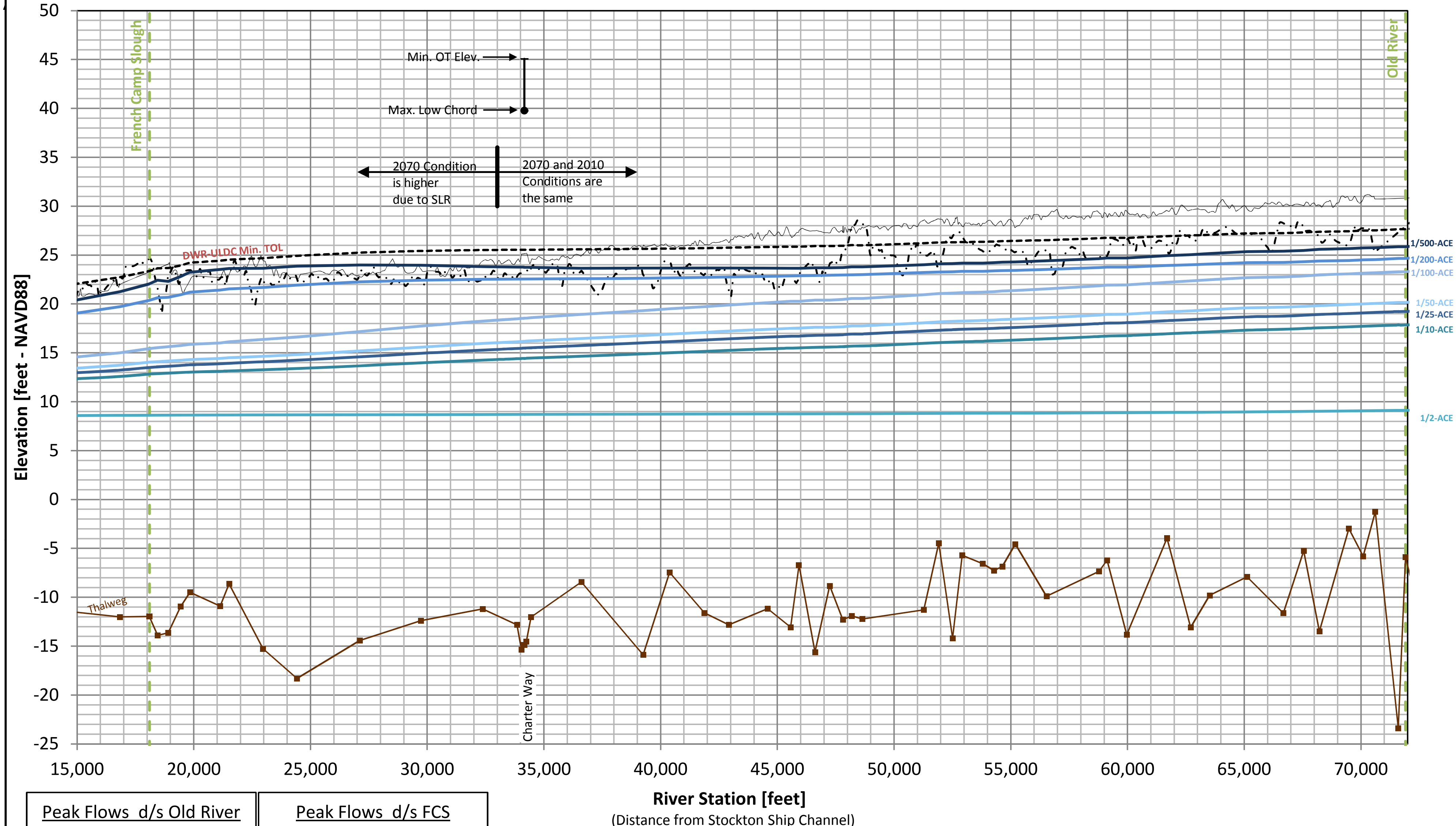
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
Stockton Ship Channel to French Camp Slough
2070 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 29A



Peak Flows d/s Old River
1/2-ACE - 1,700 cfs
1/10-ACE - 7,700 cfs
1/25-ACE - 9,000 cfs
1/50-ACE - 10,000 cfs
1/100-ACE - 13,800 cfs
1/200-ACE - 15,700 cfs
1/500-ACE - 16,900 cfs

Peak Flows d/s FCS
1/2-ACE - 2,270 cfs
1/10-ACE - 9,780 cfs
1/25-ACE - 11,040 cfs
1/50-ACE - 12,200 cfs
1/100-ACE - 15,800 cfs
1/200-ACE - 27,900 cfs
1/500-ACE - 34,600 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

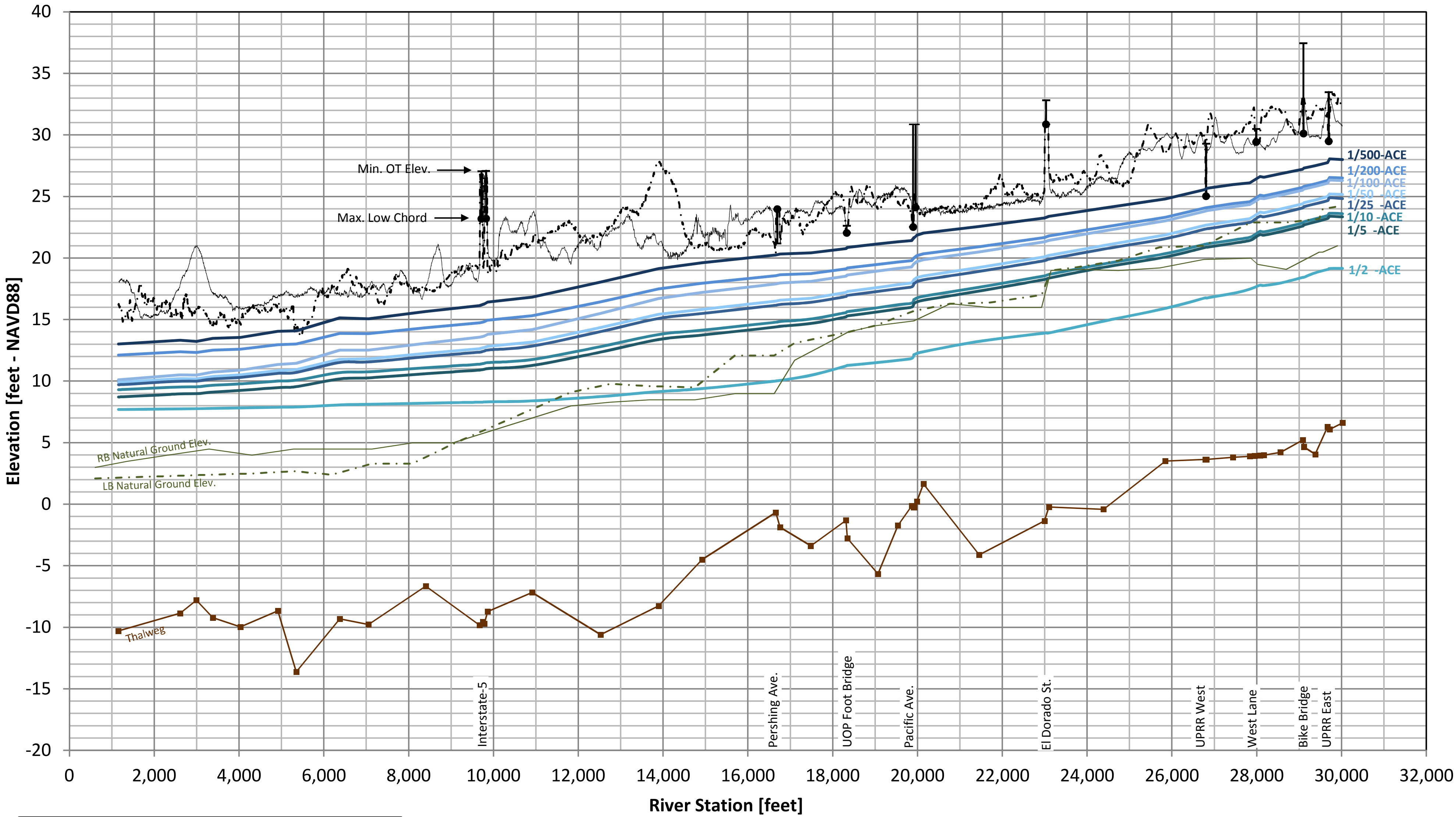
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
French Camp Slough to Old River
2070 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

FIGURE 29B



<u>Peak Flows in Reach</u>

1/2-ACE - 3,850 cfs	1/50-ACE - 12,850 cfs
1/5-ACE - 9,500 cfs	1/100-ACE - 15,360 cfs
1/10-ACE - 9,860 cfs	1/200-ACE - 15,750 cfs
1/25-ACE - 12,280 cfs	1/500-ACE - 19,130 cfs

Top of Levee - RB

Top of Levee - LB

Bridge (Max. Low Chord; Min. OT Elev.)

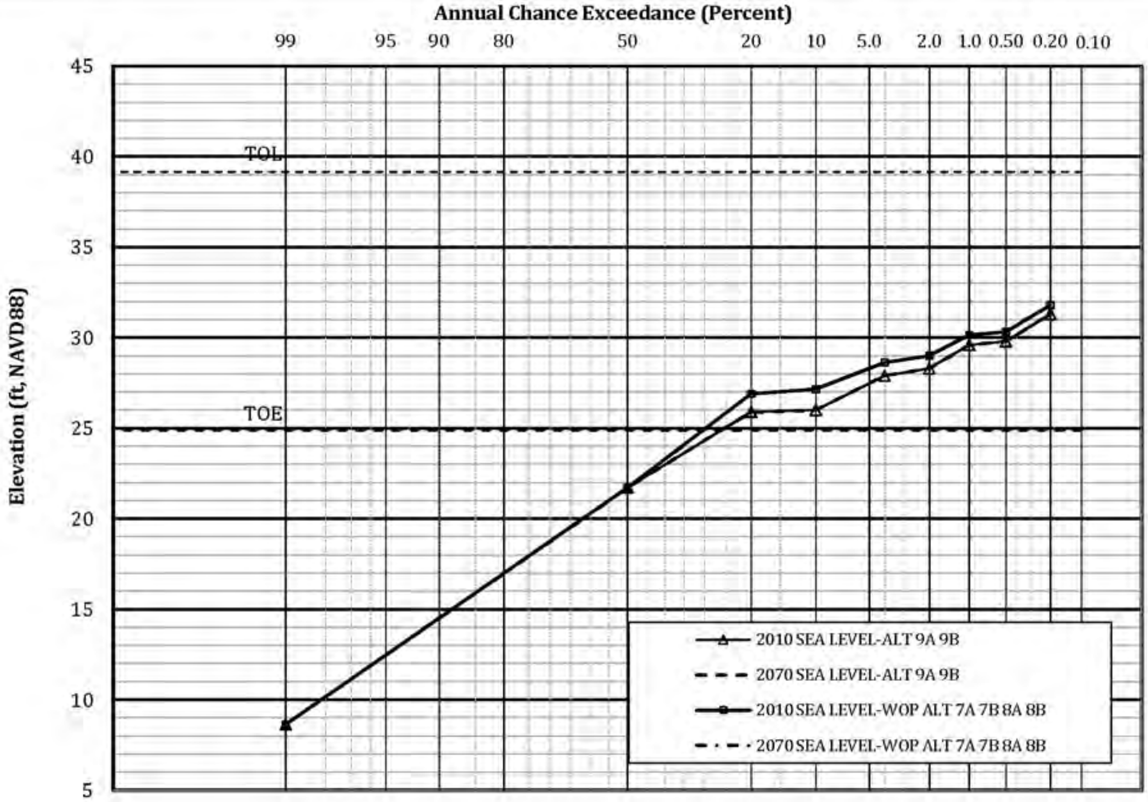
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

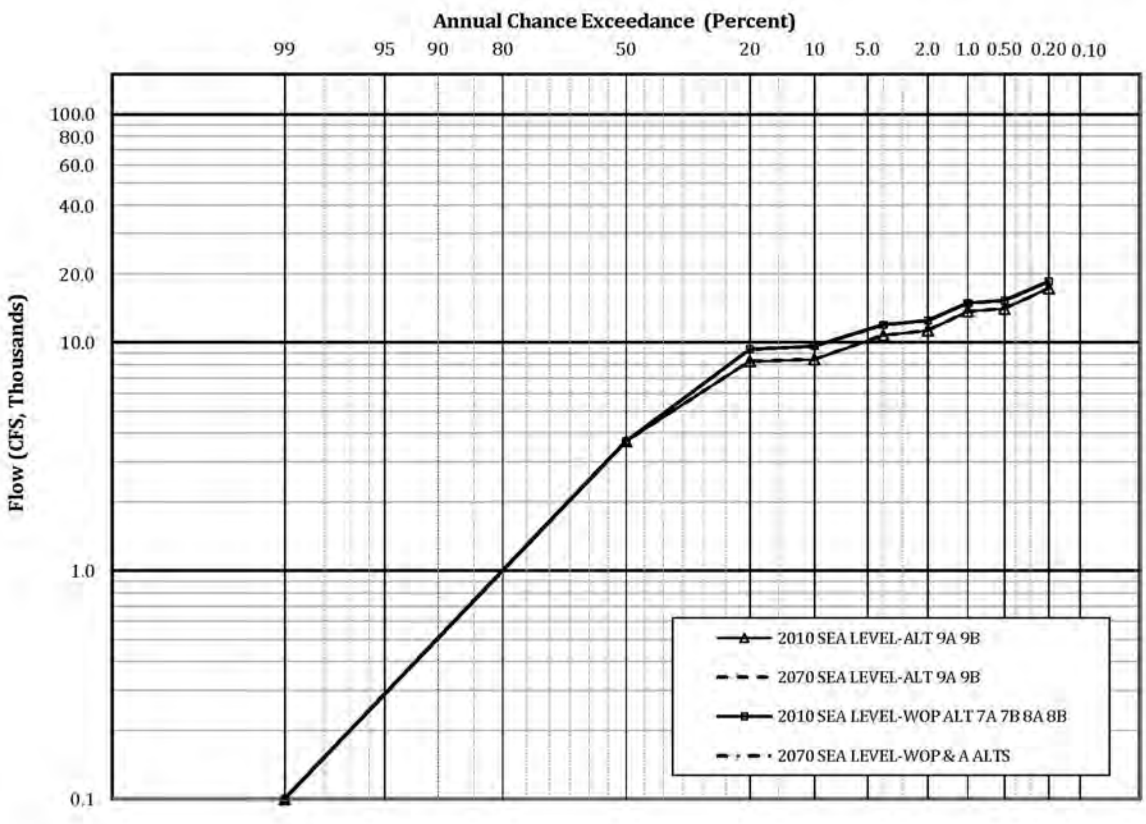
Calaveras River
Downstream of Diverting Canal
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 26





NOTES:

-Stages and Flows for Index Point F-SL1 are from Stockton Diverting Canal at RS 4644

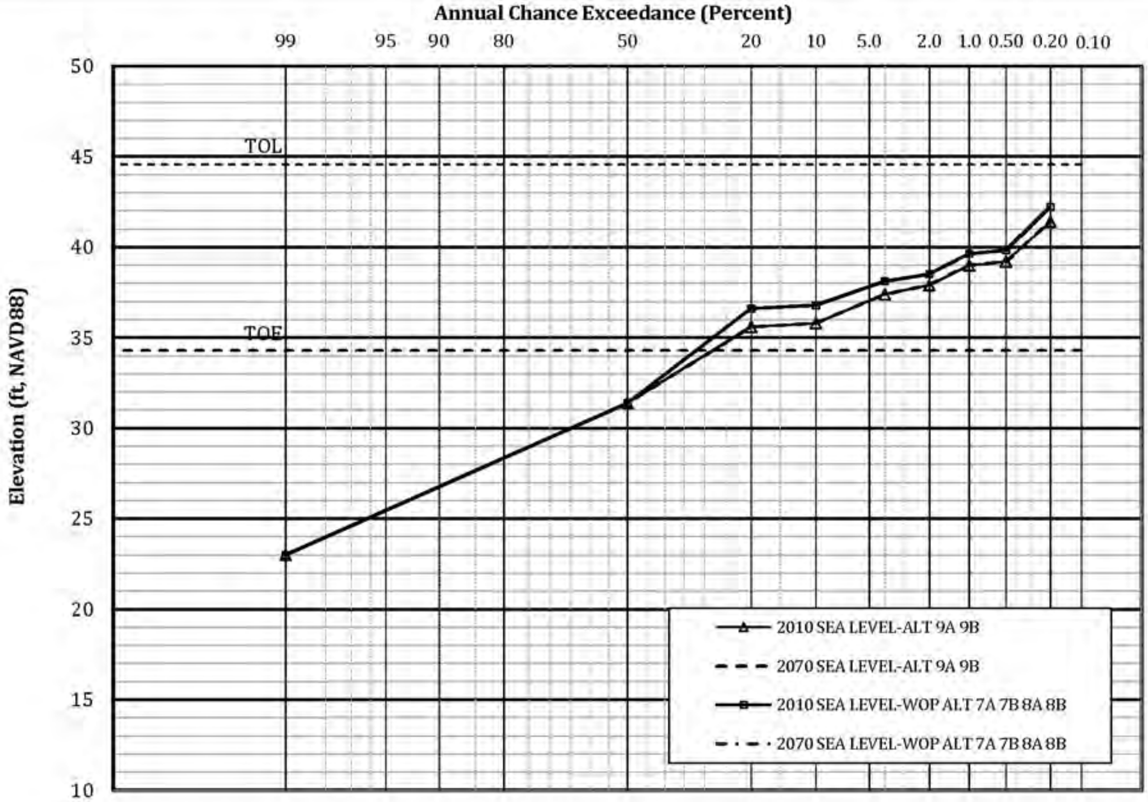
-TOL - Top of levee elevation

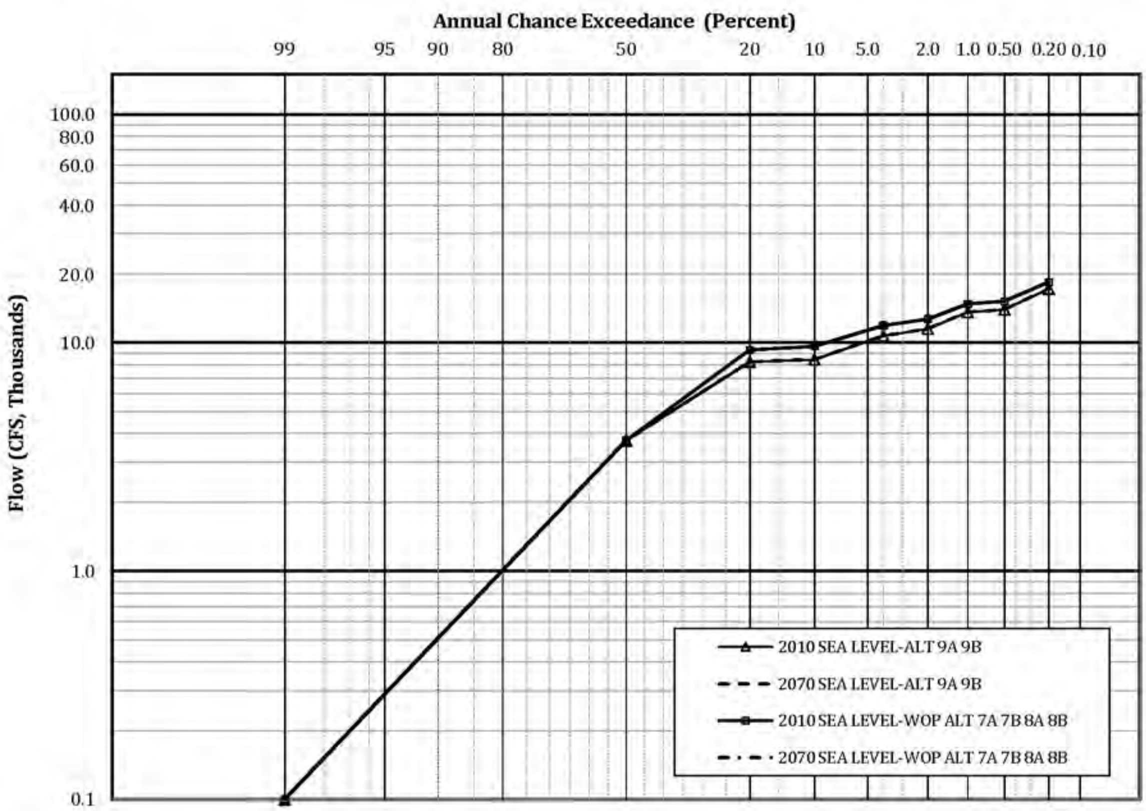
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE FREQUENCY CURVES AT INDEX POINT F-SL1

United States Army Corps of Engineers
Sacramento District





NOTES:

-Stages and Flows for Index Point F-SL2 are from Stockton Diverting Canal at RS 17391

-TOL - Top of levee elevation

-TOE - Approx. elevation of natural floodplain adjacent to levee

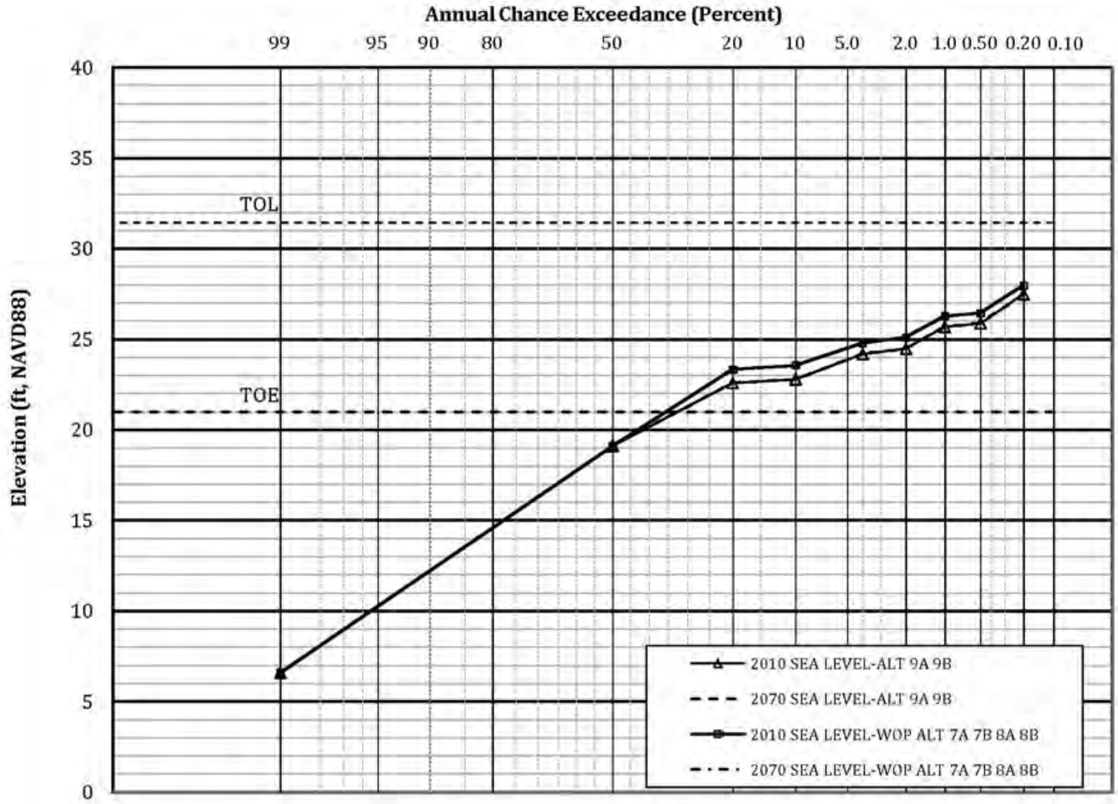
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

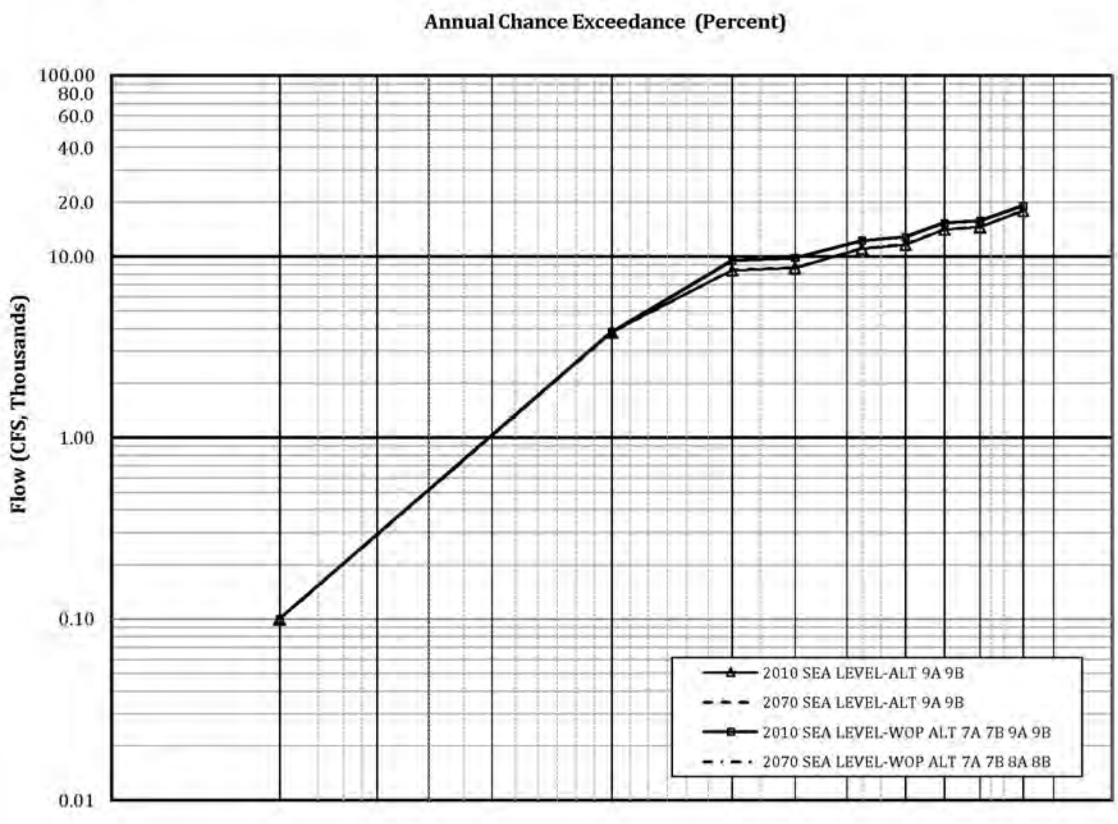
STAGE AND DISCHARGE FREQUENCY CURVES AT INDEX POINT F-SL2

United States Army Corps of Engineers
Sacramento District

DEC 2015

FIGURE 31B





NOTES:

Curves based on HEC-RAS simulations

Stages amd Flows based on Calaveras River at RS 30024.

Top of Levee = TOL
Toe of Levee = Approximate elevation of ground adjacent to levee.

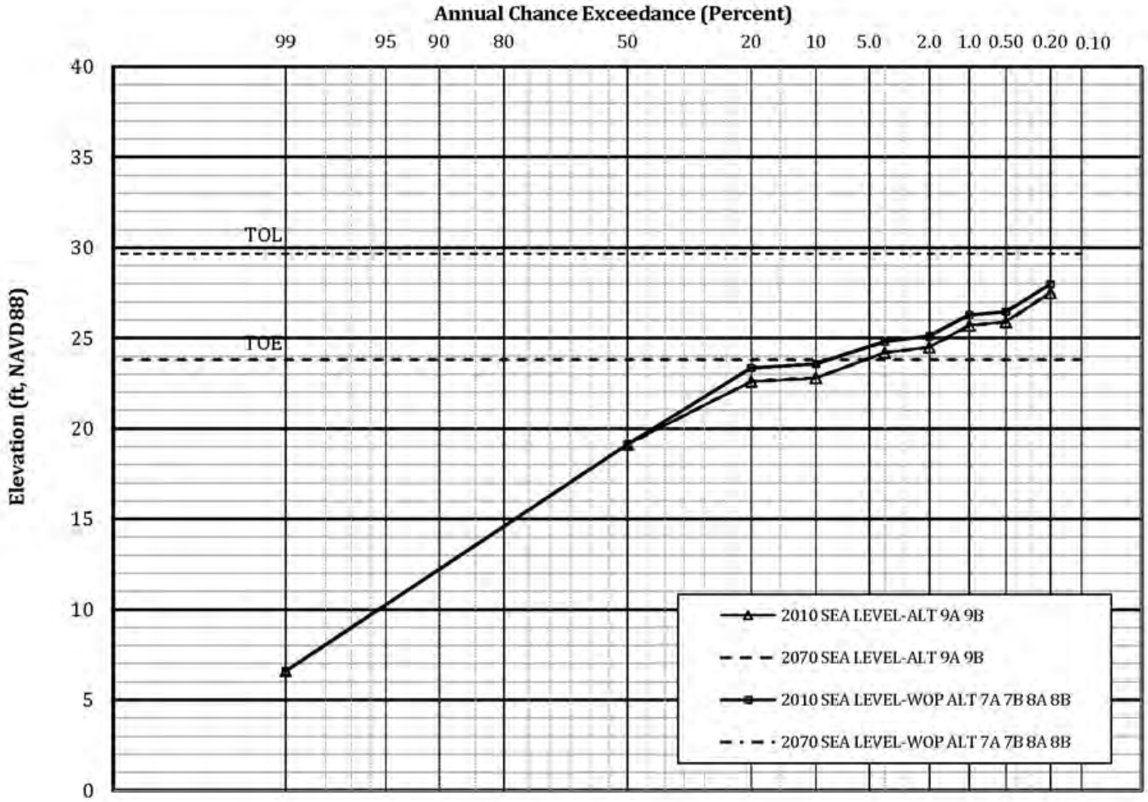
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

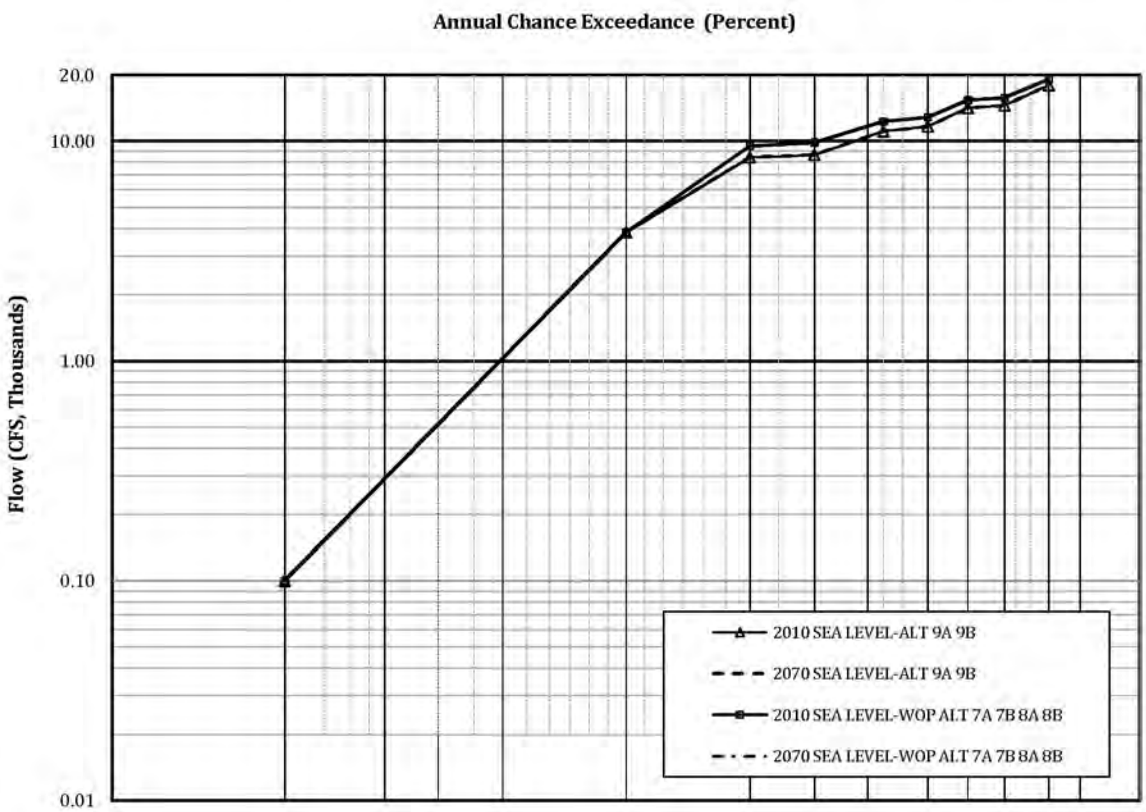
STAGE AND DISCHARGE
FREQUENCY CURVES
INDEX POINT F-CL2

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

DEC 2015

FIGURE 31C





NOTES:

-Stages and Flows for Index Point CR2 are from the Calaveras River at RS 30639

-TOL - Top of levee elevation

-TOE - Approx. elevation of natural floodplain adjacent to levee

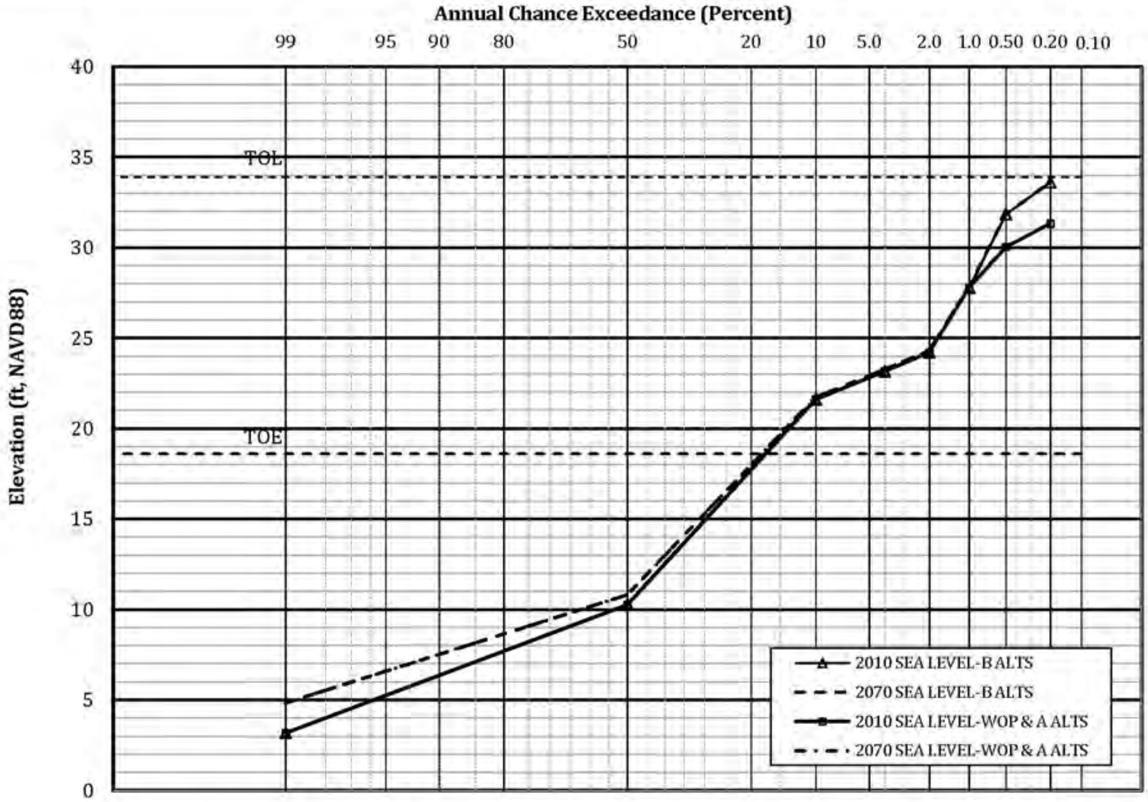
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

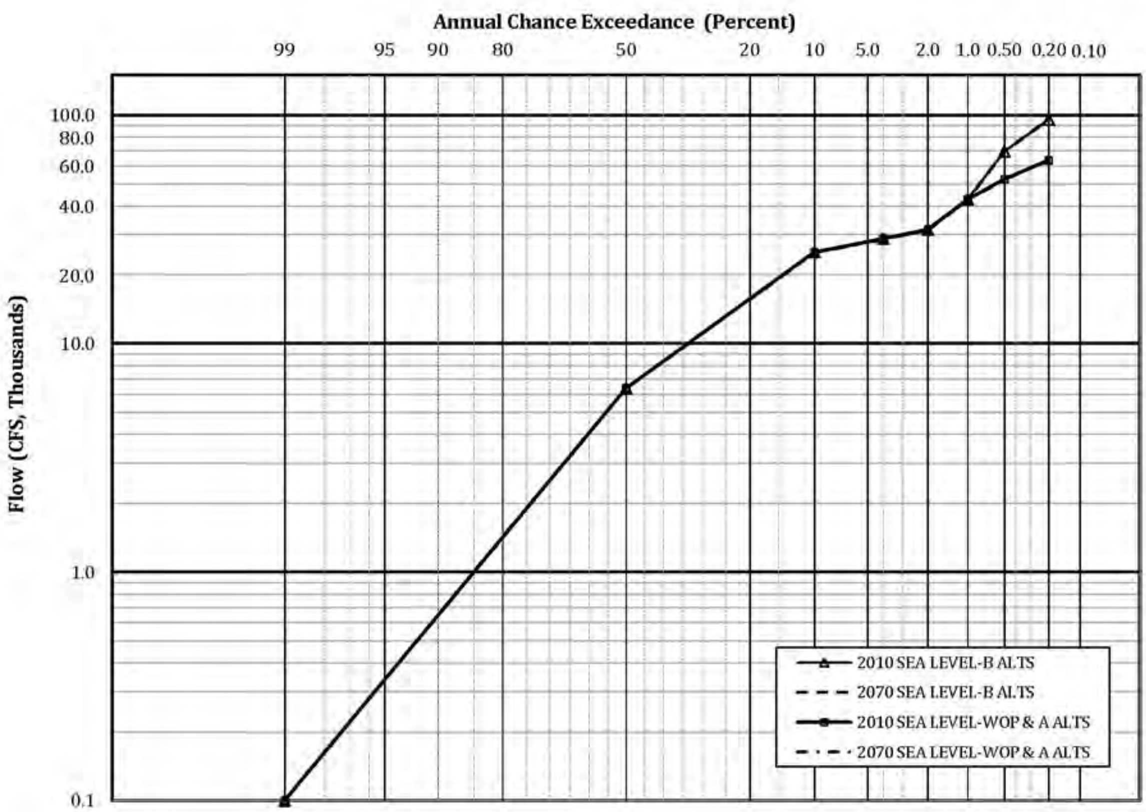
STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-CR2

United States Army Corps of Engineers
Sacramento District

DEC 2015

FIGURE 31D





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Index Point F-LR4 located on San Joaquin River at RS 57.05

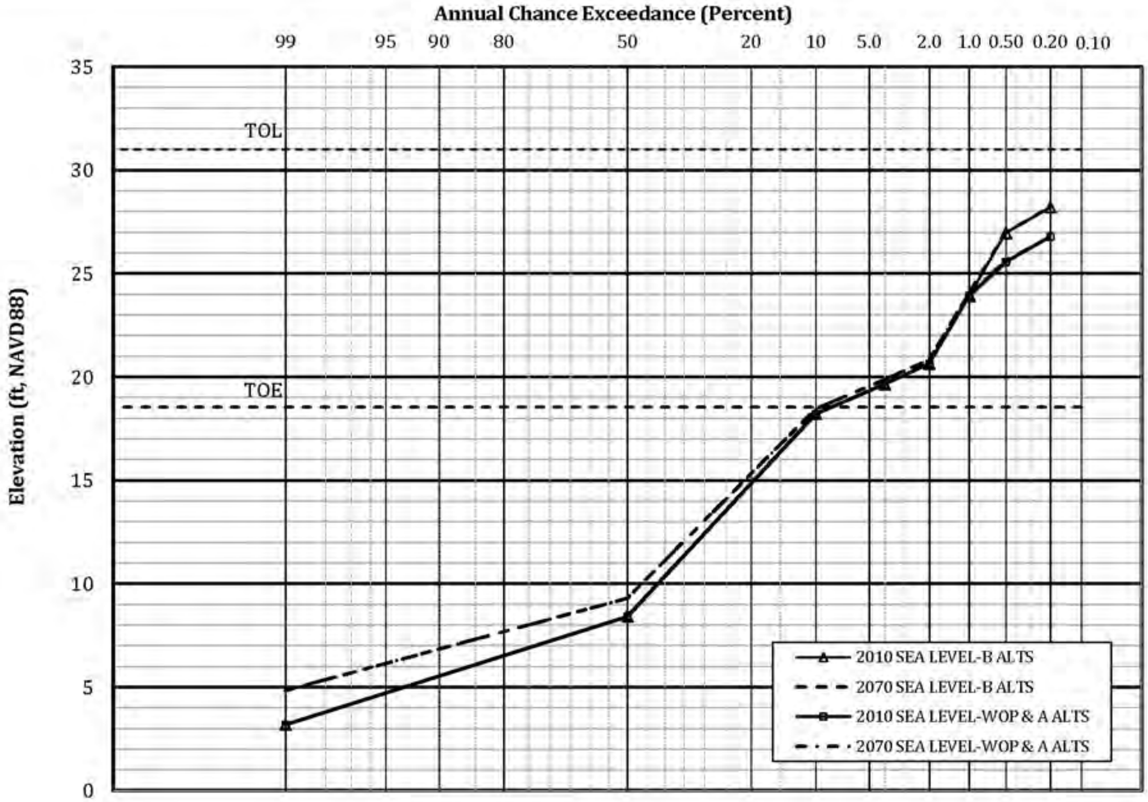
-TOL - Top of levee elevation

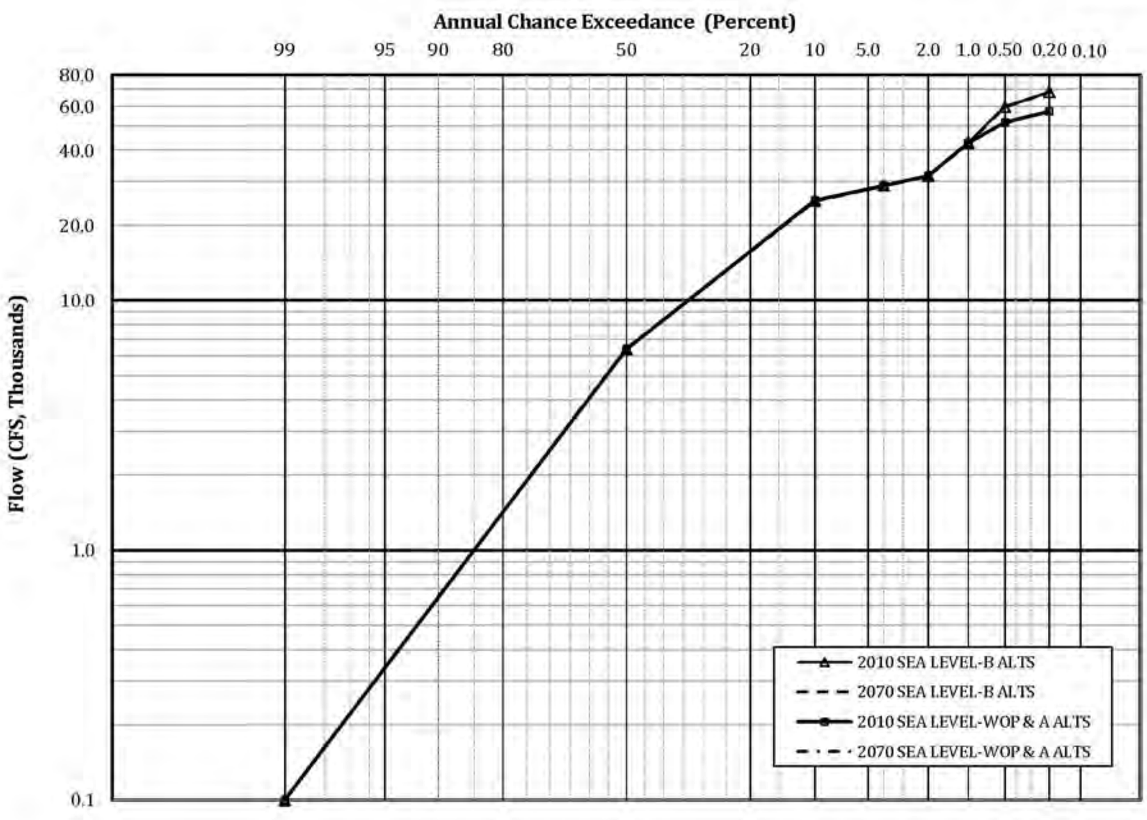
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-LR4

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Index Point F-LR3 located on San Joaquin River at RS 53.89

-TOL - Top of levee elevation

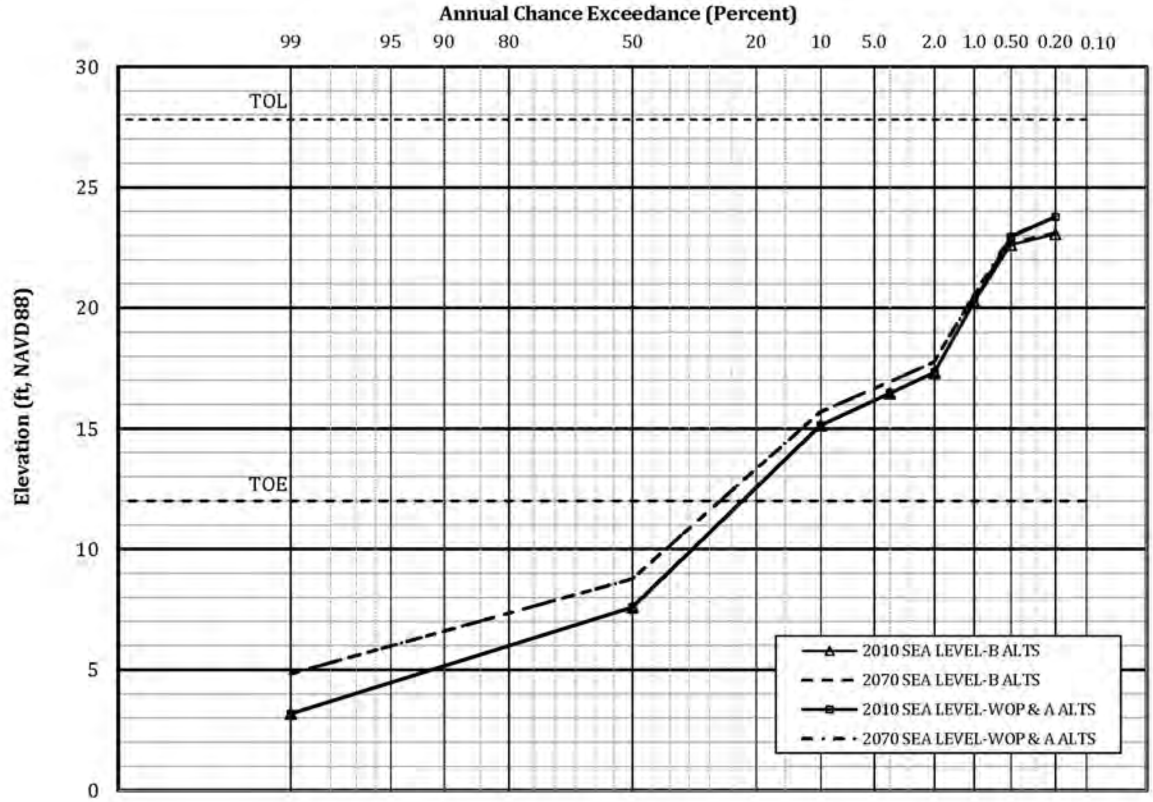
-TOE - Approx. elevation of natural floodplain adjacent to levee

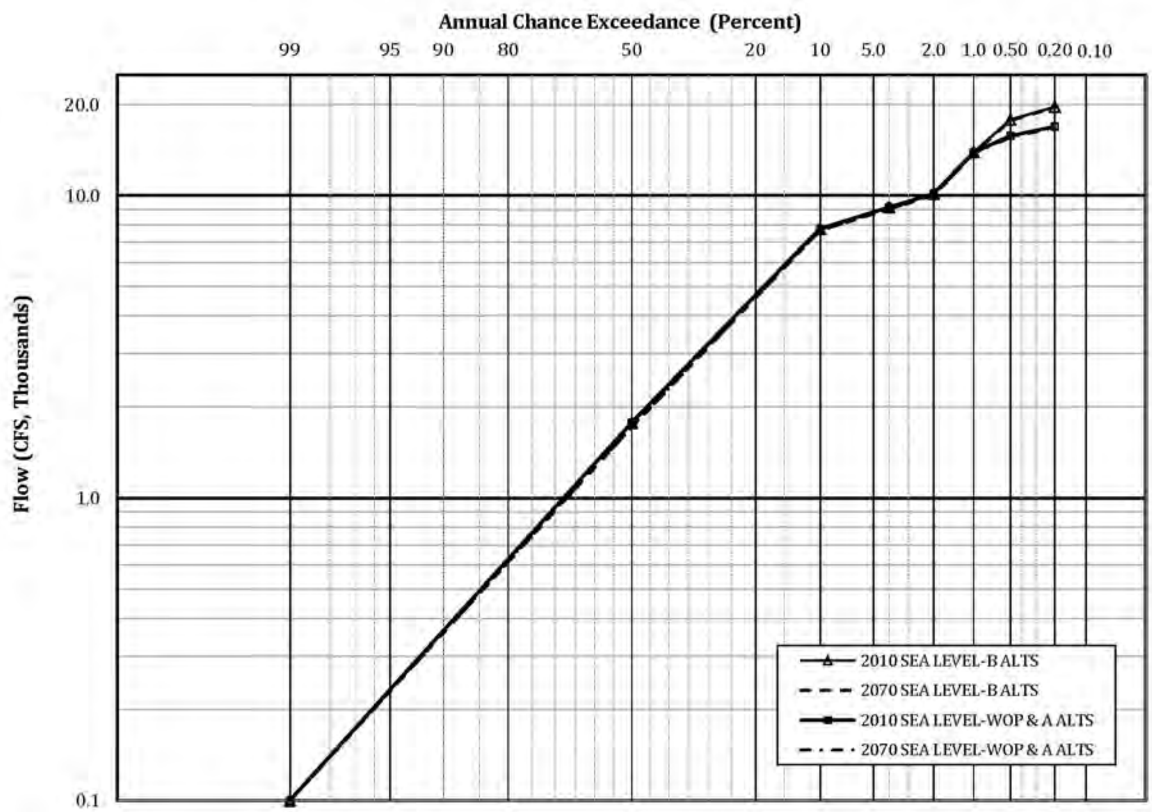
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-LR3

United States Army Corps of Engineers
Sacramento District

DEC 2015





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Index Point F-LR2 located on San Joaquin River at RS 48.89

-TOL - Top of levee elevation

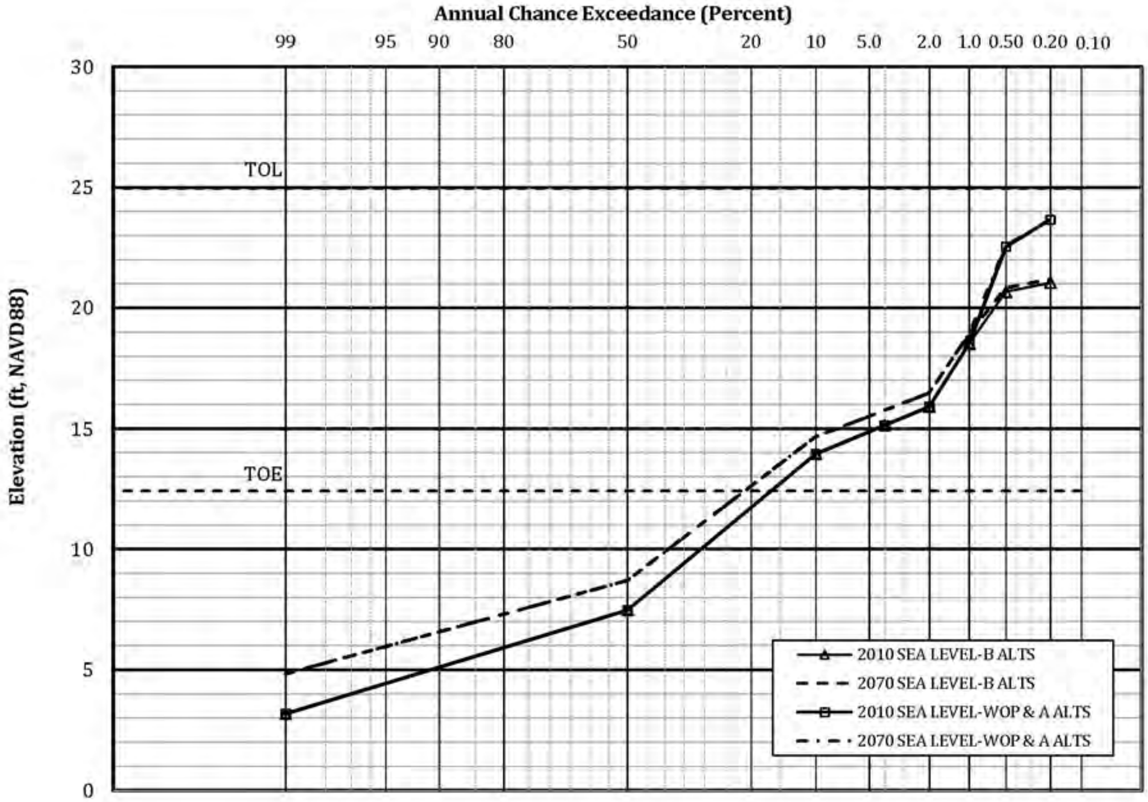
-TOE - Approx. elevation of natural floodplain adjacent to levee

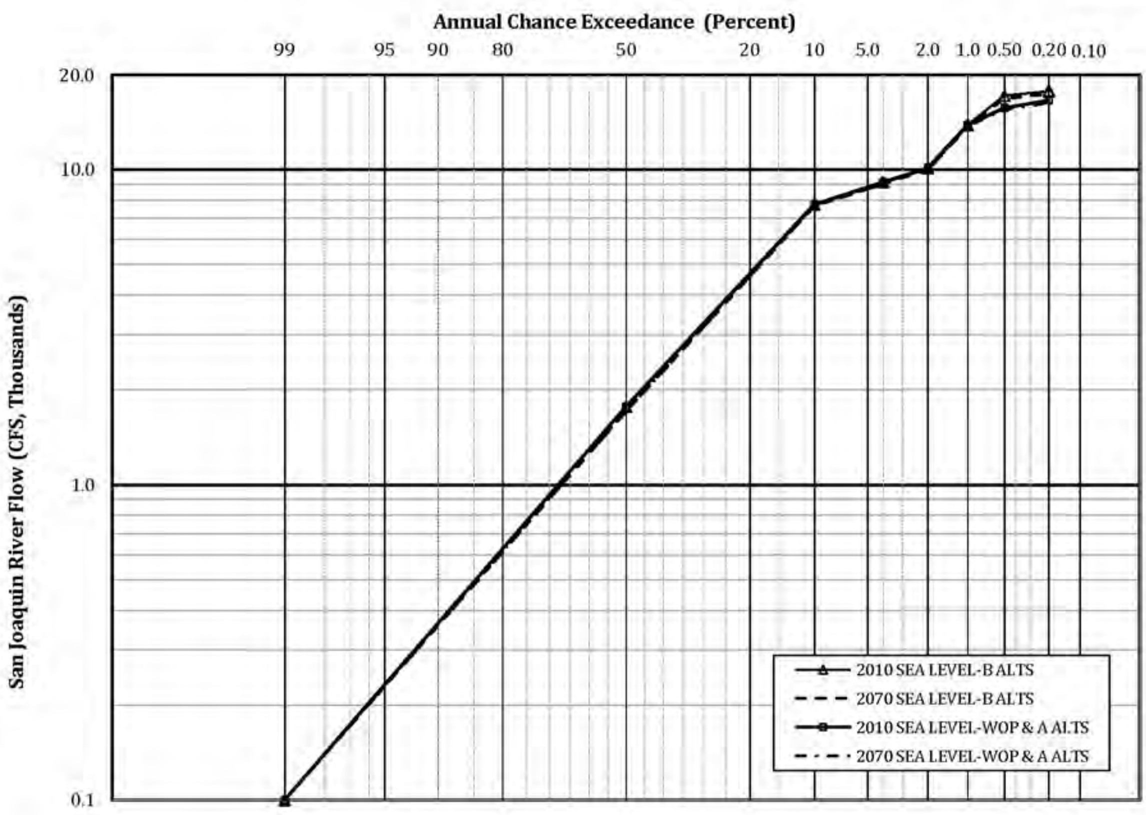
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE FREQUENCY CURVES AT INDEX POINT F-LR2

United States Army Corps of Engineers
Sacramento District

DEC 2015





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Index Point F-LR1 based on conditions on San Joaquin River at RS 46.61

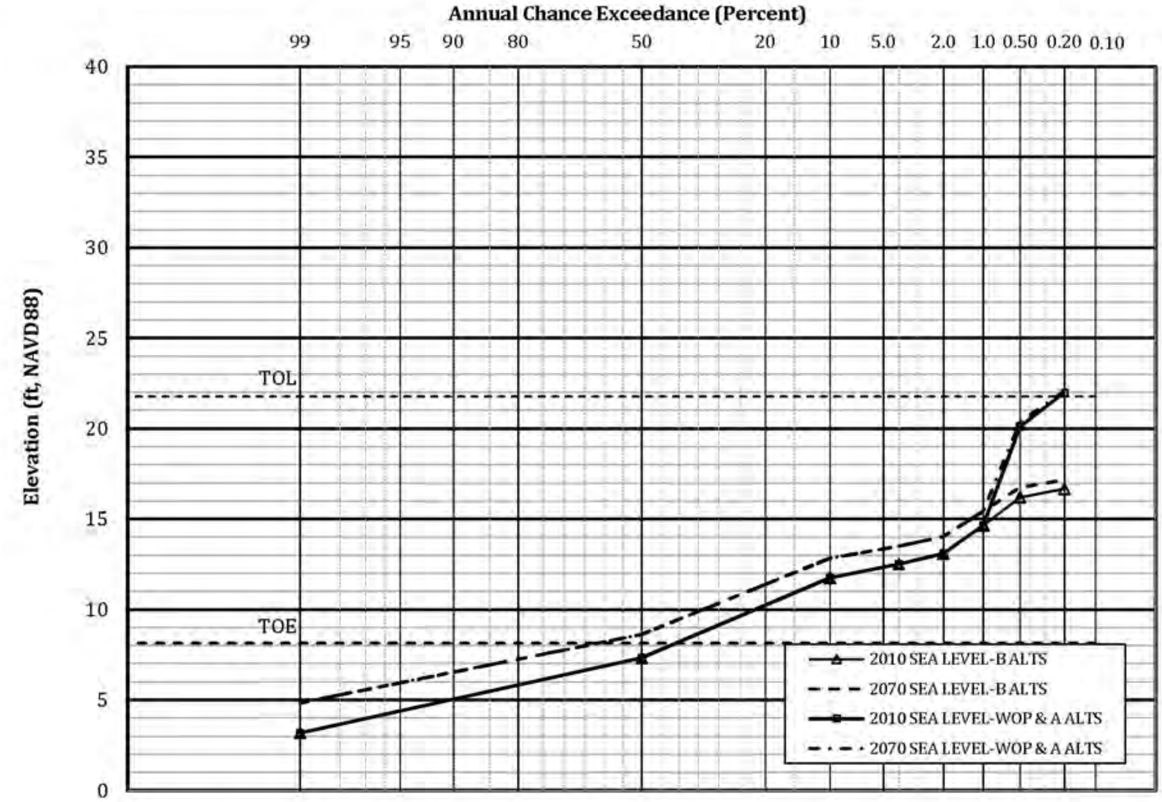
-TOL - Top of levee elevation

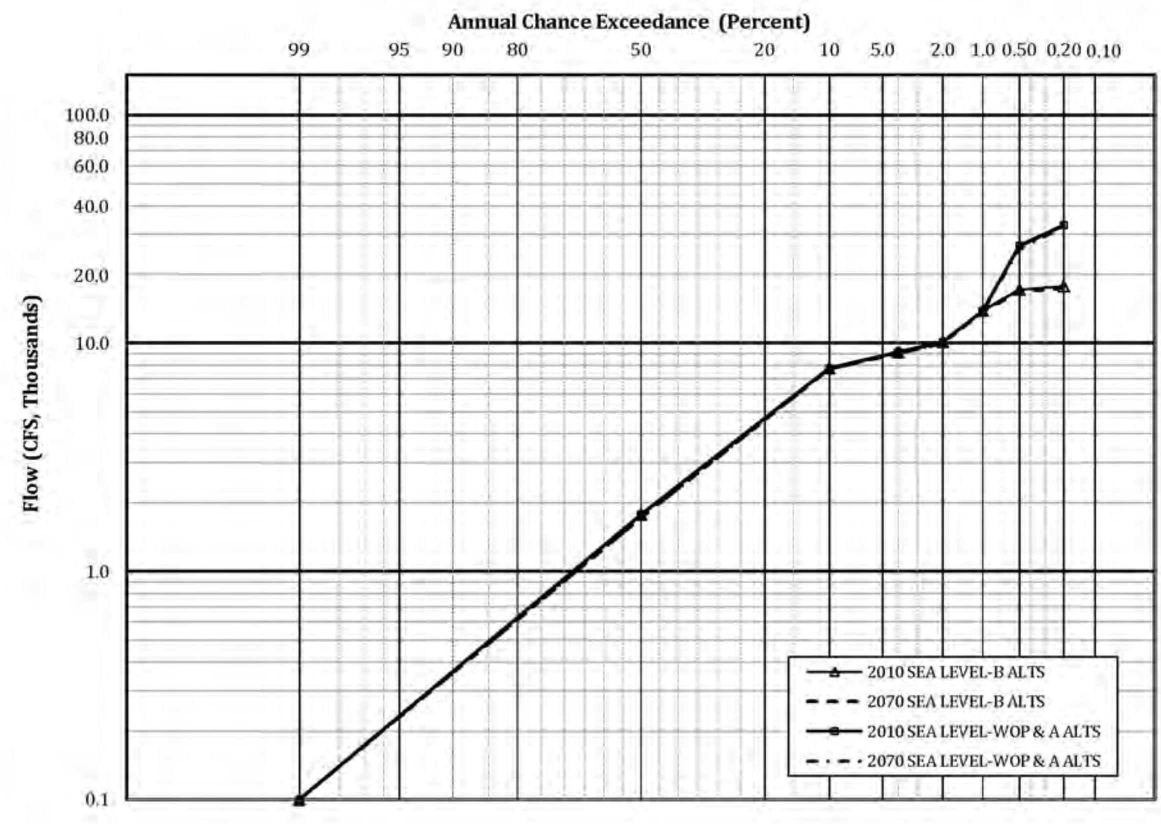
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-LR1

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Stages and Flows for Index Point FL1 are from San Joaquin River at RS 43.1

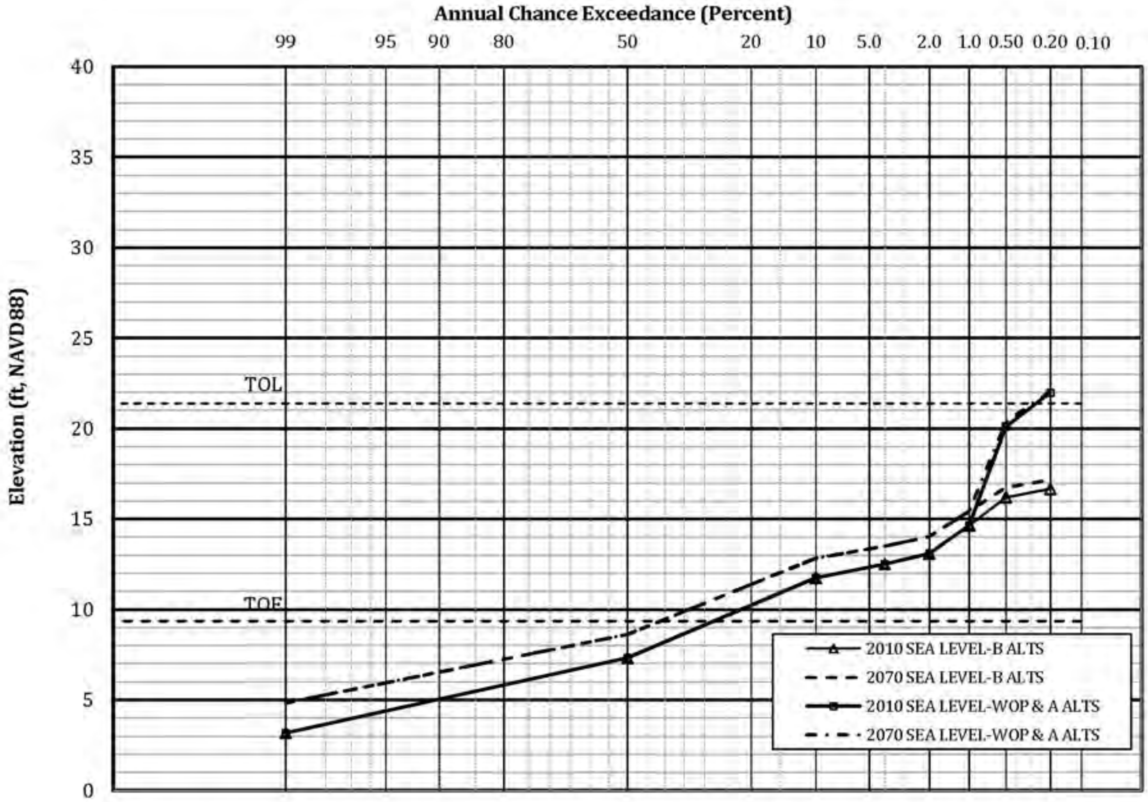
-TOL - Top of levee elevation

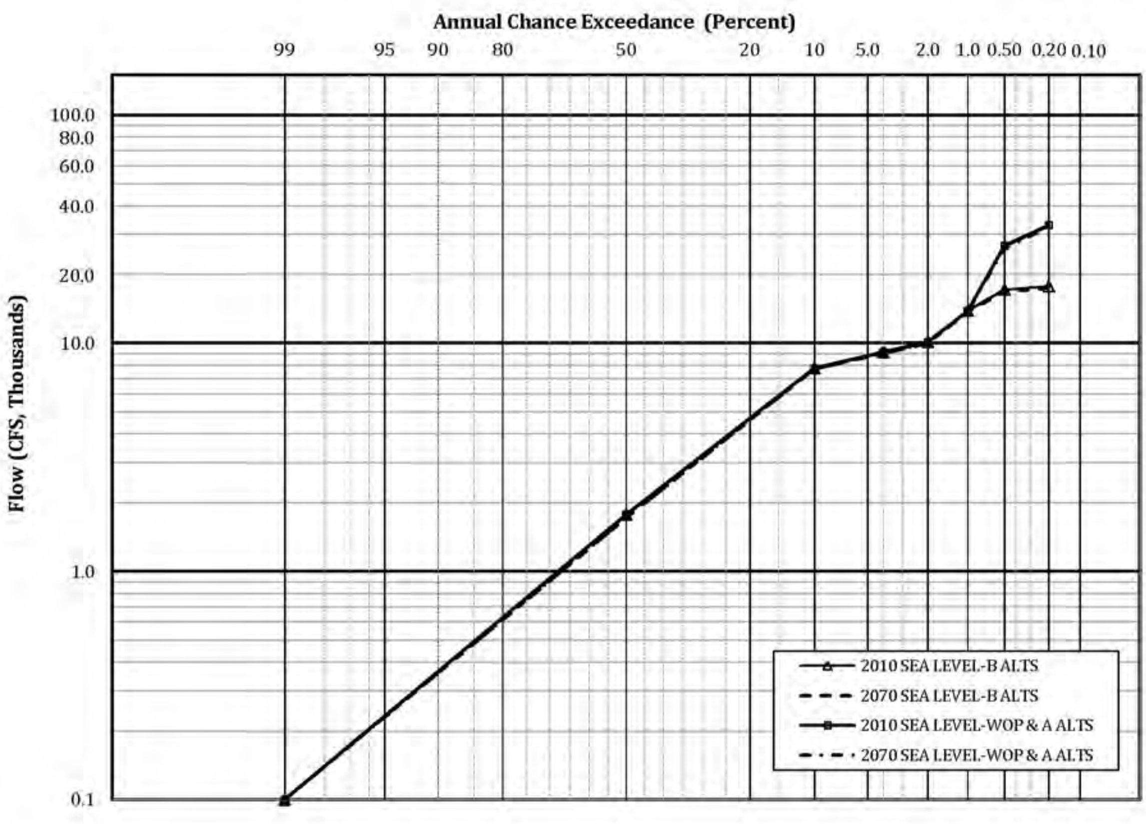
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
FR1

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Stages and Flows for Index Point FL1 are from San Joaquin River at RS 43.1

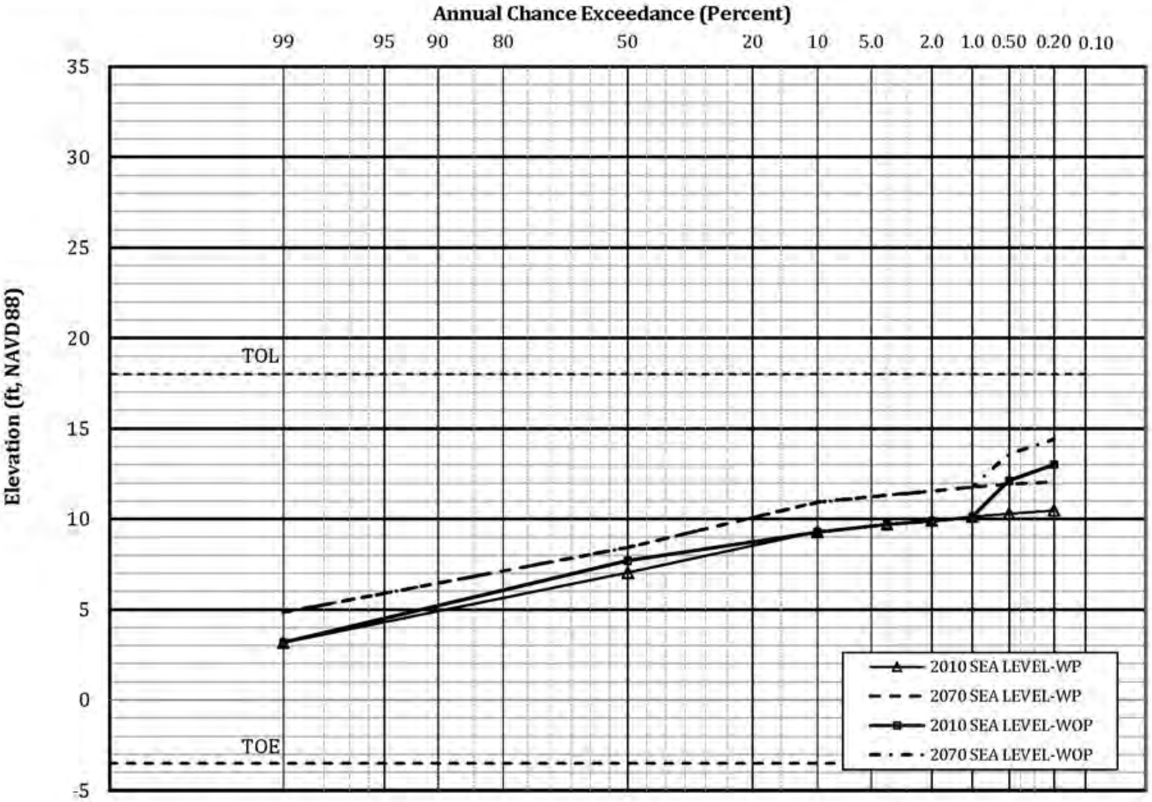
-TOL - Top of levee elevation

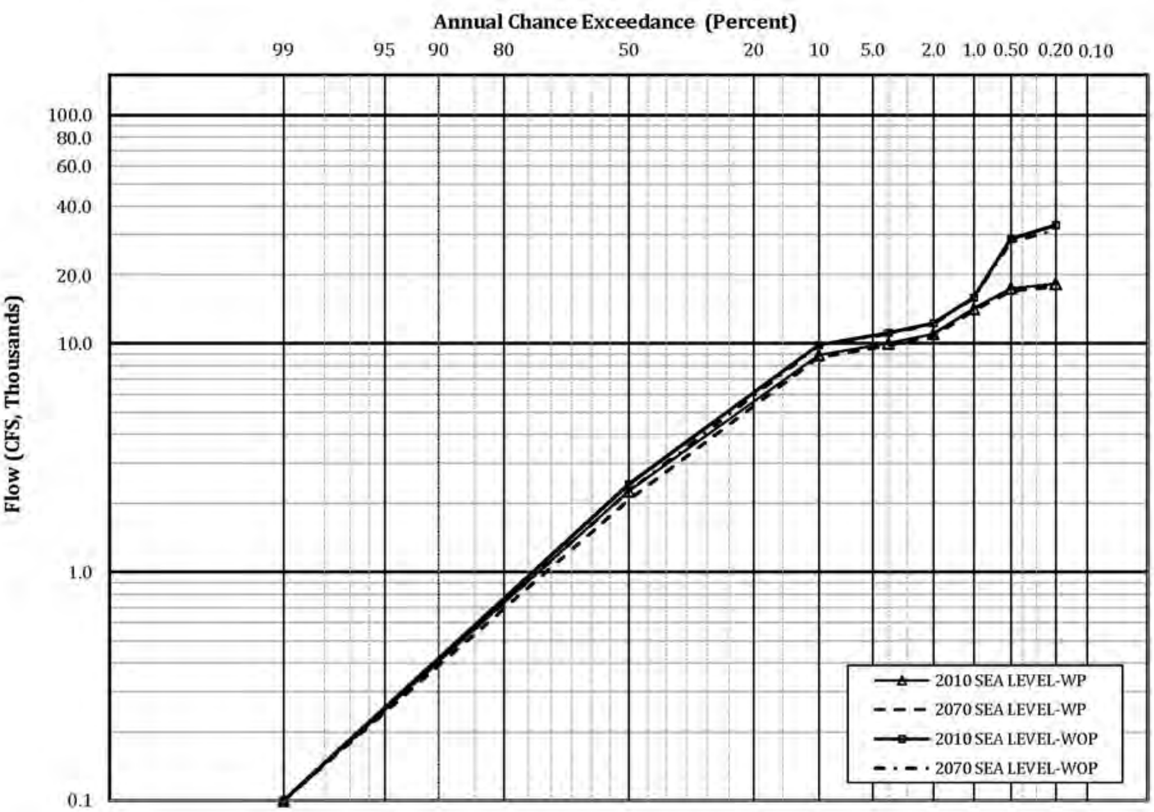
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
FL1

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative
With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Index Point is located on the Ship Channel at RS 37.83

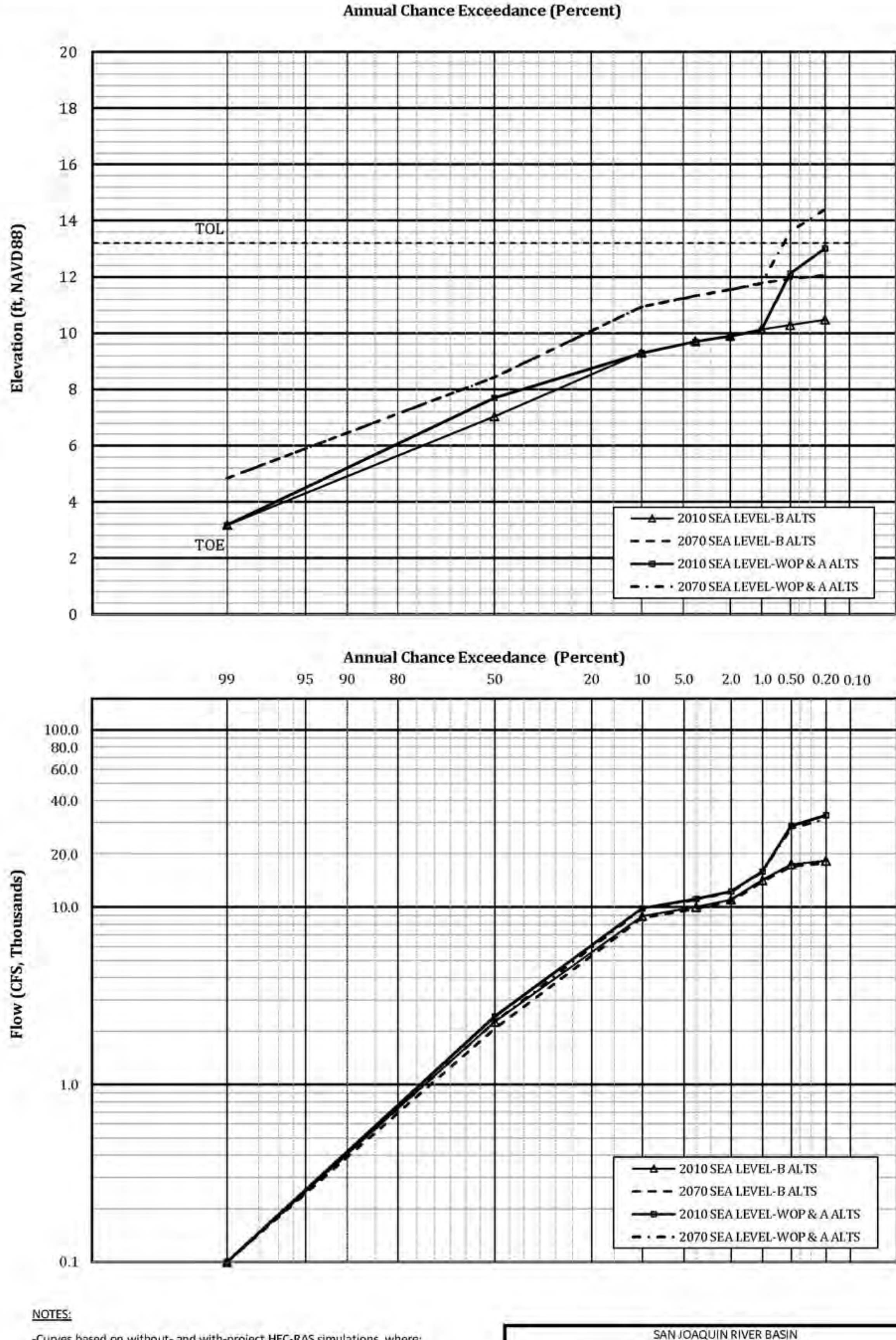
-TOE - Approx. elevation of natural floodplain adjacent to left bank levee levee toe estimated where land flattens closest to levee within Rough and Ready Island in line with HEC-RAS cross section

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-D-BS

United States Army Corps of Engineers
Sacramento District

JUN 2014



NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative

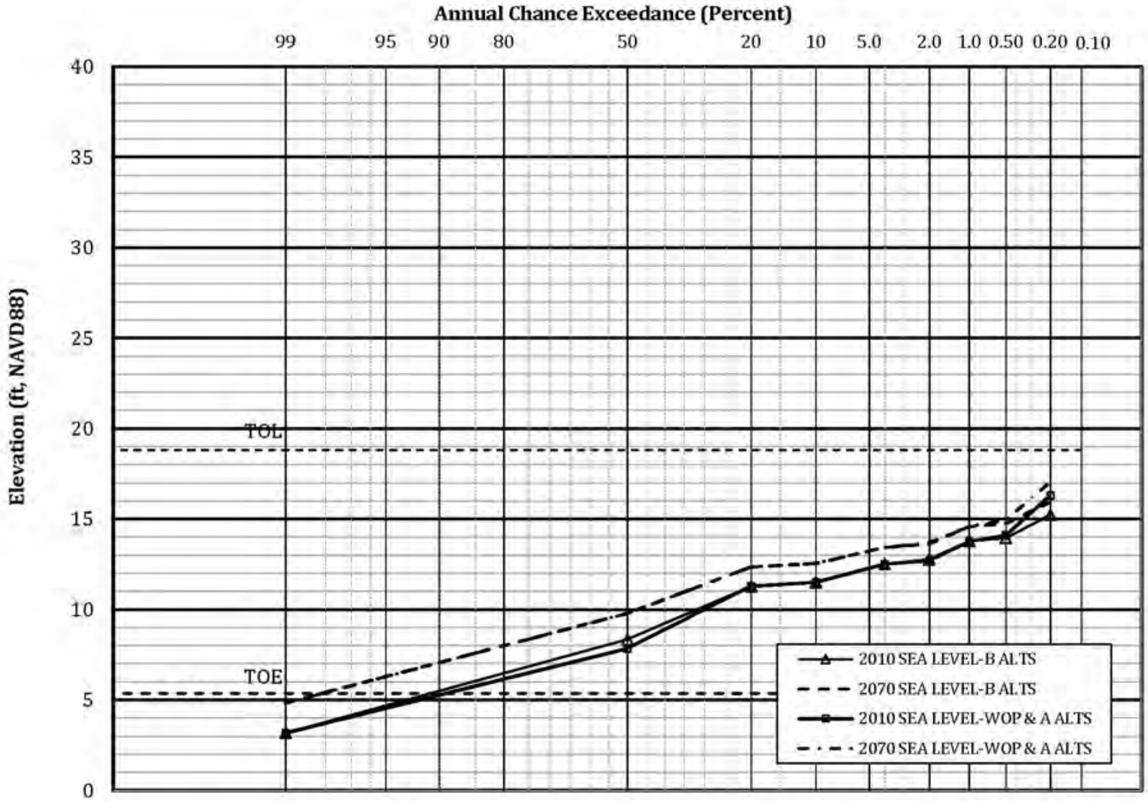
-Stage and Flow data is from the Ship Channel at RS 37.83

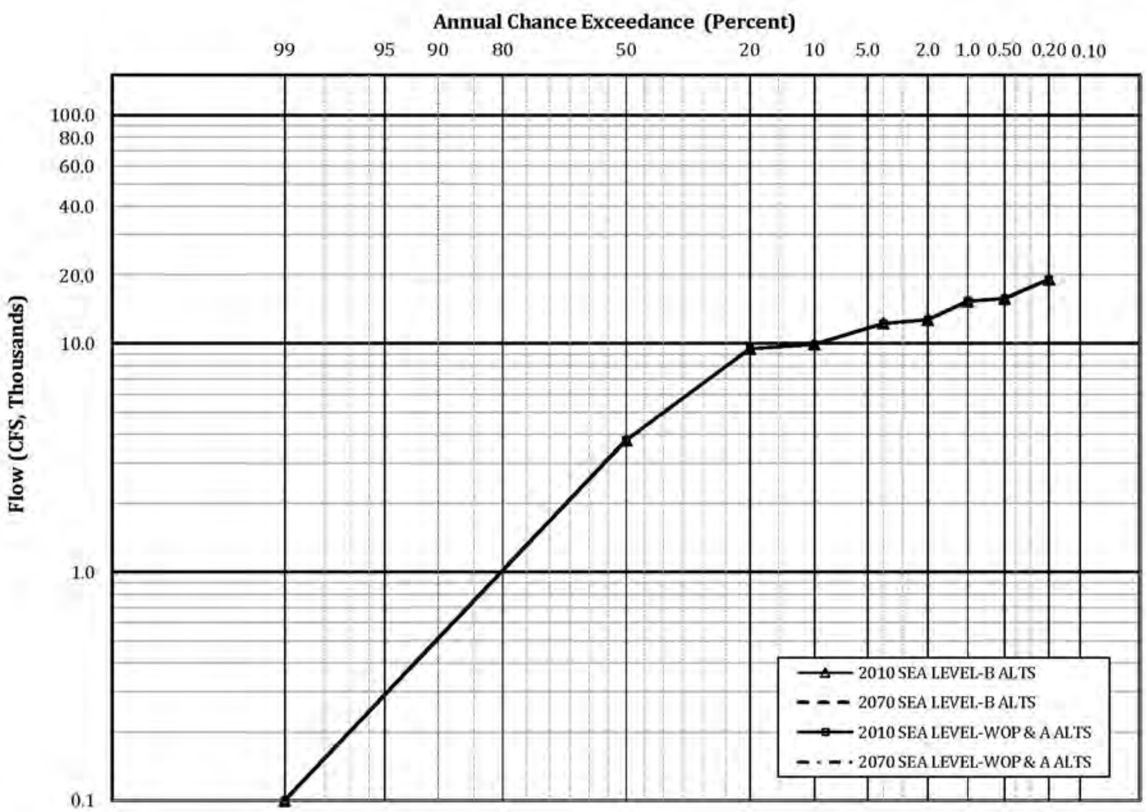
-TOE - Approx. elevation of natural floodplain adjacent to left bank levee levee toe estimated where land flattens closest to levee within Rough and Ready Island in line with HEC-RAS cross section

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-D3

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative

-Stages and Flows for Index Point F-D4 are from the Calaveras River at RS 9862

-TOL - Top of levee elevation

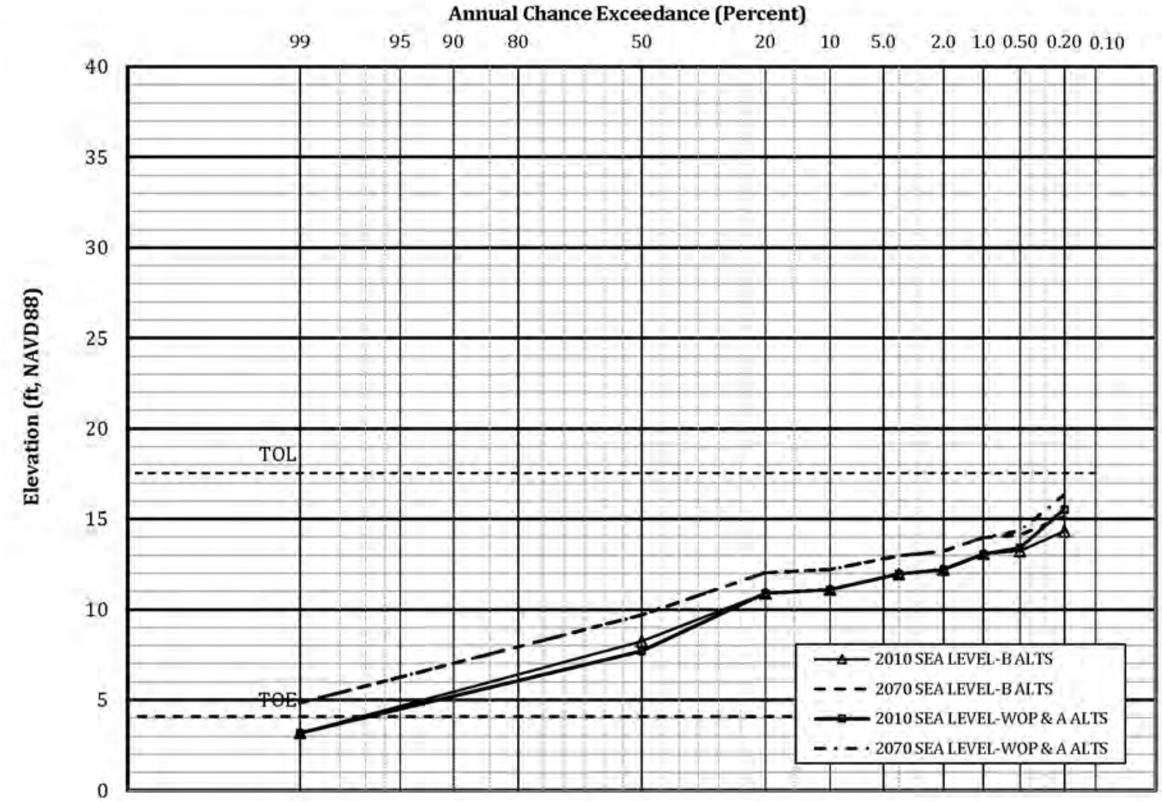
-TOE - Approx. elevation of natural floodplain adjacent to levee

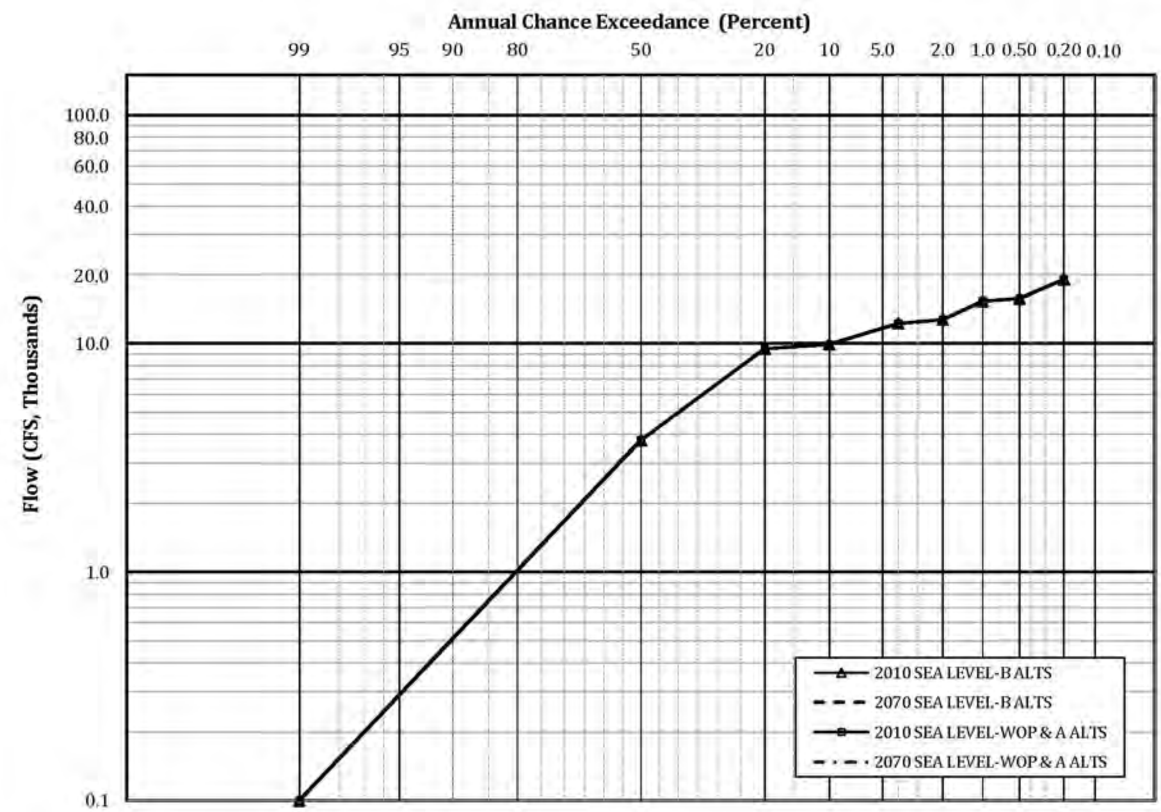
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-D4

United States Army Corps of Engineers
Sacramento District

DEC 2015





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Stages and Flows for Index Point F-D5 are from the Calaveras River at RS 8401

-TOL - Top of levee elevation

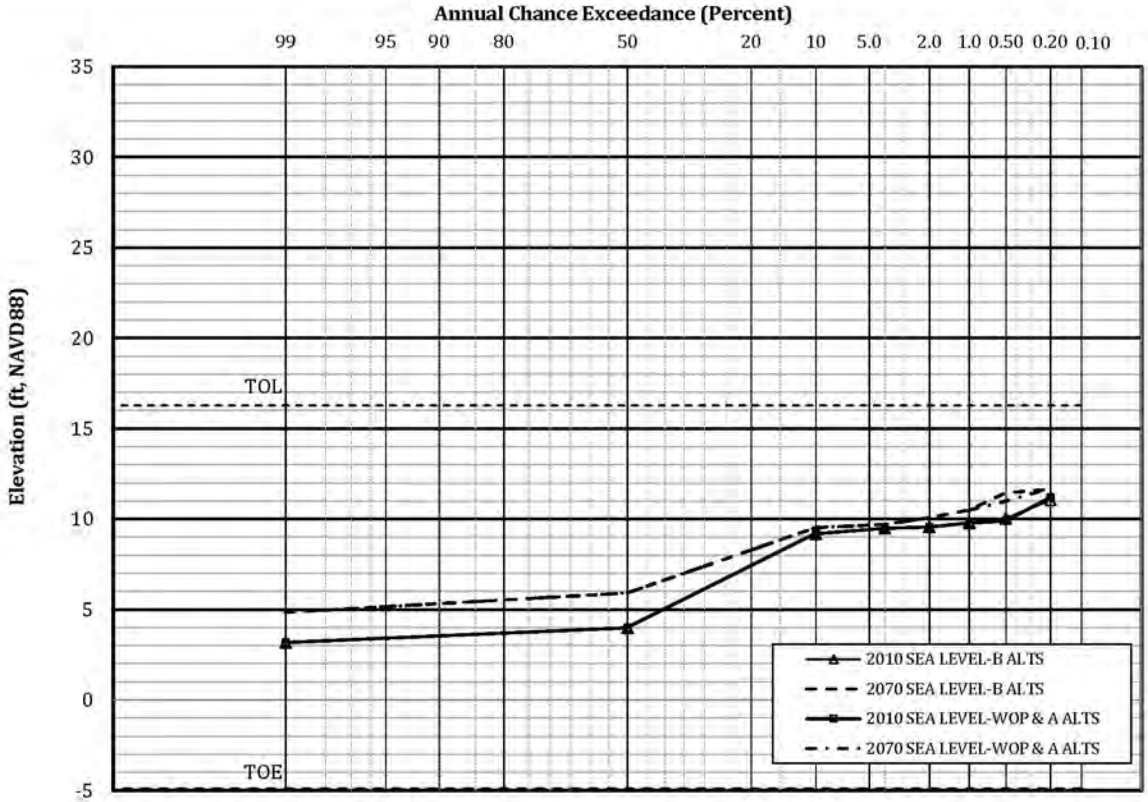
-TOE - Approx. elevation of natural floodplain adjacent to levee

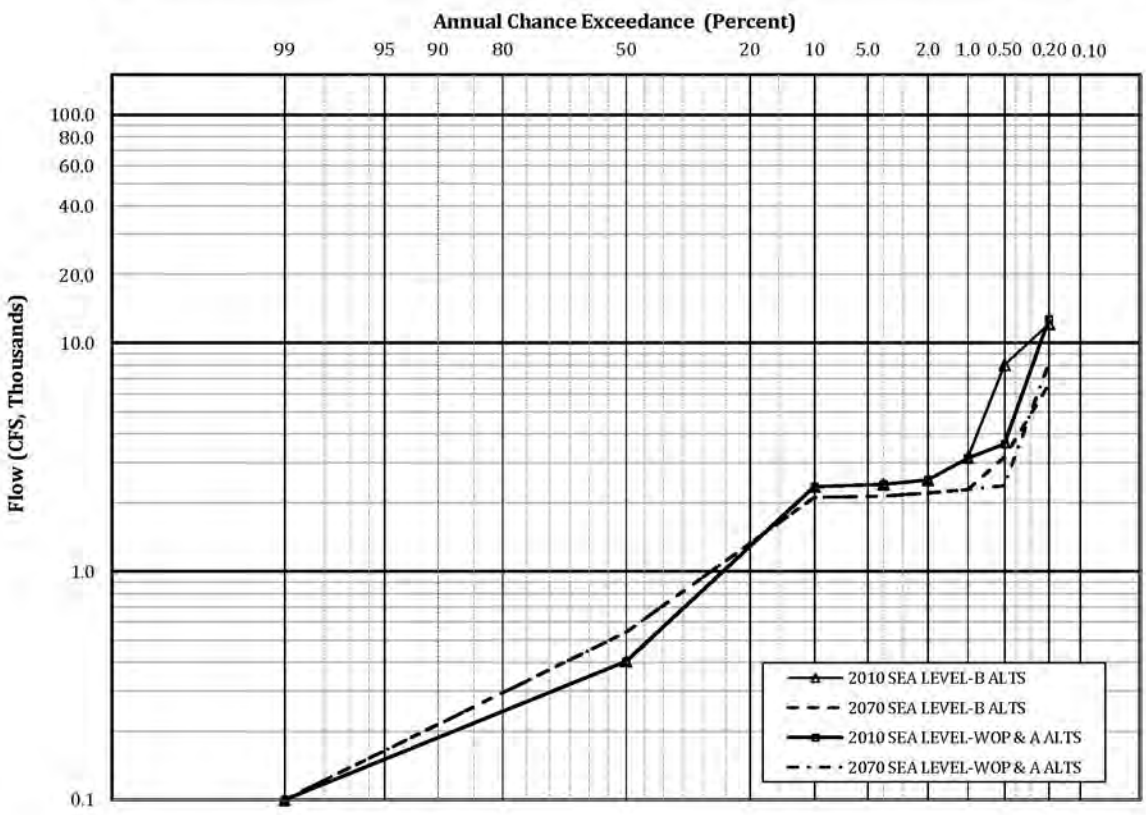
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-D5

United States Army Corps of Engineers
Sacramento District

DEC 2015





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Stages and Flows for Index Point Middle River @ Borden Hwy are from Middle River at RS 15.923

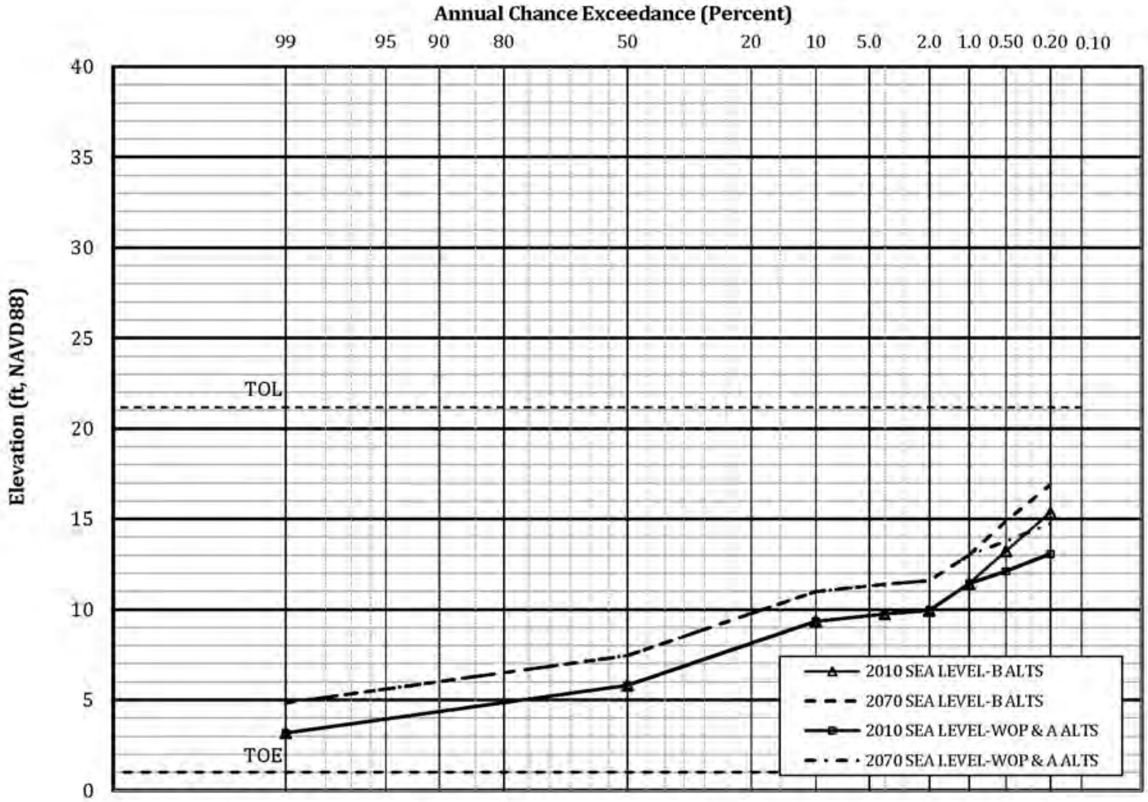
-TOL - Top of levee elevation

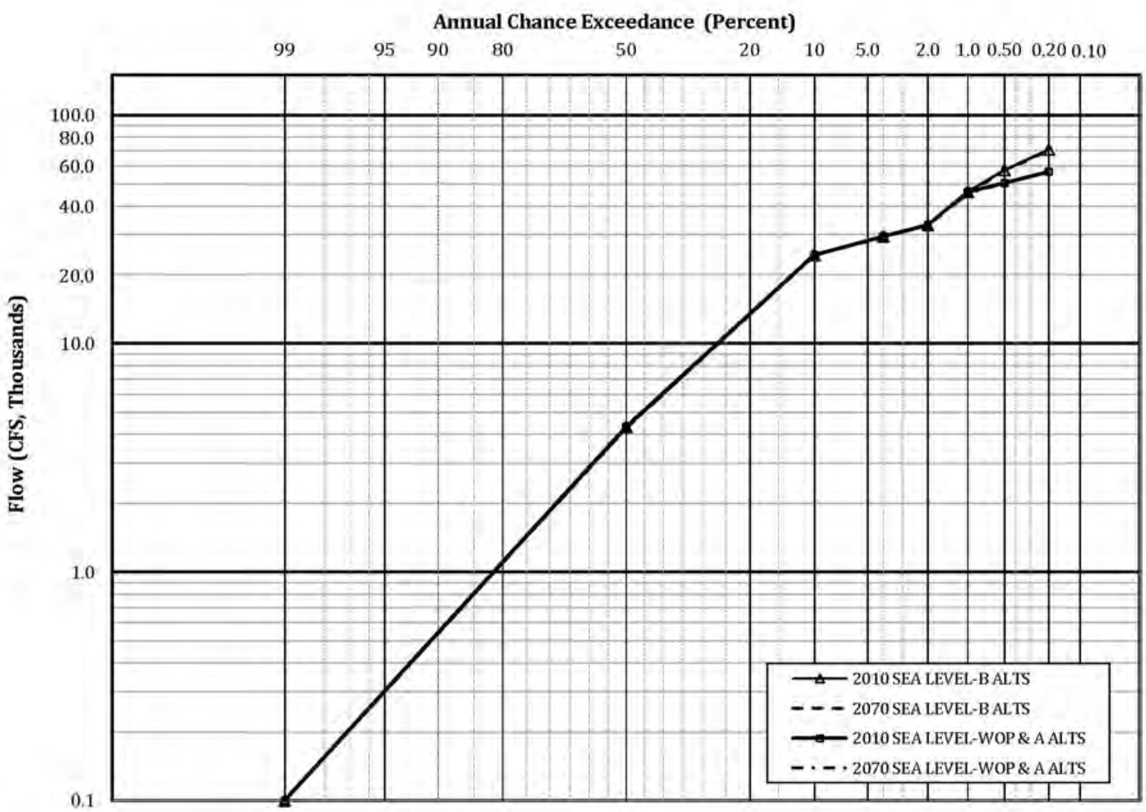
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
MIDDLE RIVER AT BORDEN HWY

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Stages and Flows for Index Point Old River @ Clifton Court are from Old River at RS 20.092

-TOL - Top of levee elevation

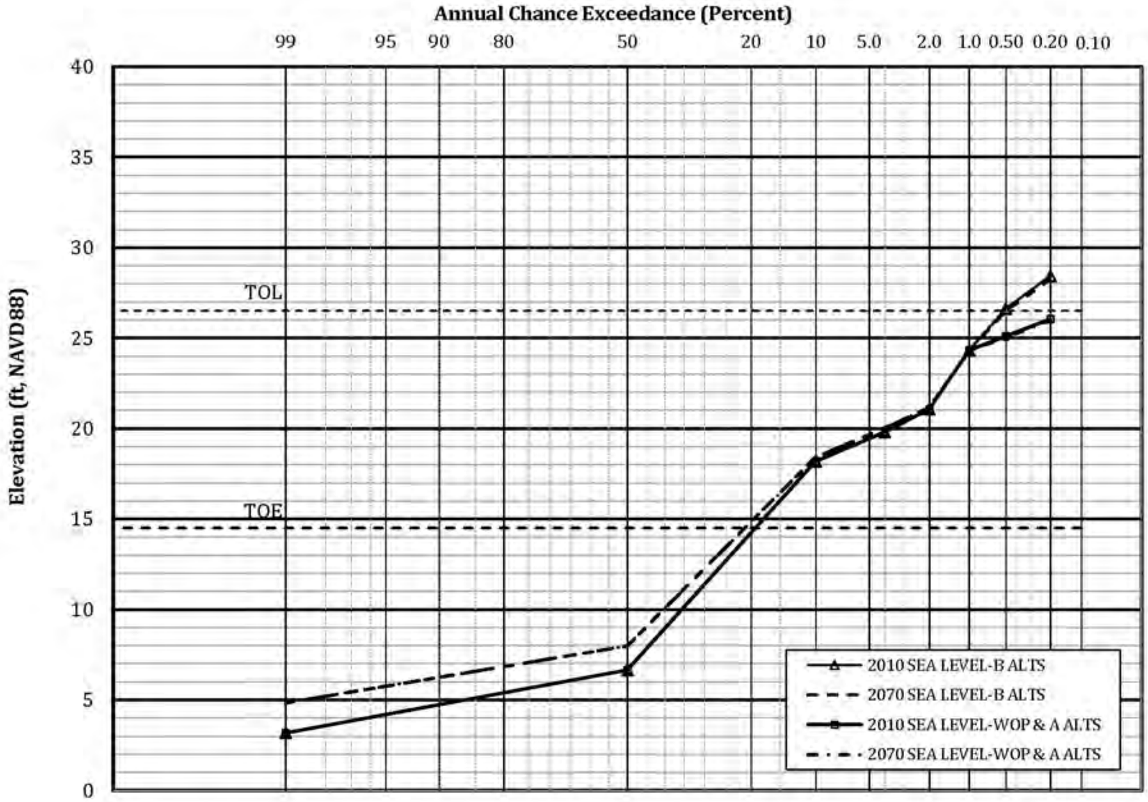
-TOE - Approx. elevation of natural floodplain adjacent to levee

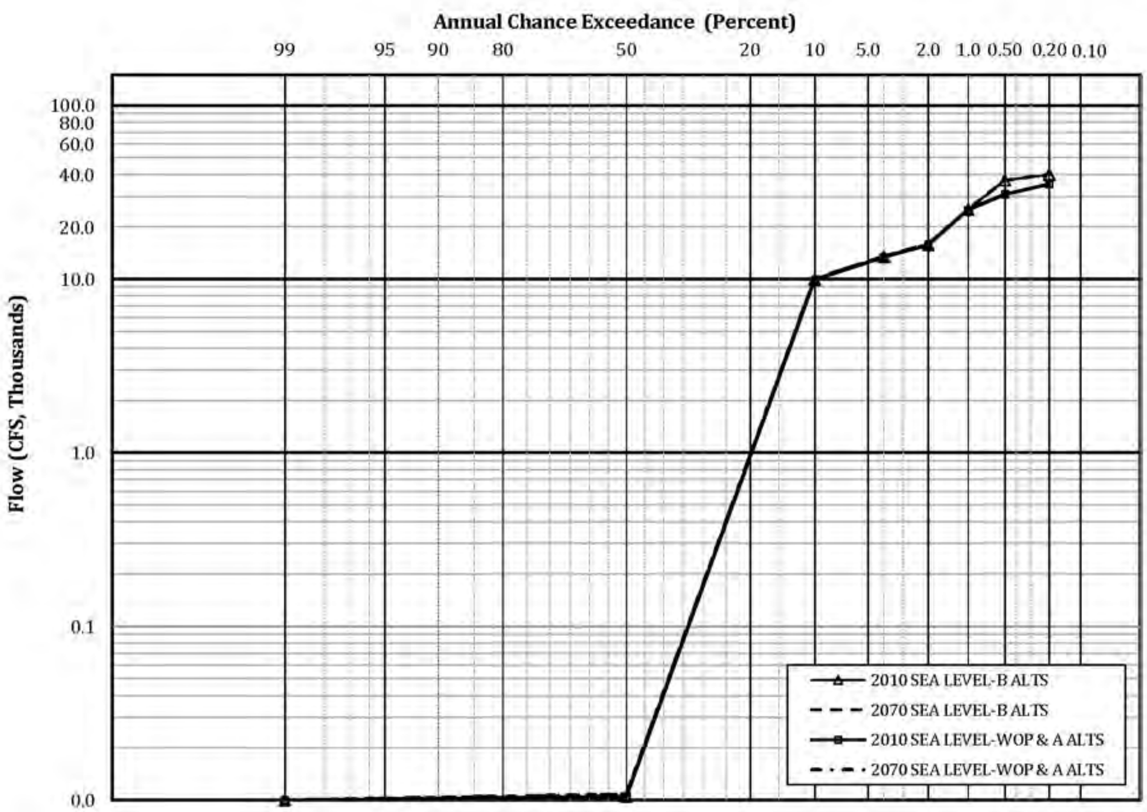
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
OLD RIVER AT CLIFTON COURT

United States Army Corps of Engineers
Sacramento District

DEC 2015





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative

-Stages and Flows for Index Point Paradise Cut at I-5 are from Paradise Cut Reach 35 at RS 6.033

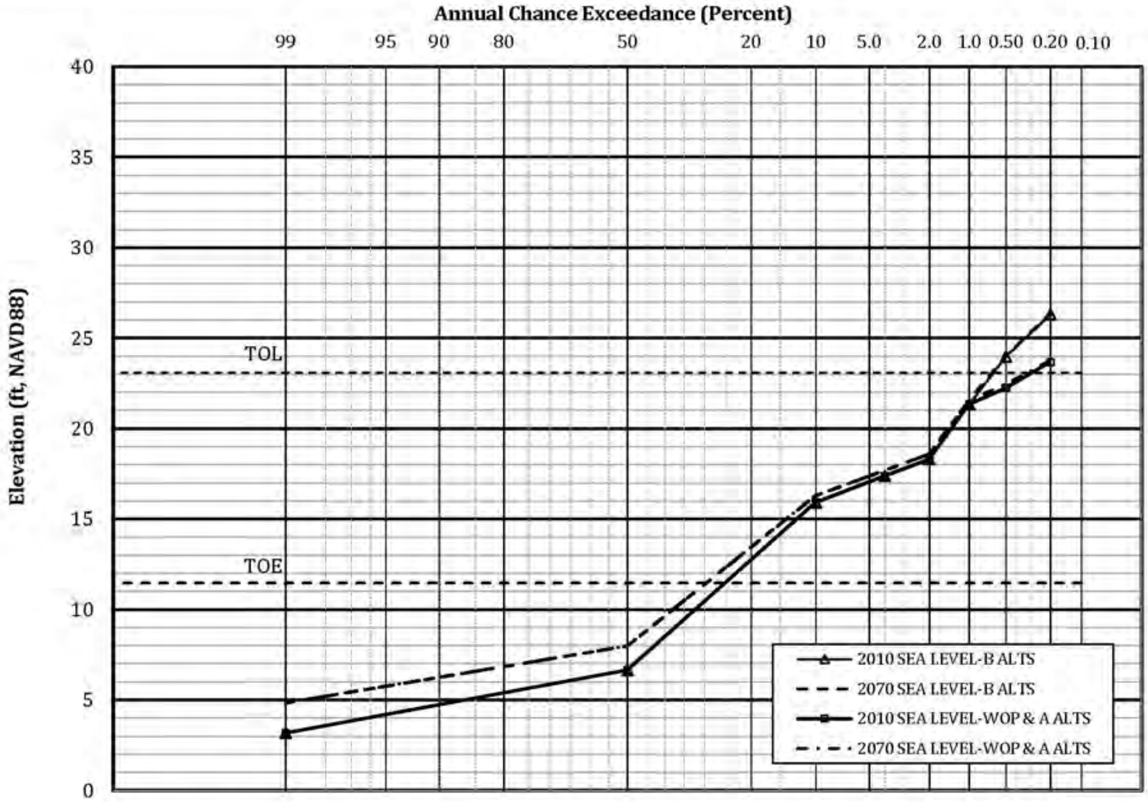
-TOL - Top of levee elevation

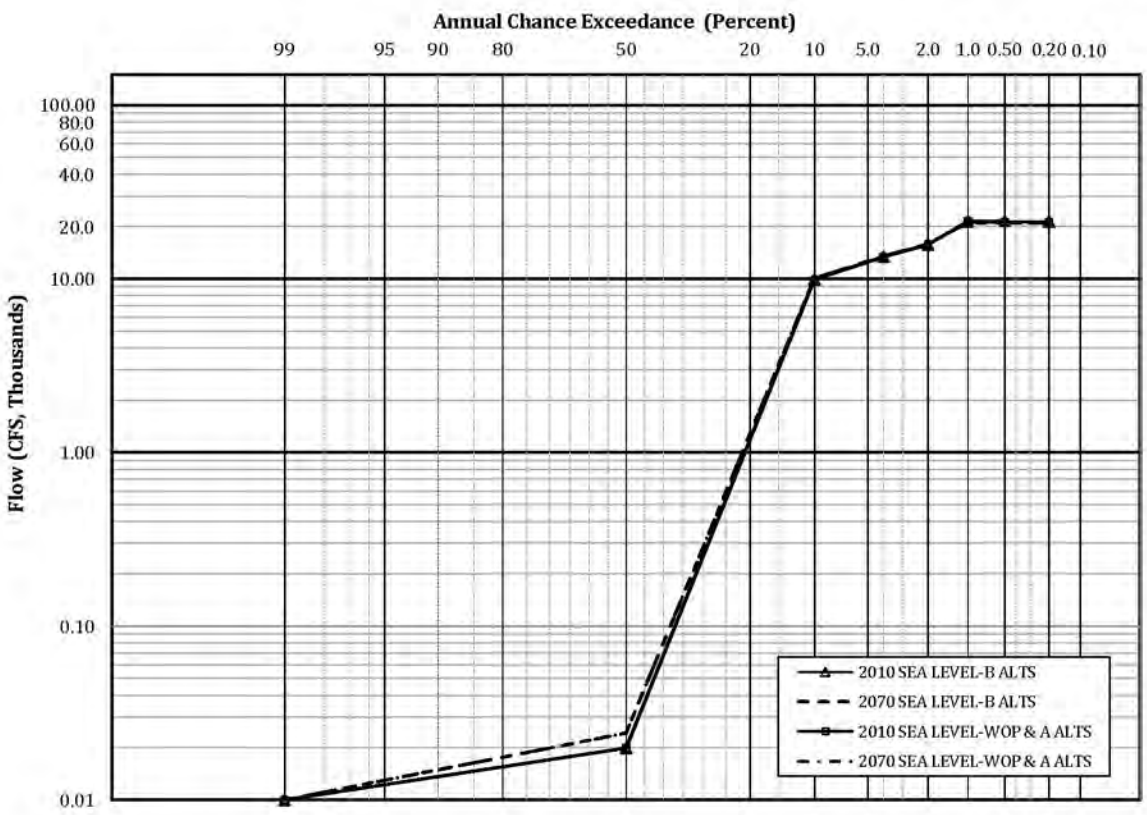
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
PARADISE CUT AT I-5

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

-Stages and Flows for Index Point Paradise Cut at Paradise Rd are from Paradise Cut Reach 35 at RS 2.893

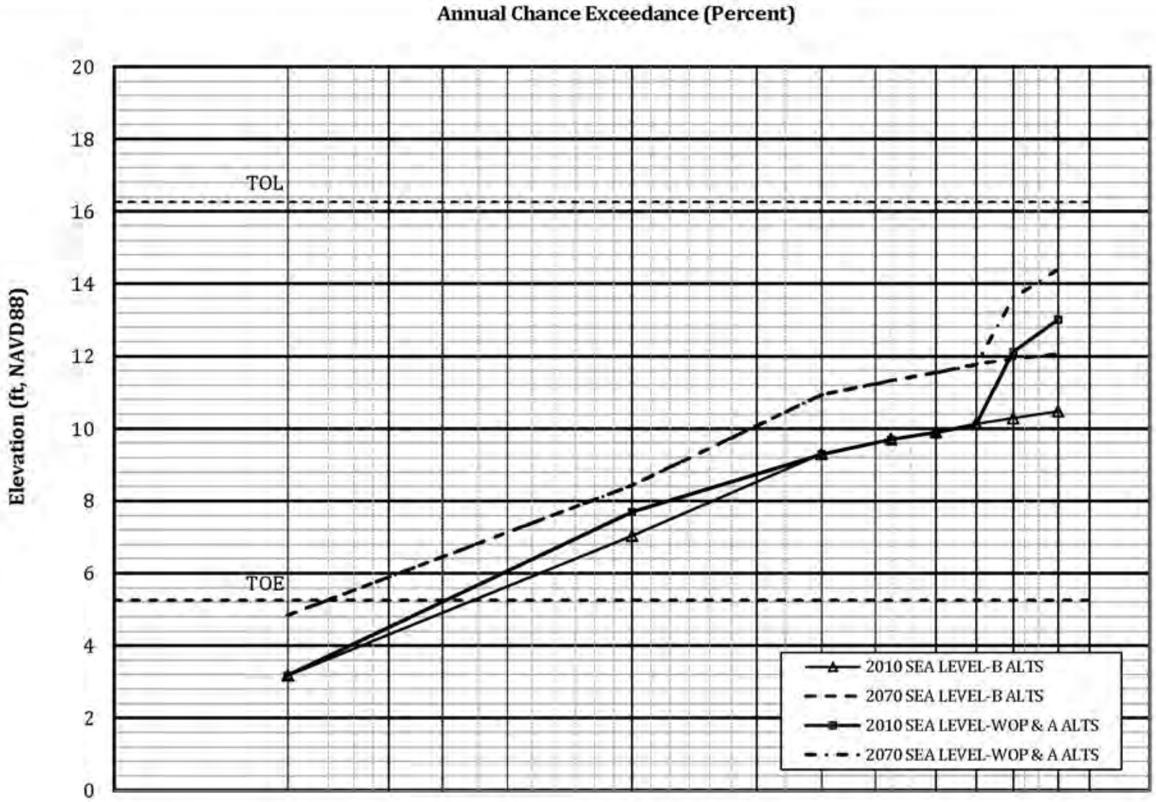
-TOL - Top of levee elevation

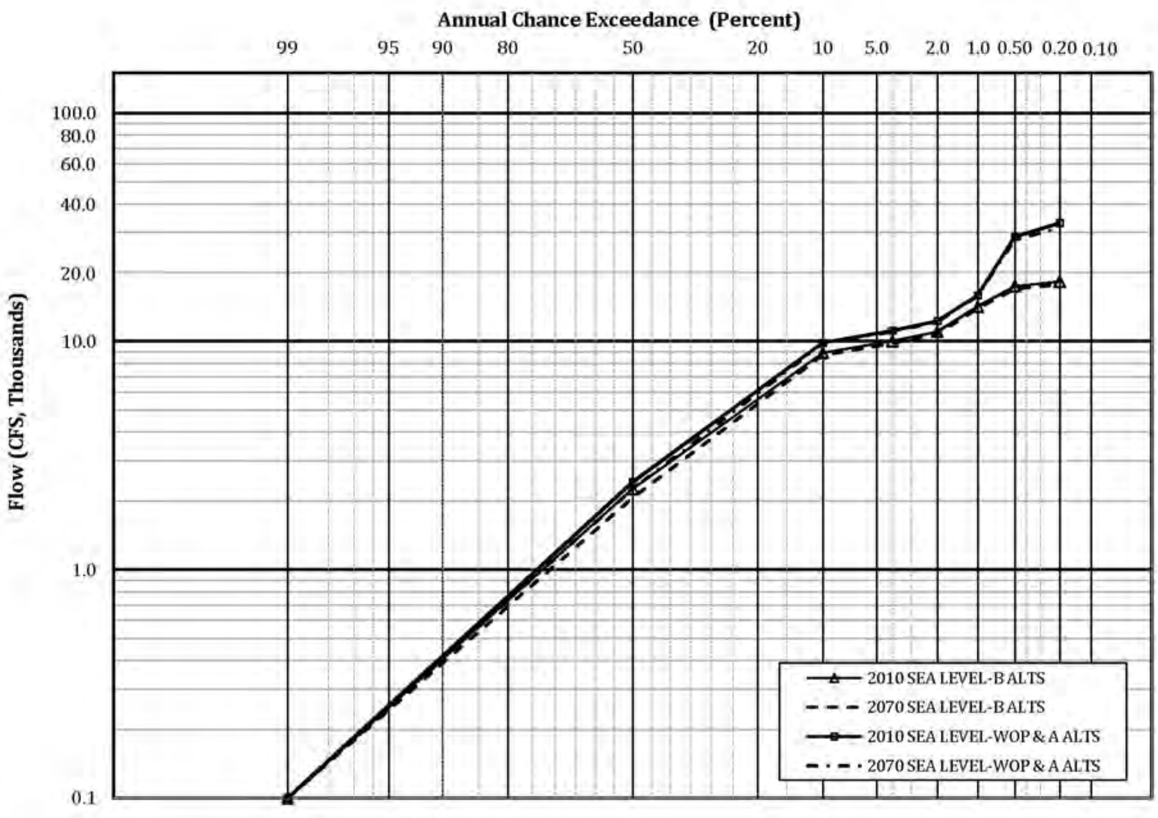
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
PARADISE CUT AT PARADISE RD

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative

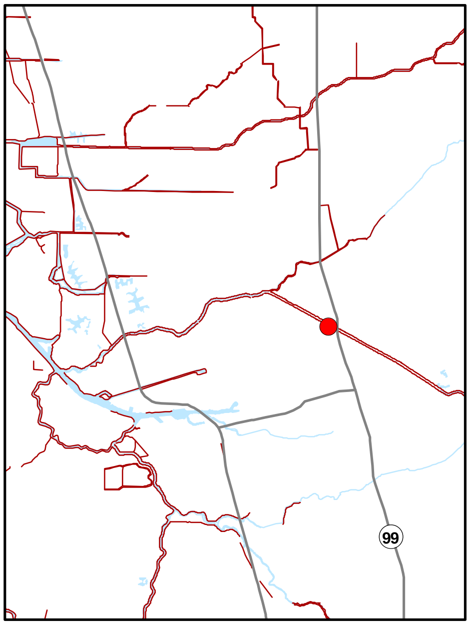
-Index Point San Joaquin River is located on the Ship Channel at RS 37.83

-TOE - Approx. elevation of natural floodplain adjacent to left bank levee
levee toe estimated where land flattens closest to levee within Rough

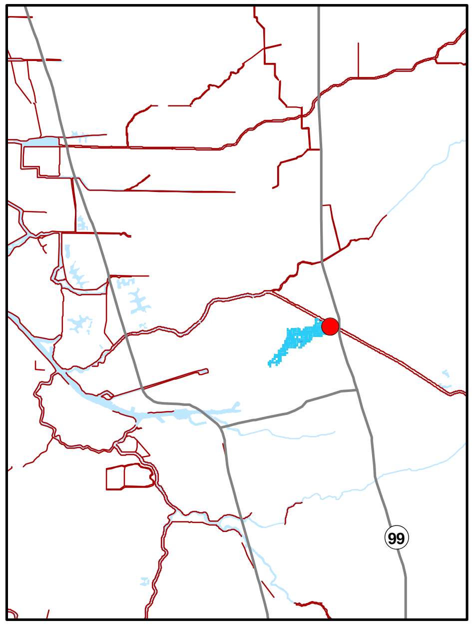
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
SJR BELOW BURNS CUTOFF

United States Army Corps of Engineers
Sacramento District



50% (1/2) ACE



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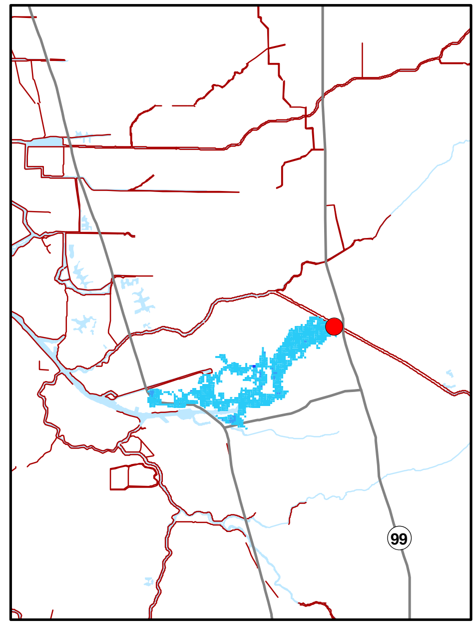

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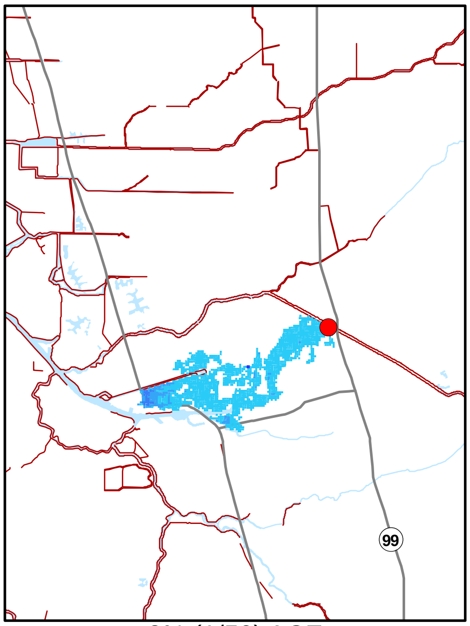

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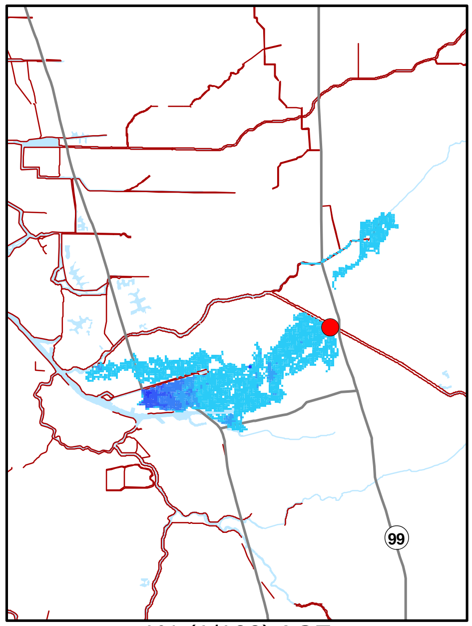
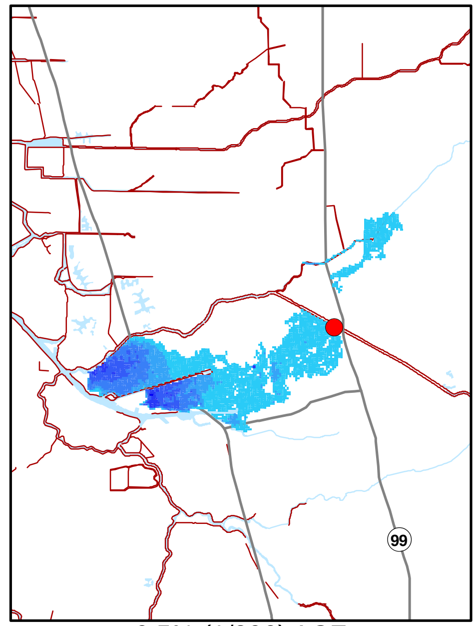
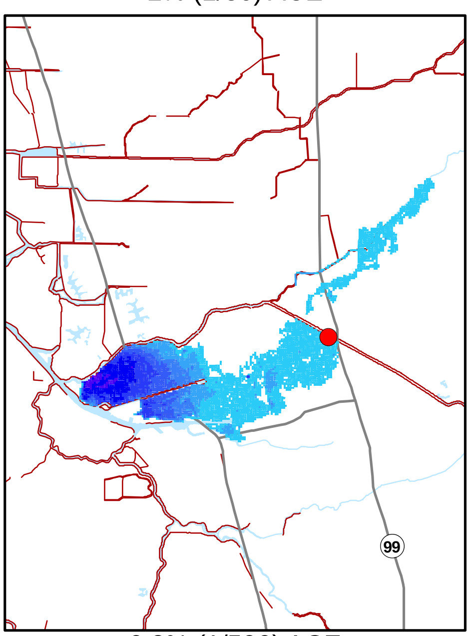


1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-SL2

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

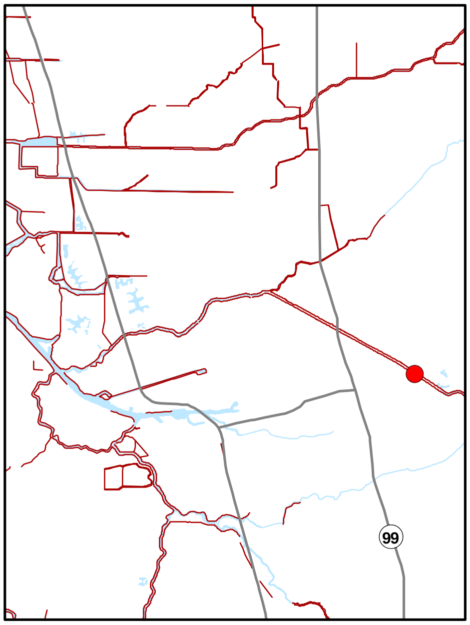
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-SL1

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

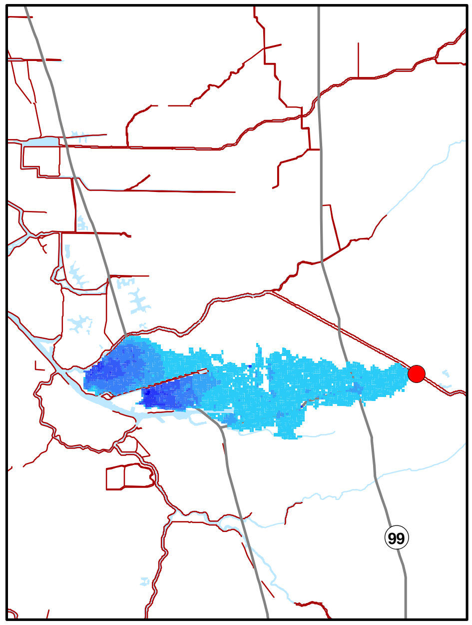
Prepared by C. Young

OCT 2014

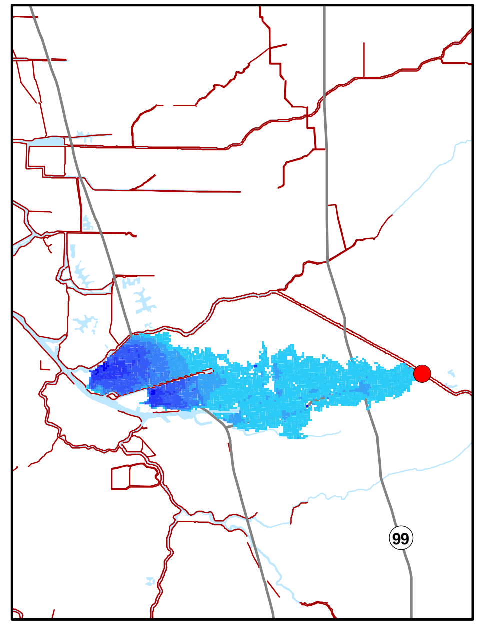
PLATE 33A



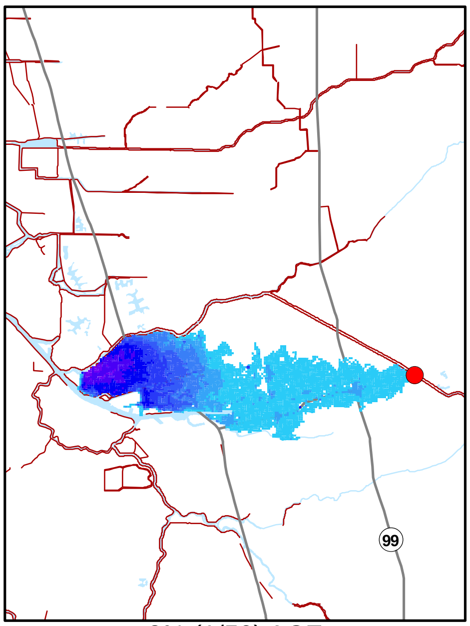
50% (1/2) ACE



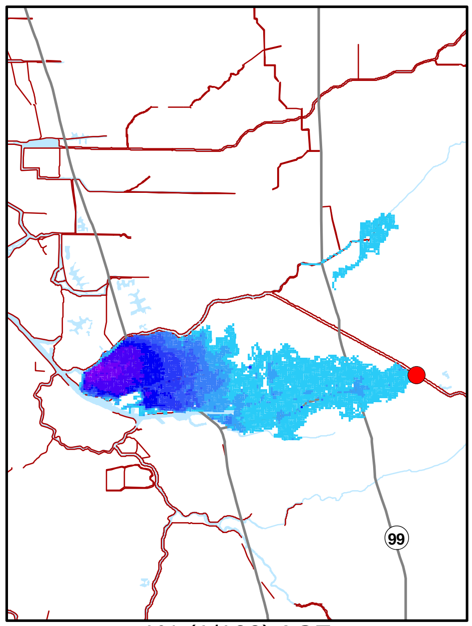
10% (1/10) ACE



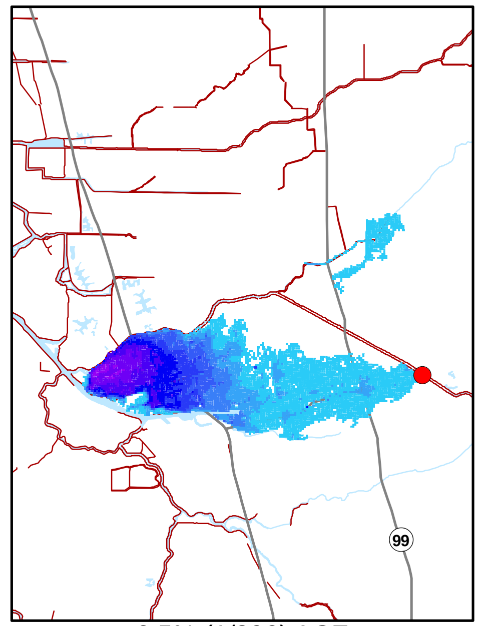
4% (1/25) ACE



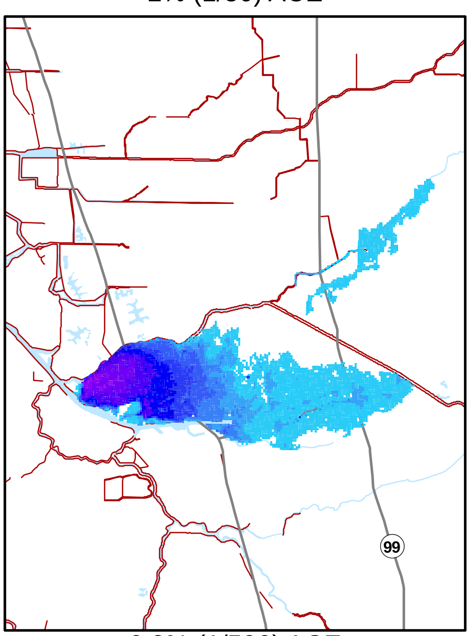
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-SL2

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

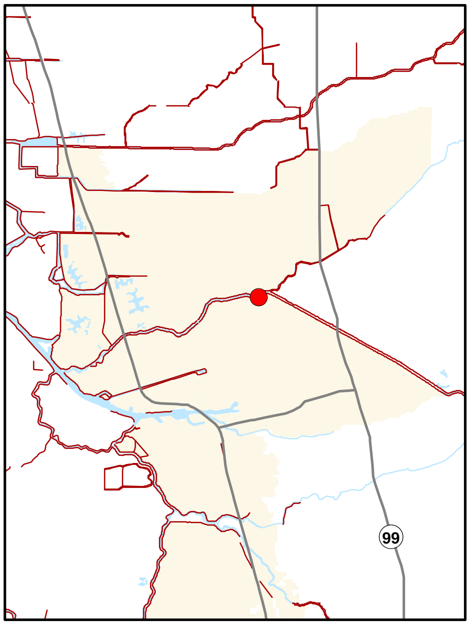
BREACH SIMULATION ALTERNATIVE 1 - NO ACTION LOCATION B-SL2

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

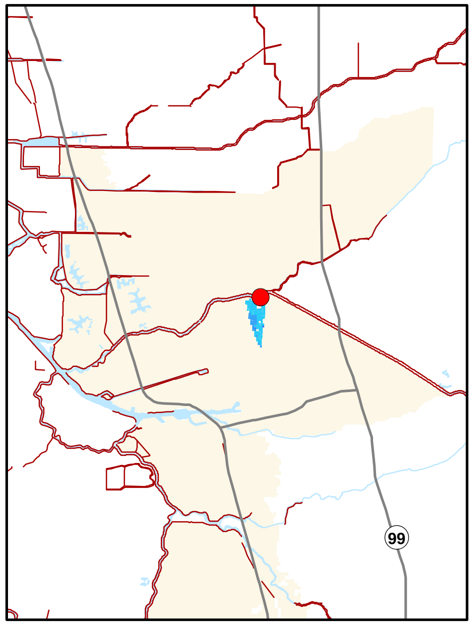
Prepared by C. Young

OCT 2014

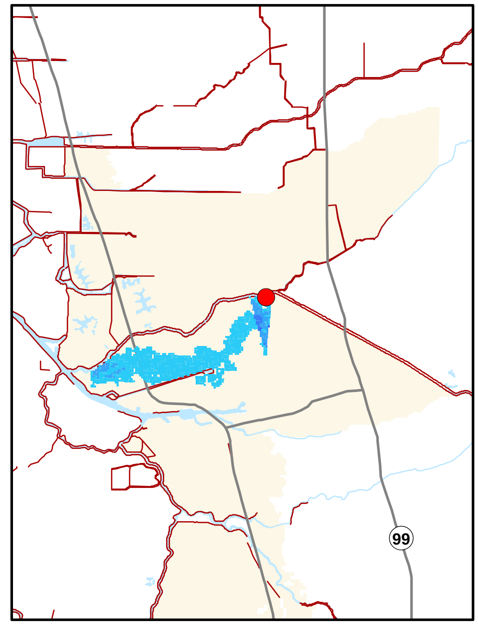
PLATE 33B



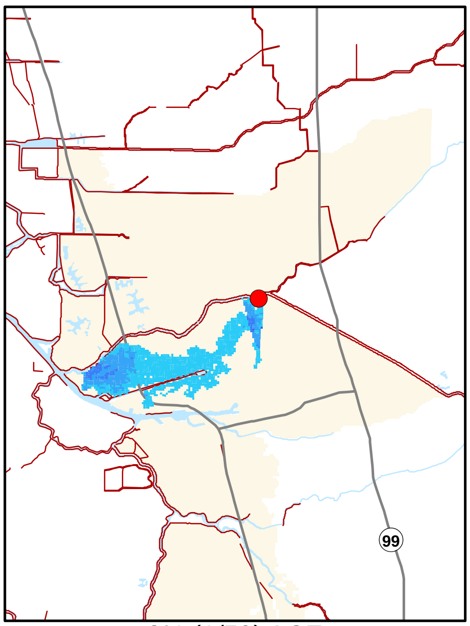
50% (1/2) ACE



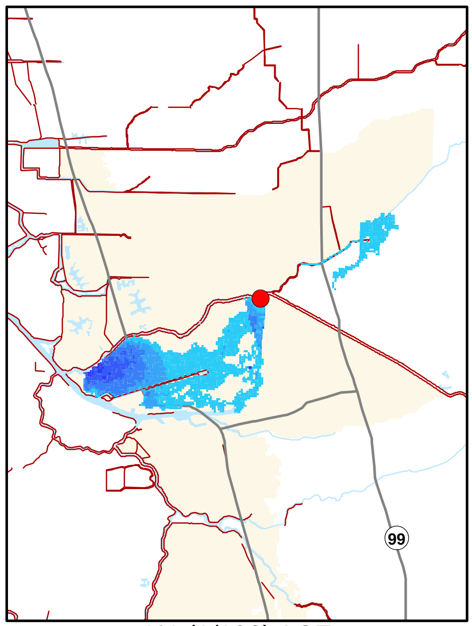
10% (1/10) ACE



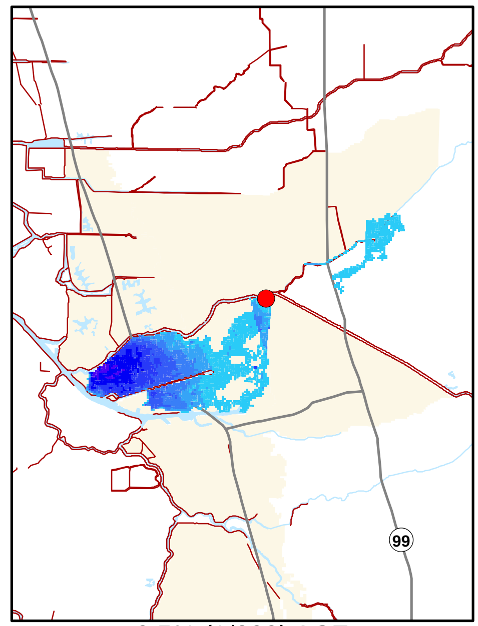
4% (1/25) ACE



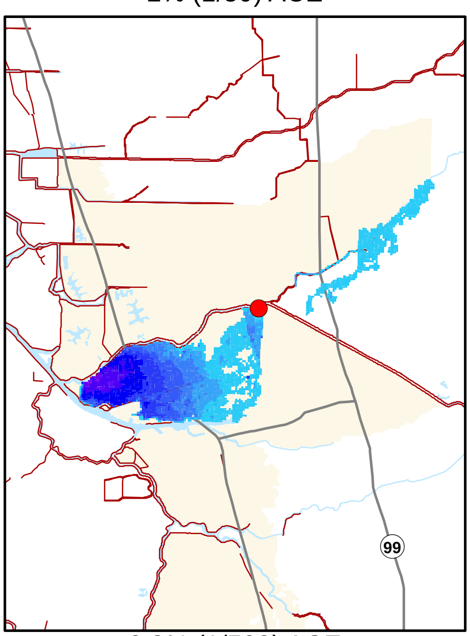
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-CL2

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00

Levees (CLD, NLD)

LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

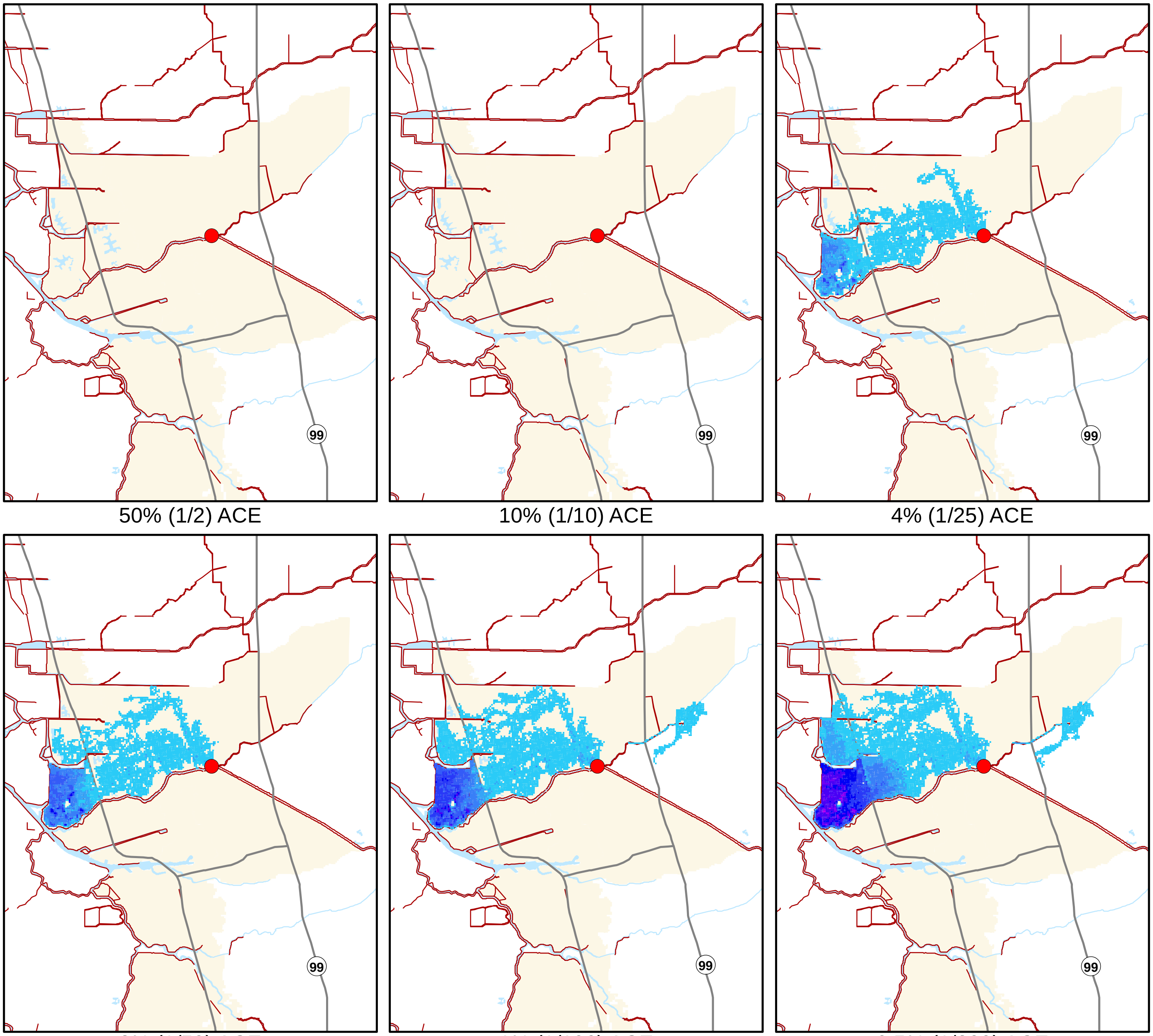
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-CL2

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

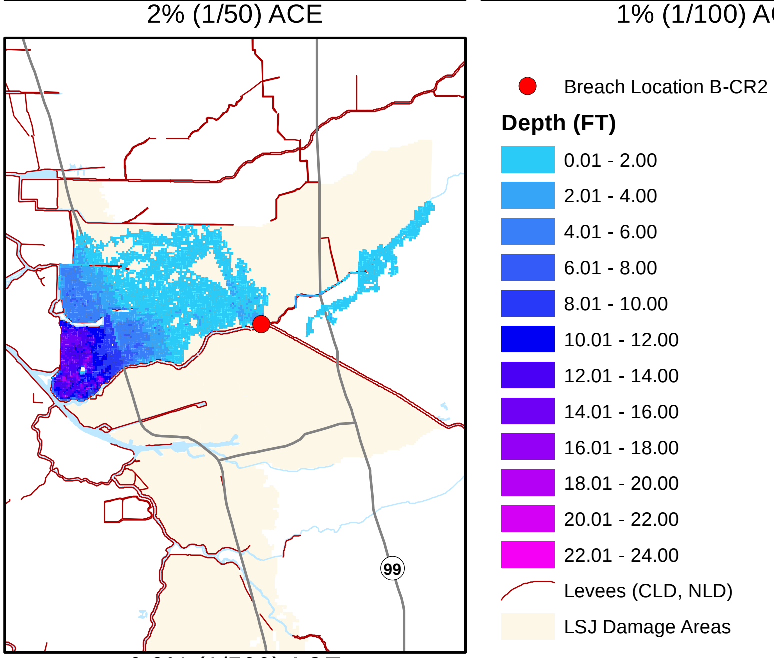
Prepared by C. Young

JUN 2014



50% (1/2) ACE | 10% (1/10) ACE | 4% (1/25) ACE

2% (1/50) ACE | 1% (1/100) ACE | 0.5% (1/200) ACE



0.2% (1/500) ACE

Miles
0 5
N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

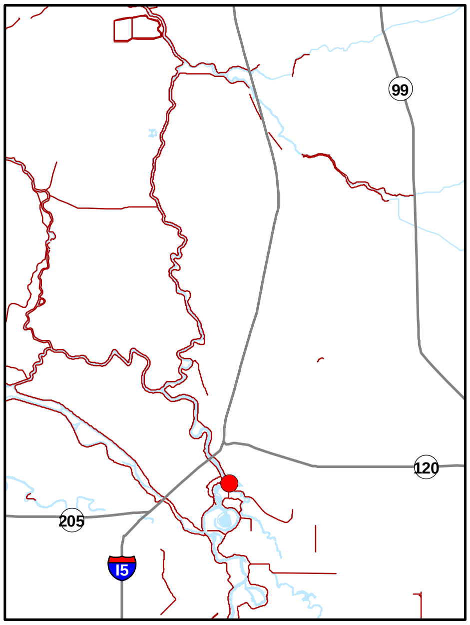
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-CR2

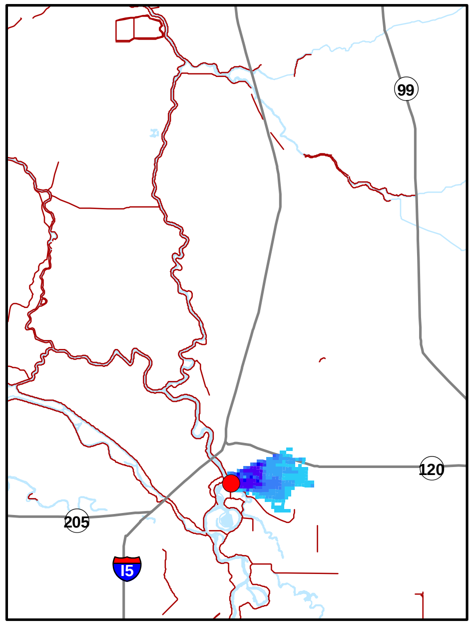
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

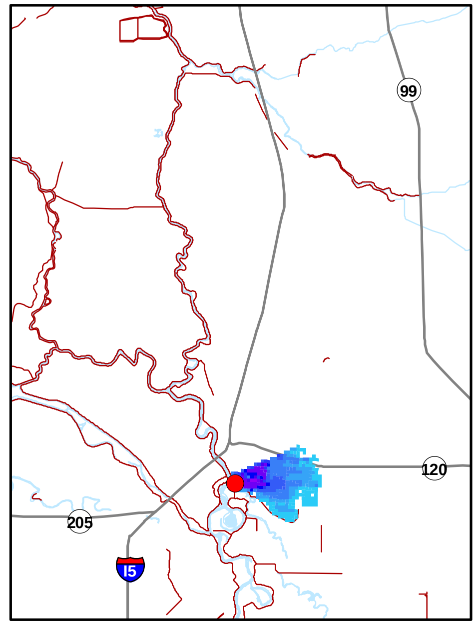
JUN 2014



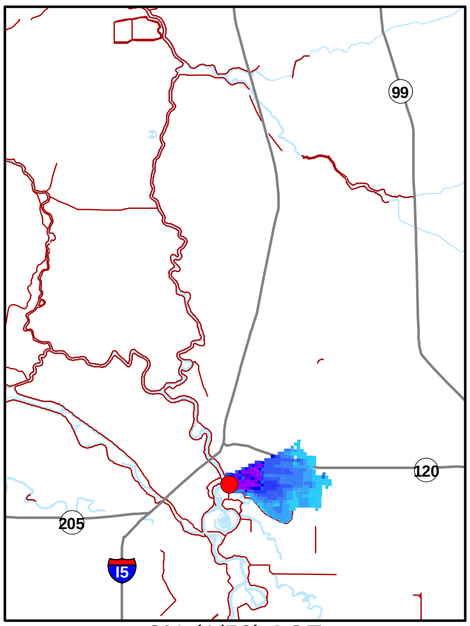
50% (1/2) ACE



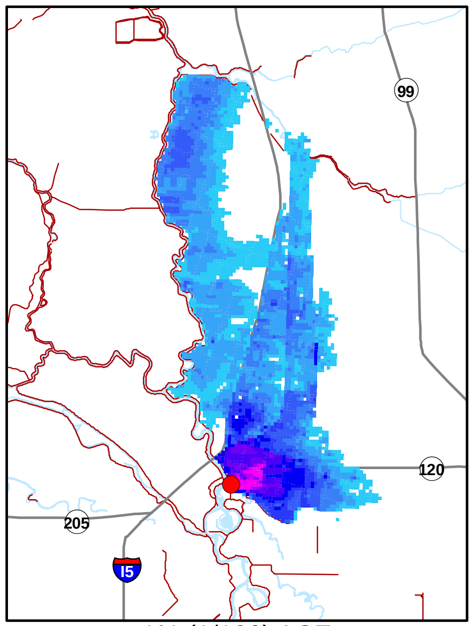
10% (1/10) ACE



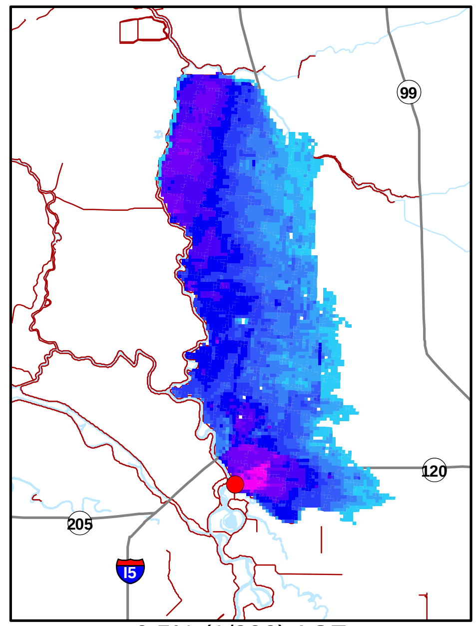
4% (1/25) ACE



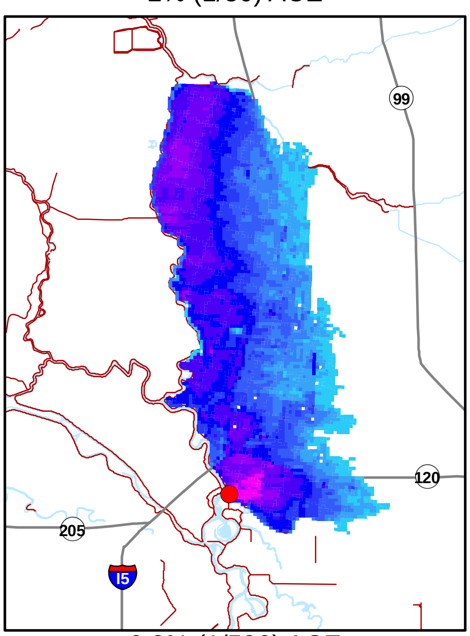
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-LR4

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

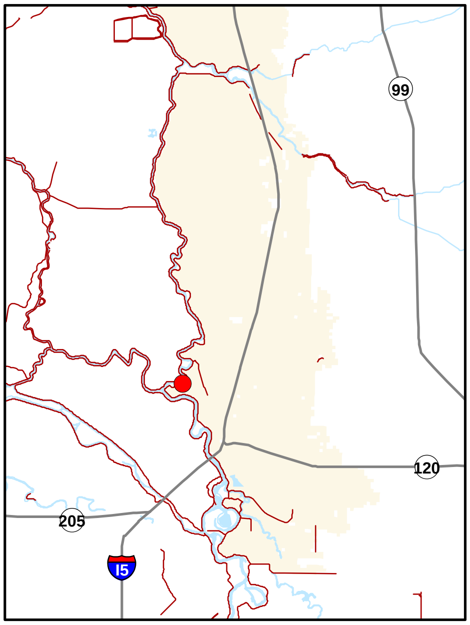
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-LR4

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

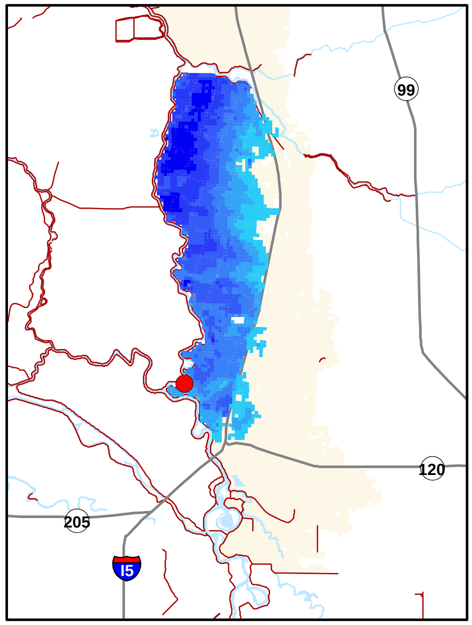
Prepared by C. Young

APR 2014

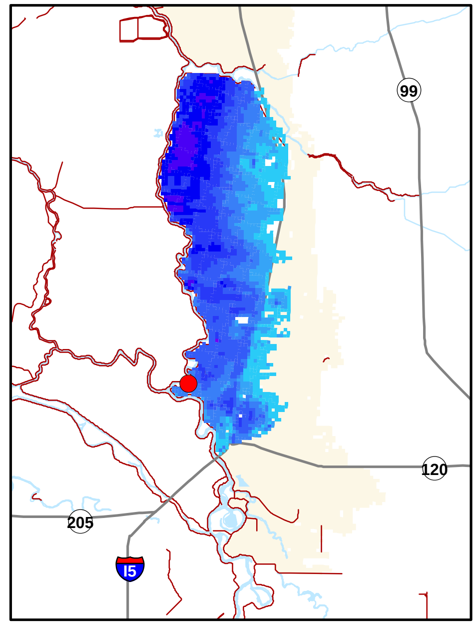
PLATE 33E



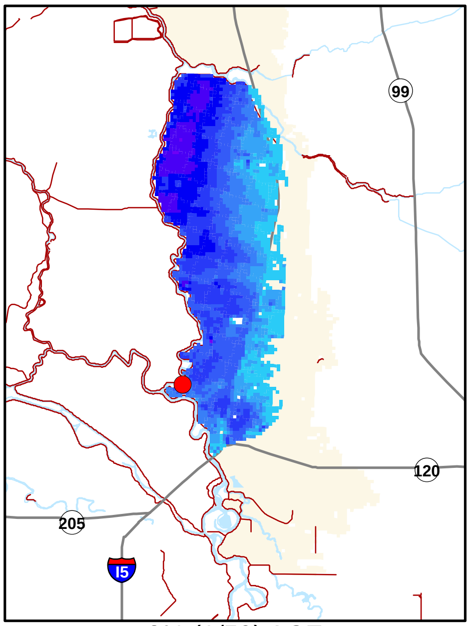
50% (1/2) ACE



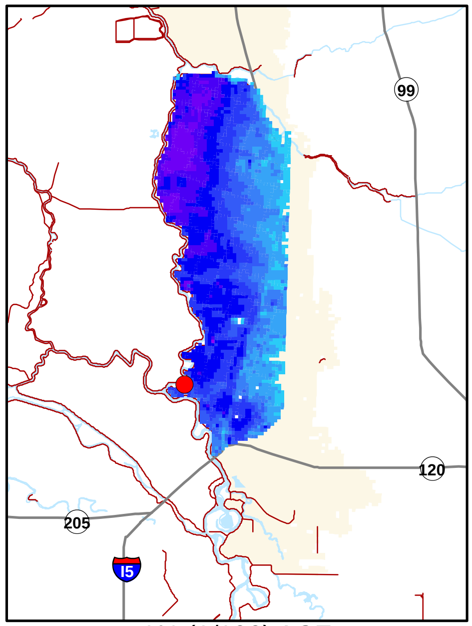
10% (1/10) ACE



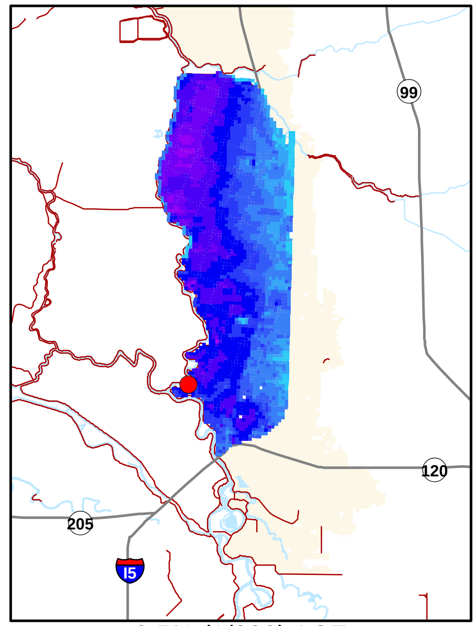
4% (1/25) ACE



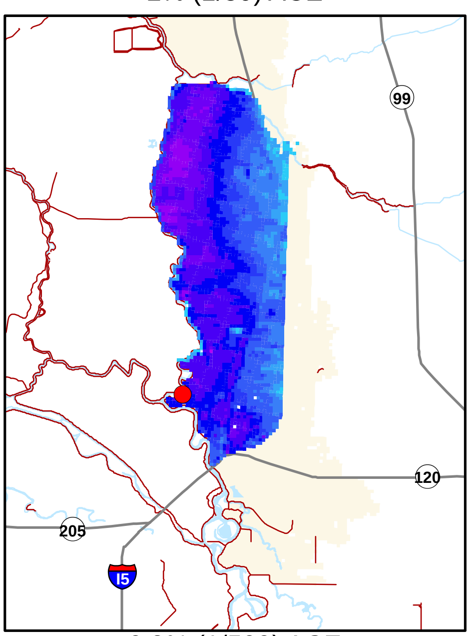
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-LR3

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

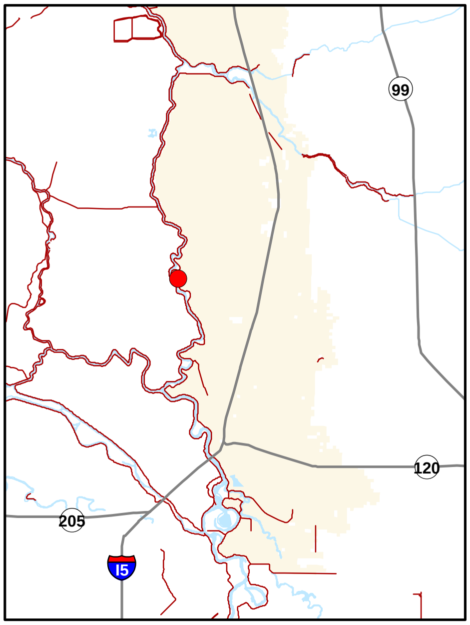
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-LR3

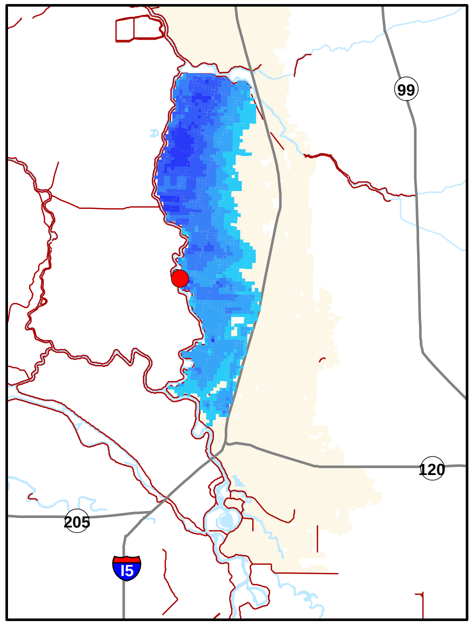
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

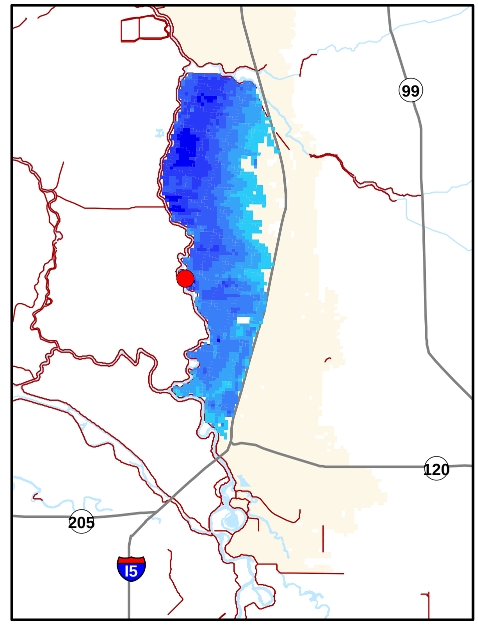
JUN 2014



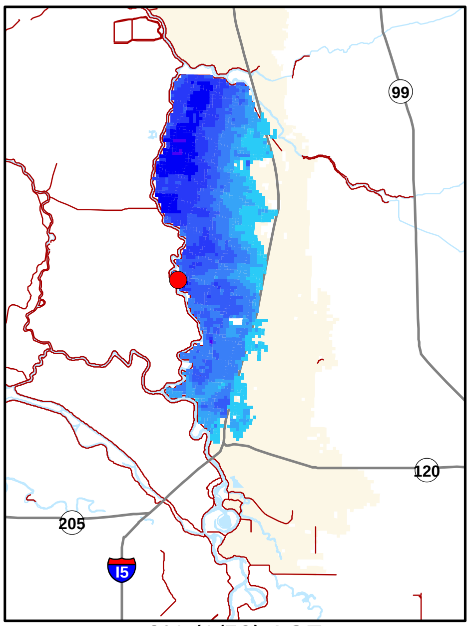
50% (1/2) ACE



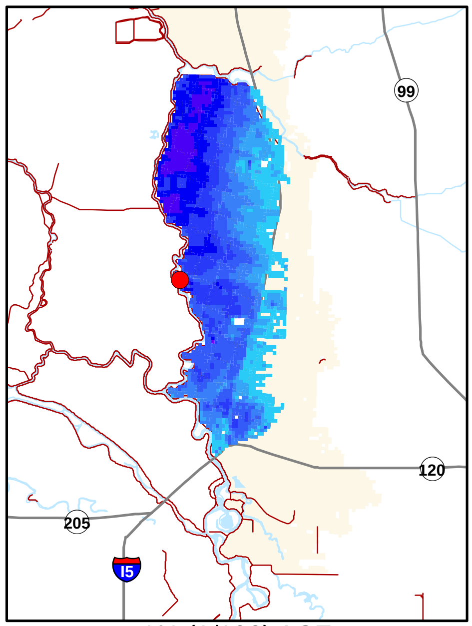
10% (1/10) ACE



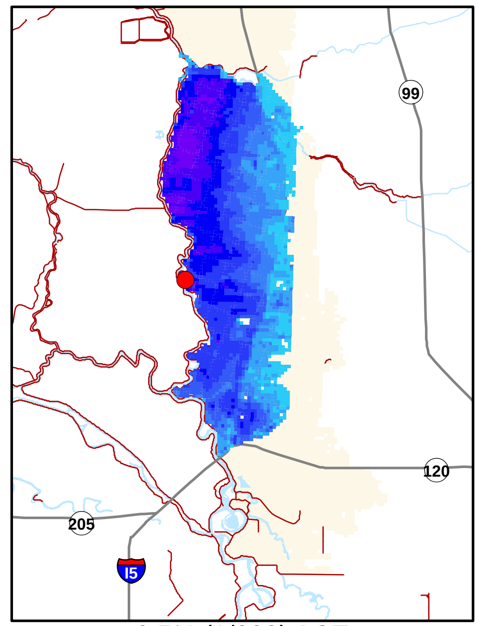
4% (1/25) ACE



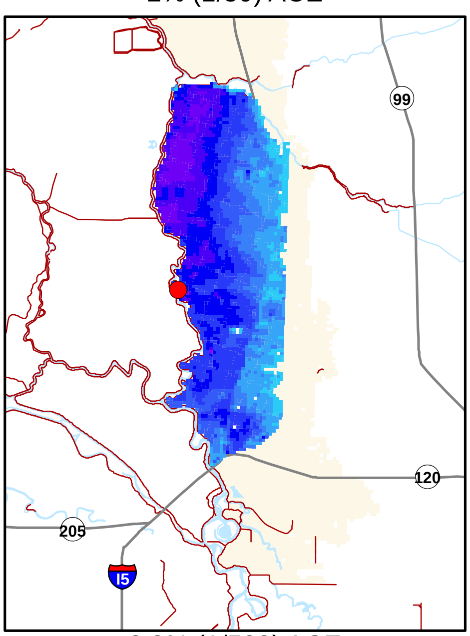
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-LR2

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

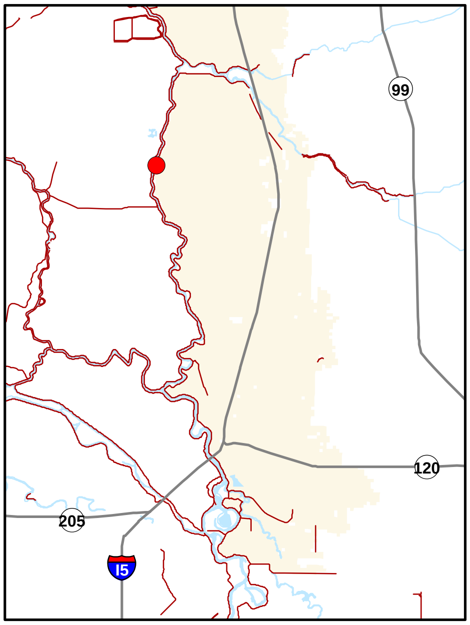
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-LR2

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

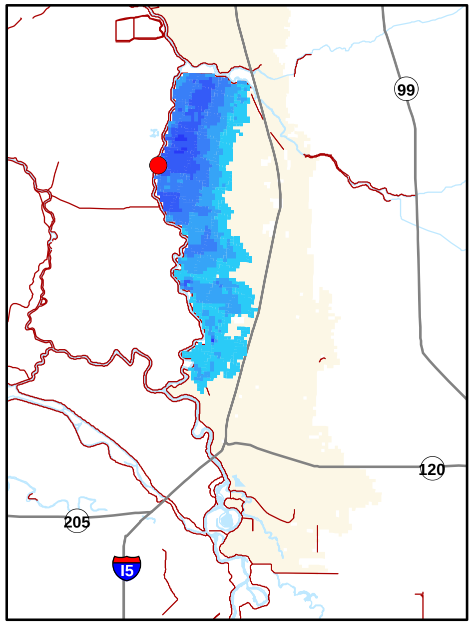
Prepared by C. Young

JUN 2014

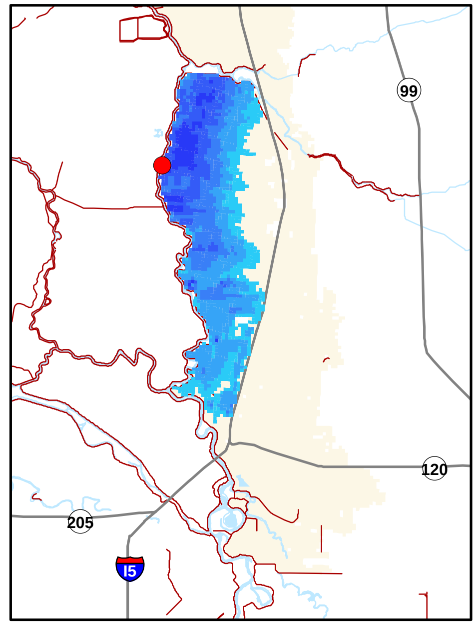
PLATE 33G



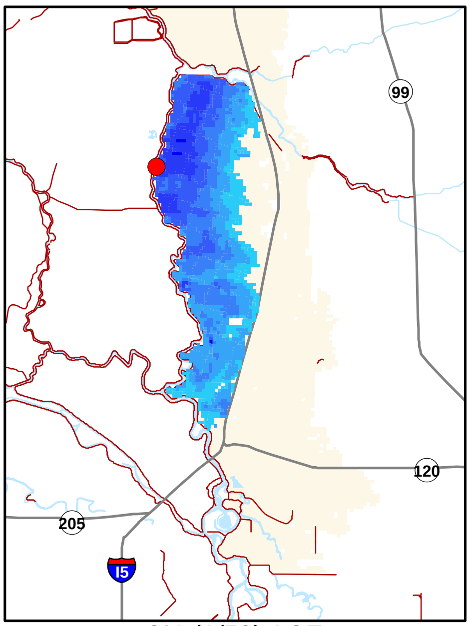
50% (1/2) ACE



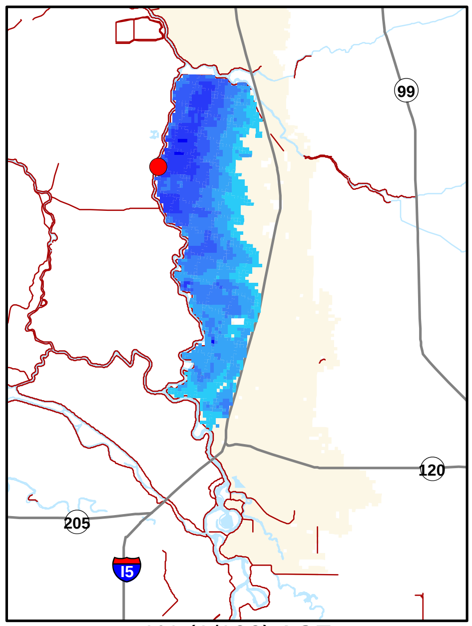
10% (1/10) ACE



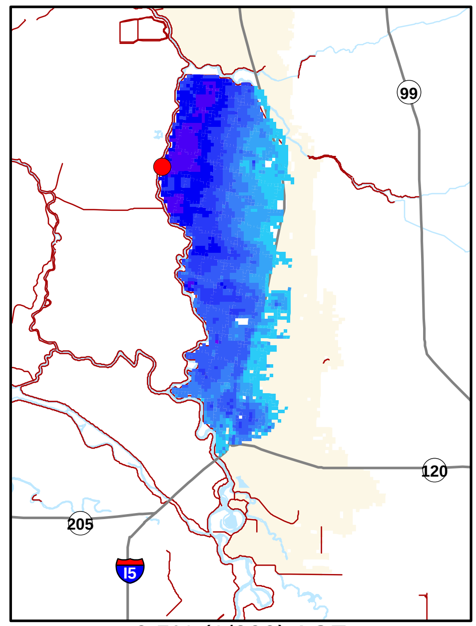
4% (1/25) ACE



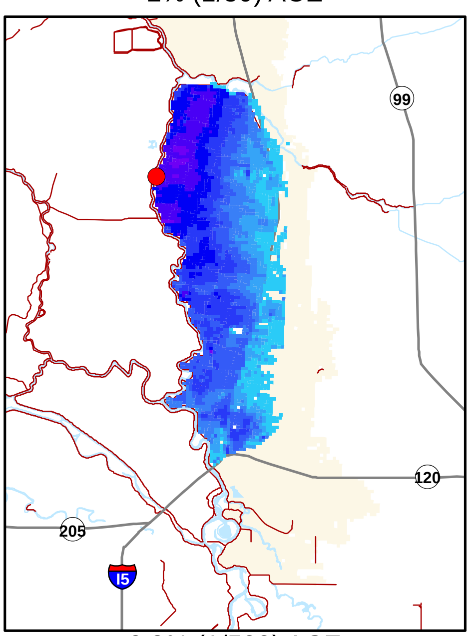
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-LR1

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00

Levees (CLD, NLD)

LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

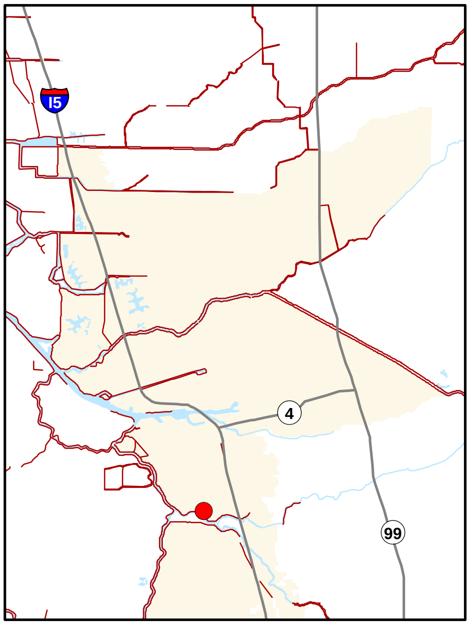
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-LR1

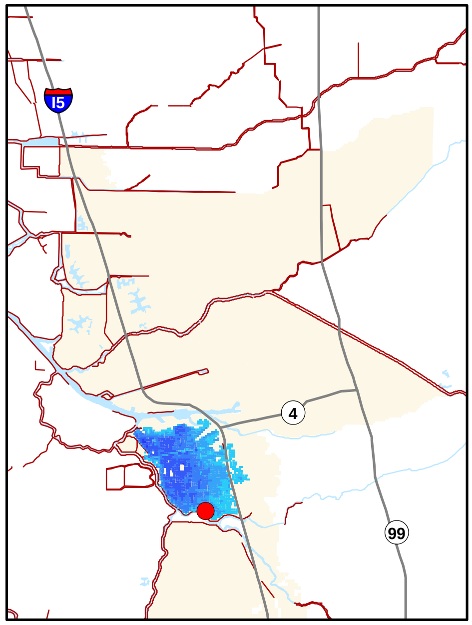
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

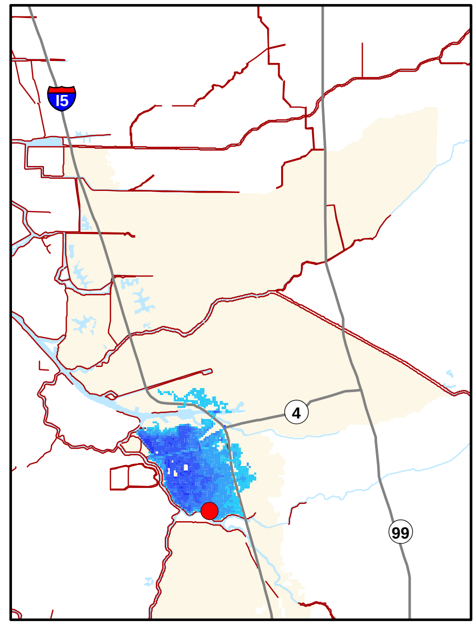
JUN 2014



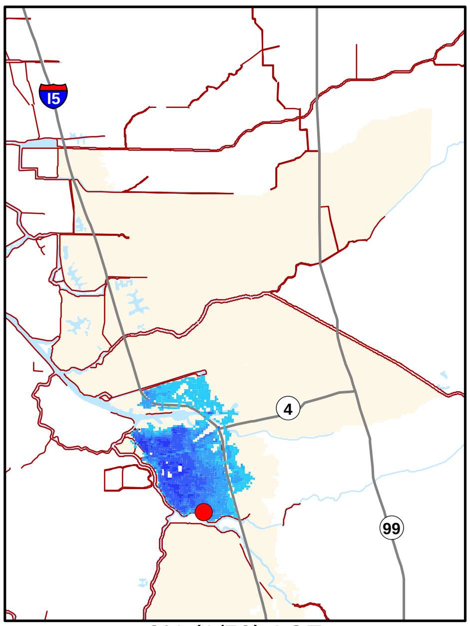
50% (1/2) ACE



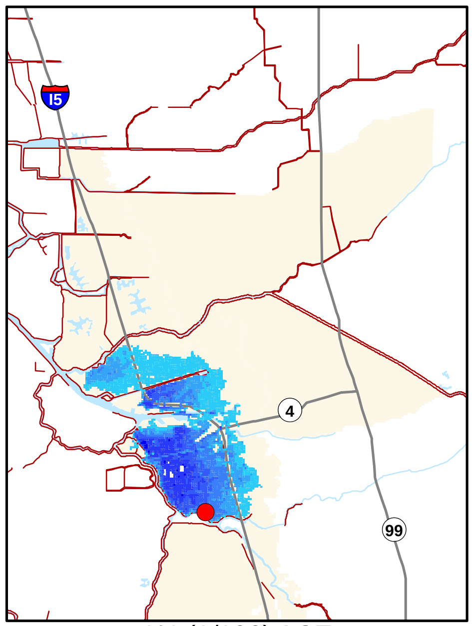
10% (1/10) ACE



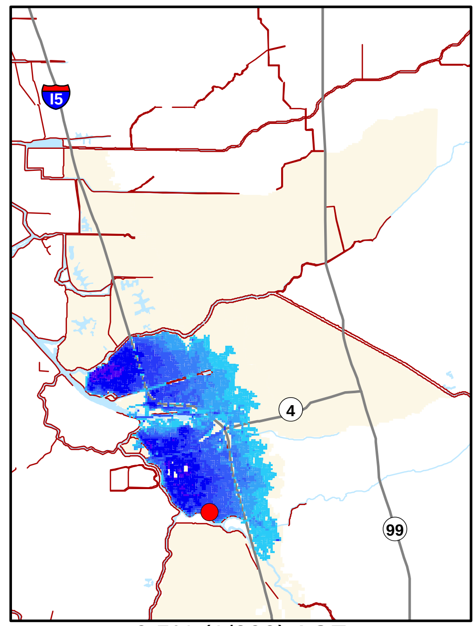
4% (1/25) ACE



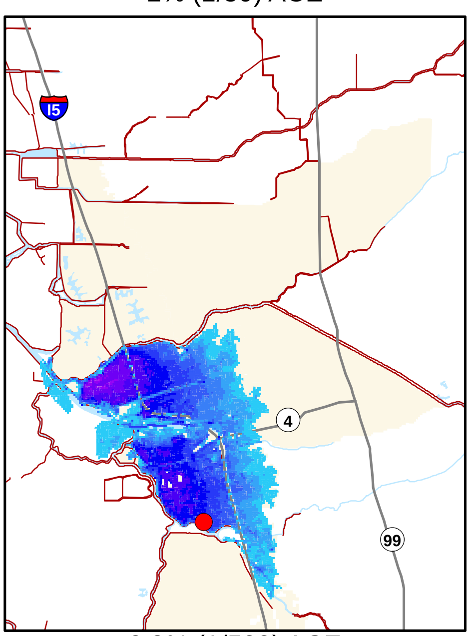
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-FR1

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00

Levees (CLD, NLD)

LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

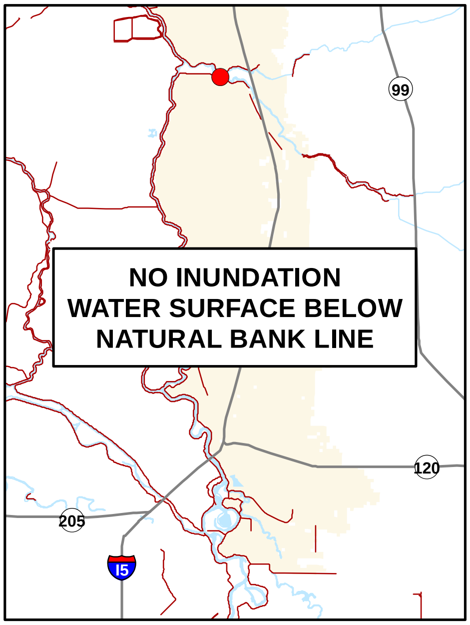
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-FR1

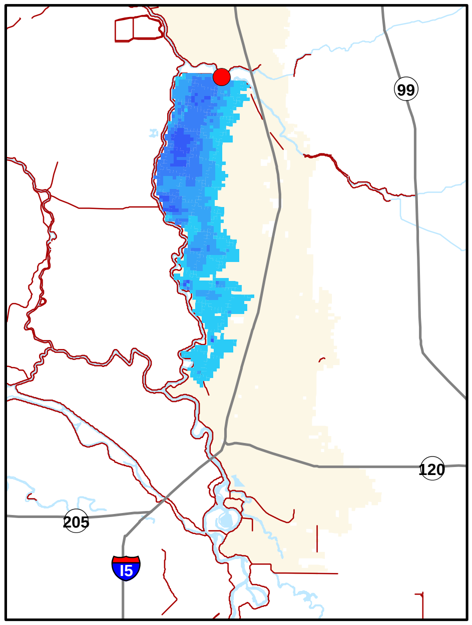
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

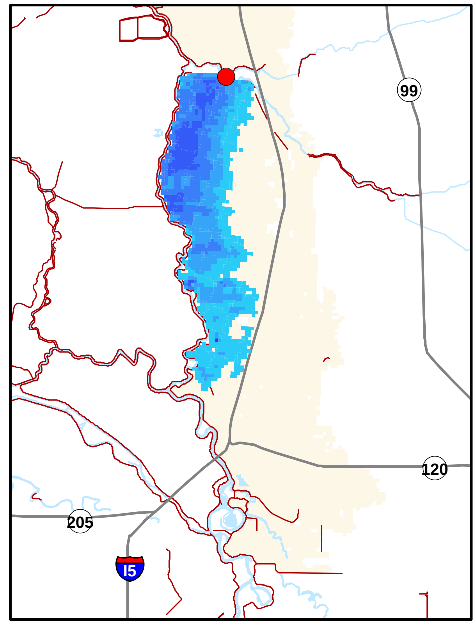
JUN 2014



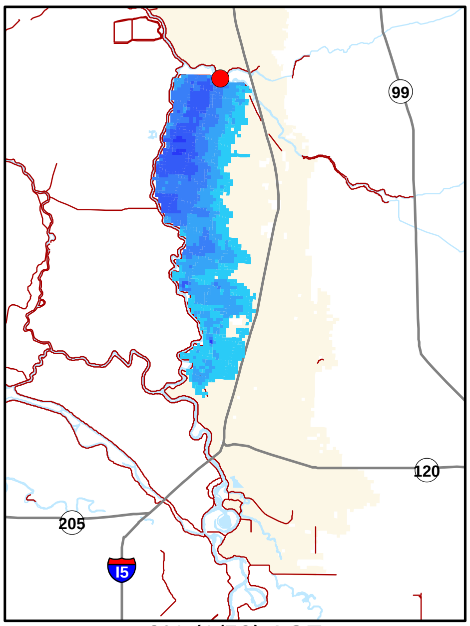
50% (1/2) ACE



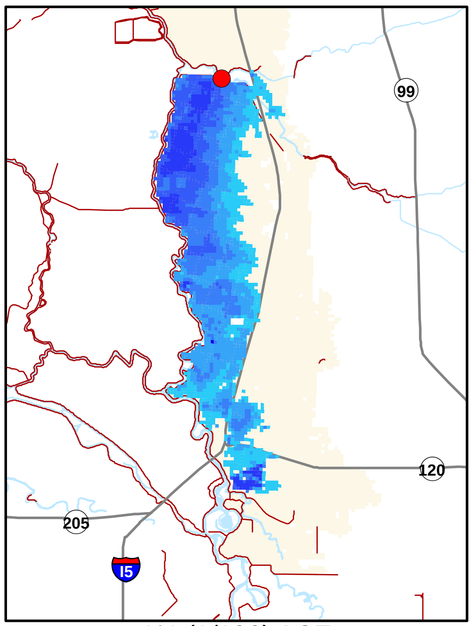
10% (1/10) ACE



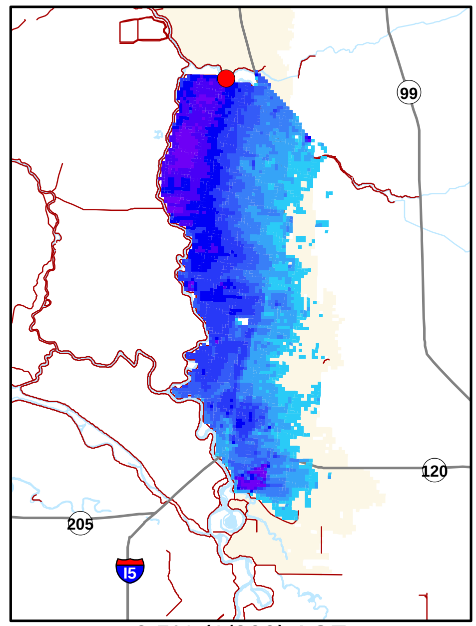
4% (1/25) ACE



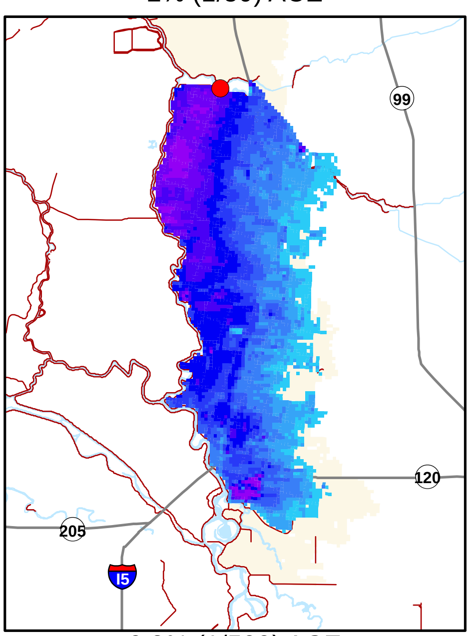
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-FL1

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

0 5 Miles

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

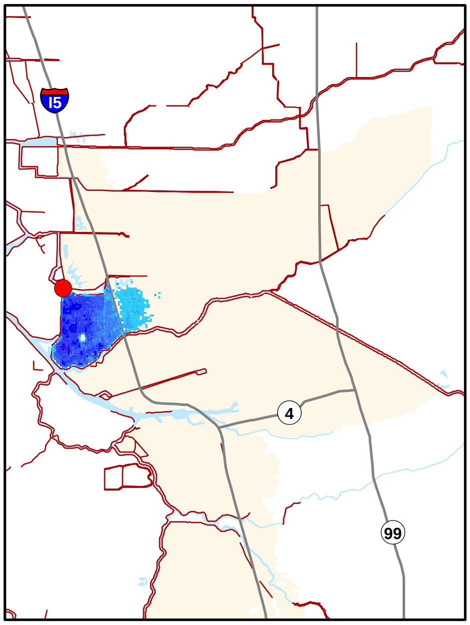
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-FL1

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

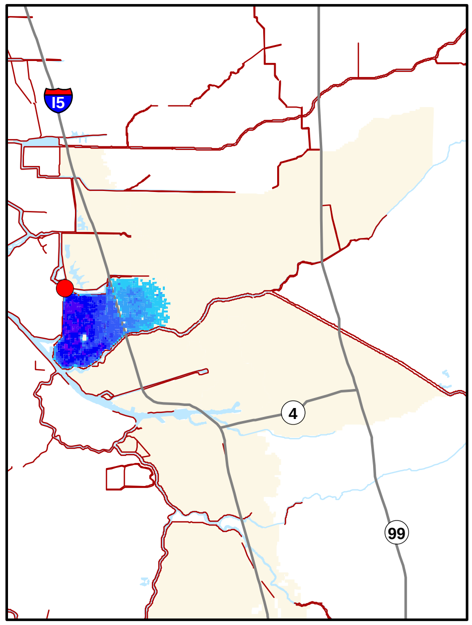
Prepared by C. Young

JUN 2014

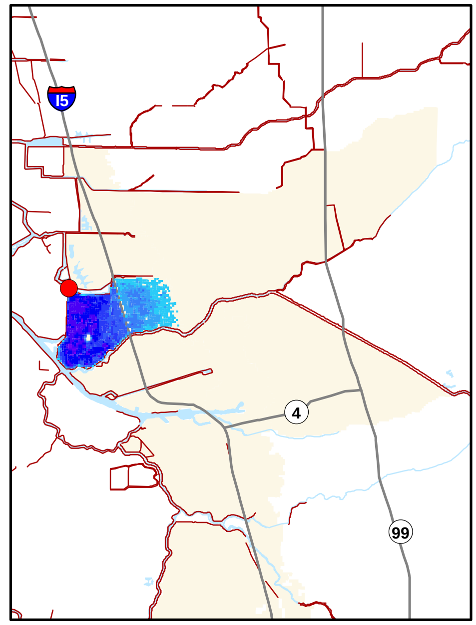
PLATE 33J



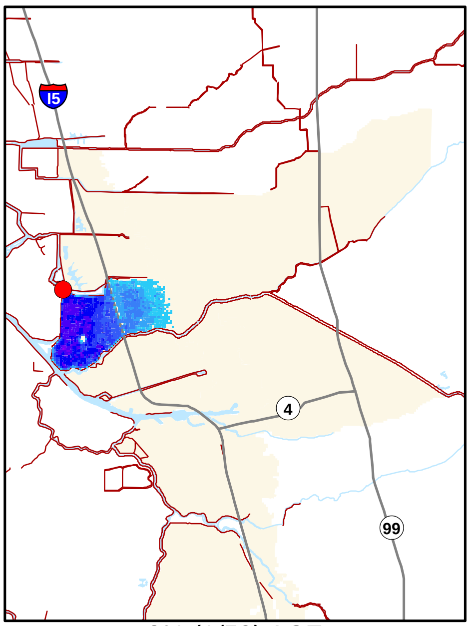
50% (1/2) ACE



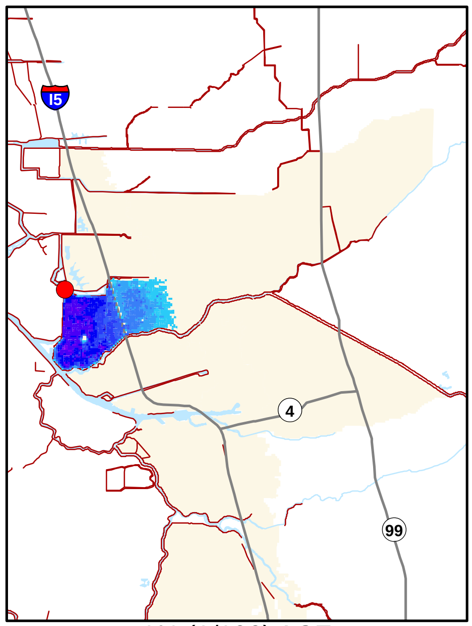
10% (1/10) ACE



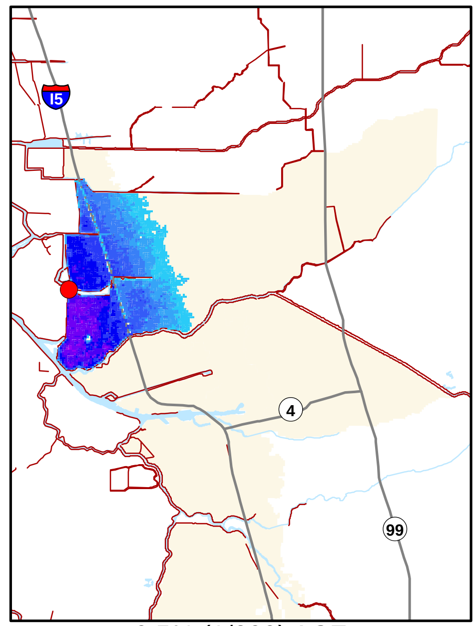
4% (1/25) ACE



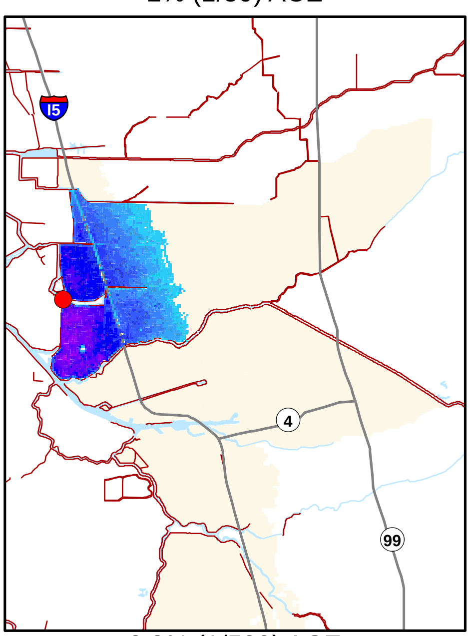
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-D-BS

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

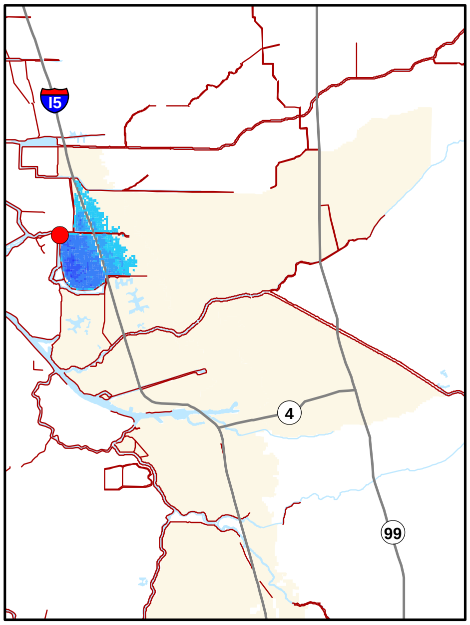
BREACH SIMULATION ALTERNATIVE 1 - NO ACTION LOCATION B-D-BS

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

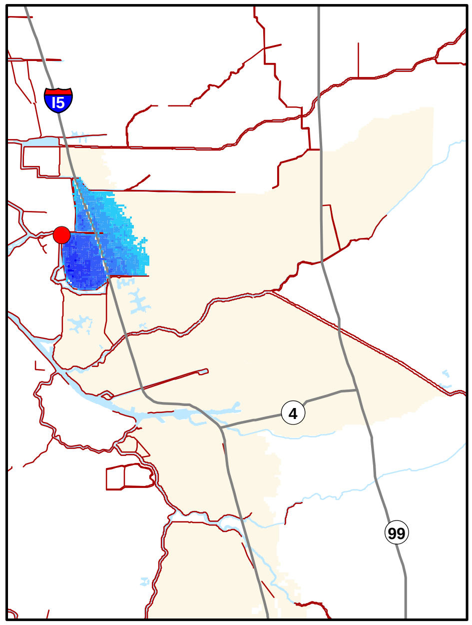
Prepared by C. Young

JUN 2014

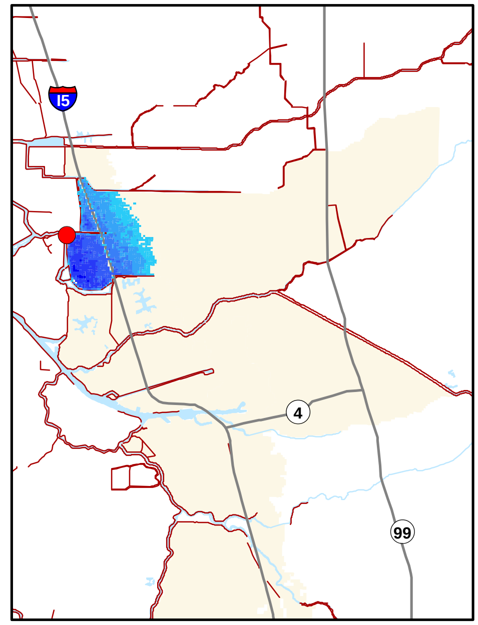
PLATE 34A



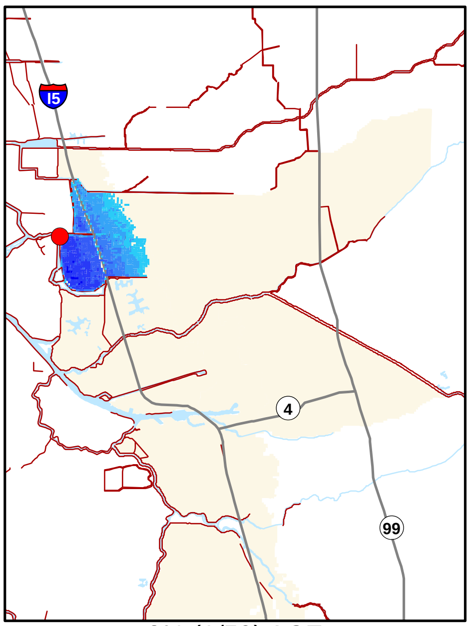
50% (1/2) ACE



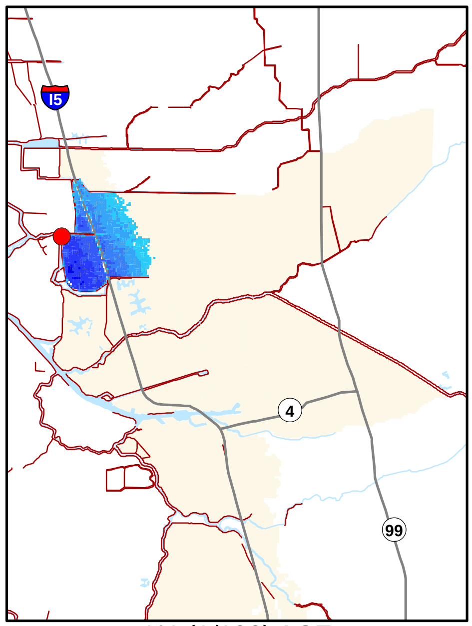
10% (1/10) ACE



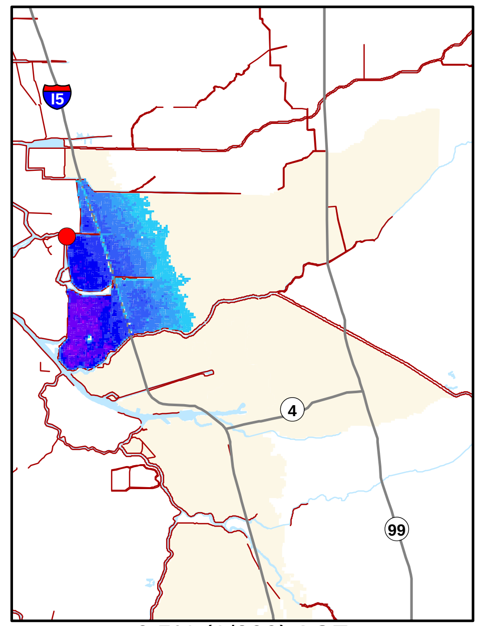
4% (1/25) ACE



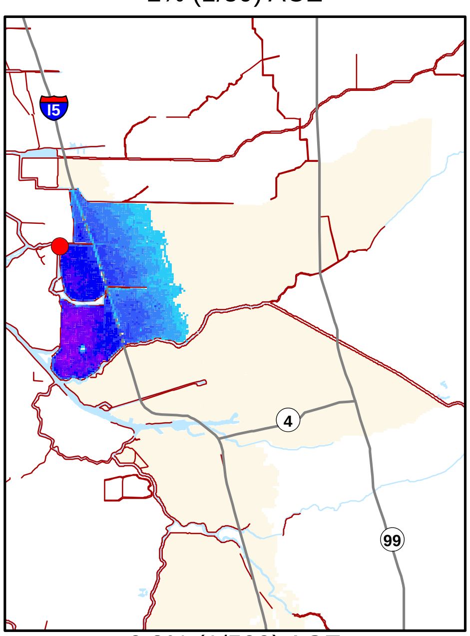
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-D3

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00

Levees (CLD, NLD)

LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

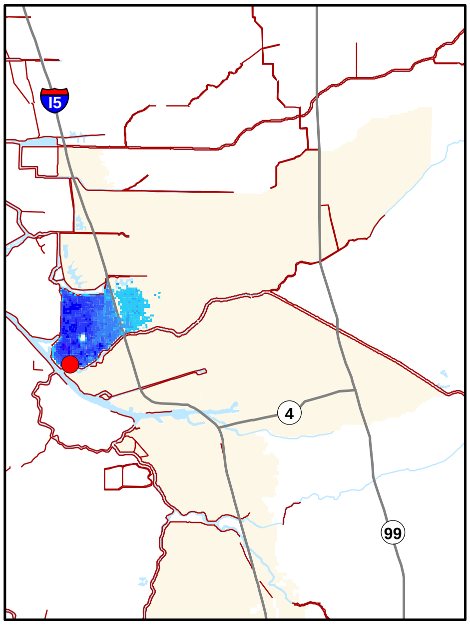
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-D3

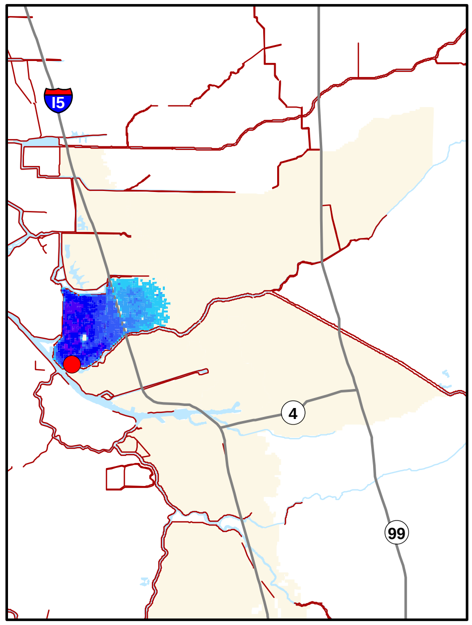
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

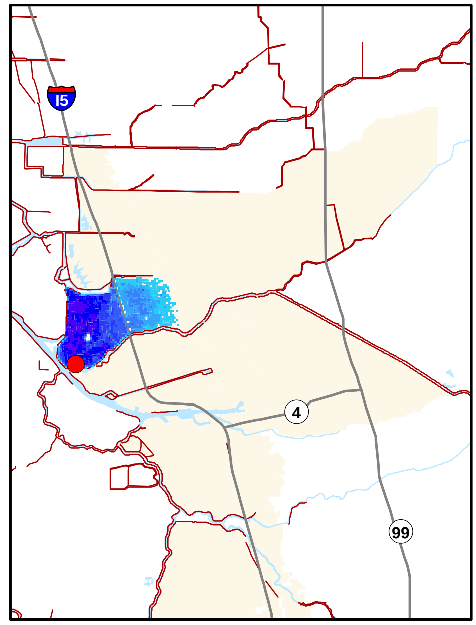
JUN 2014



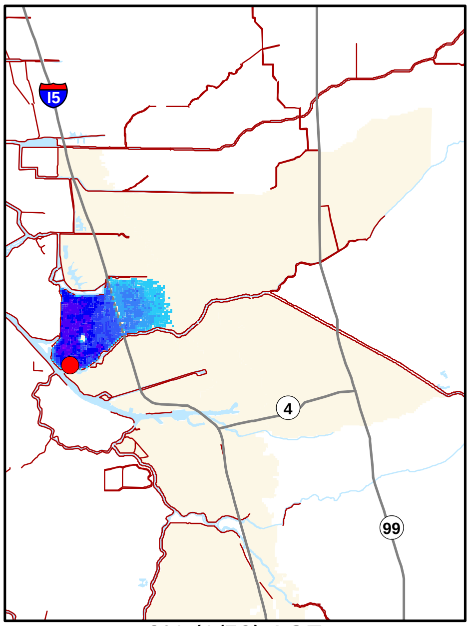
50% (1/2) ACE



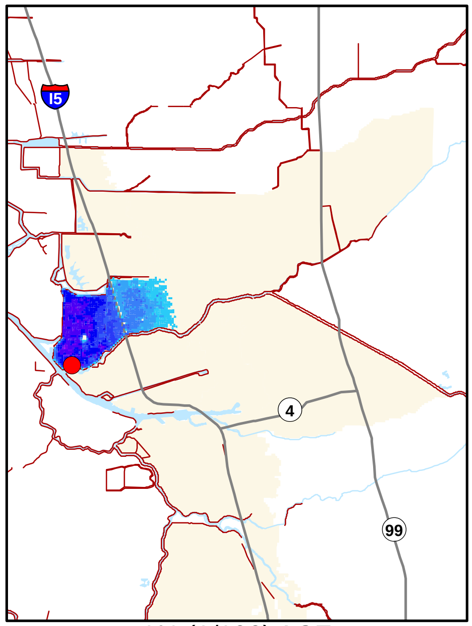
10% (1/10) ACE



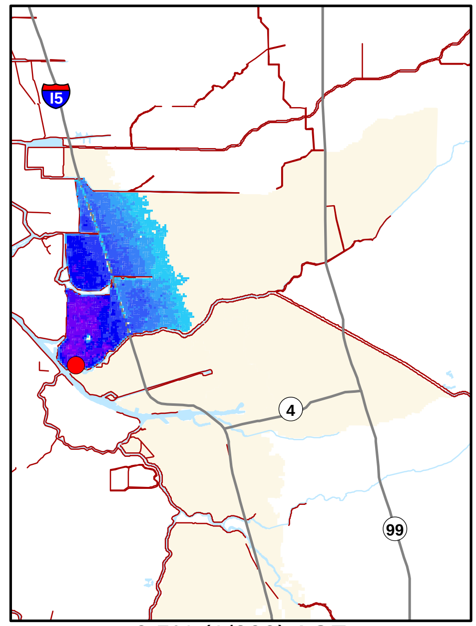
4% (1/25) ACE



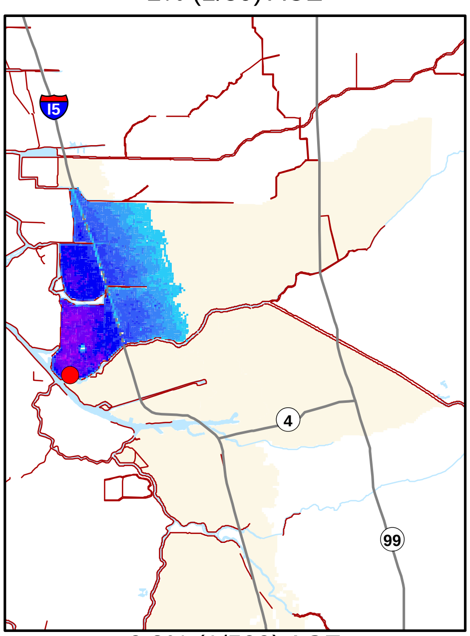
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-D5

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

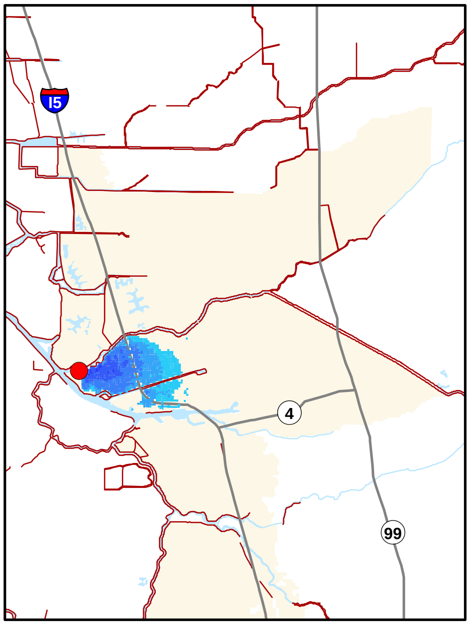
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-D4

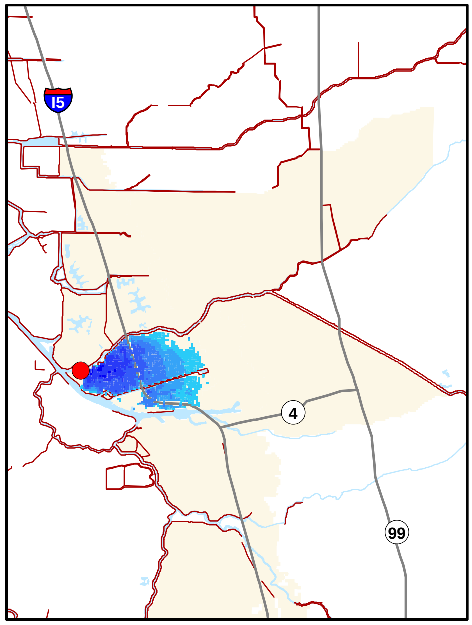
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

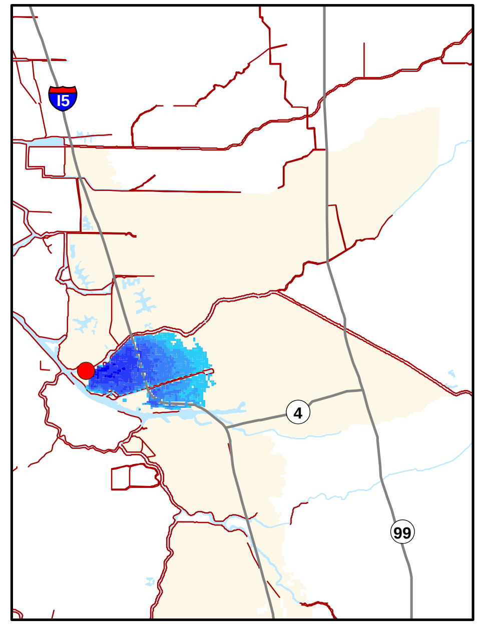
JUN 2014



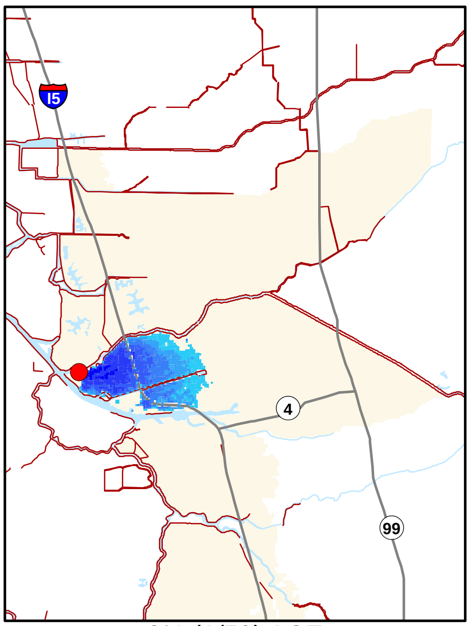
50% (1/2) ACE



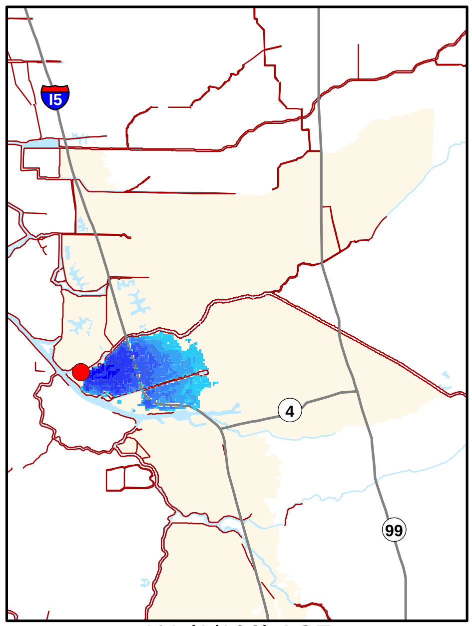
10% (1/10) ACE



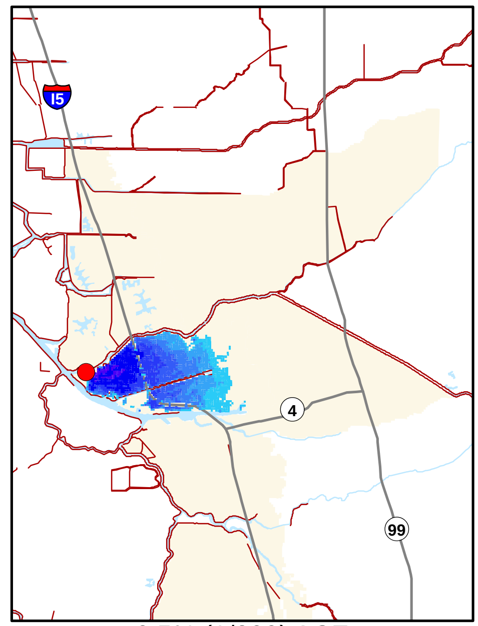
4% (1/25) ACE



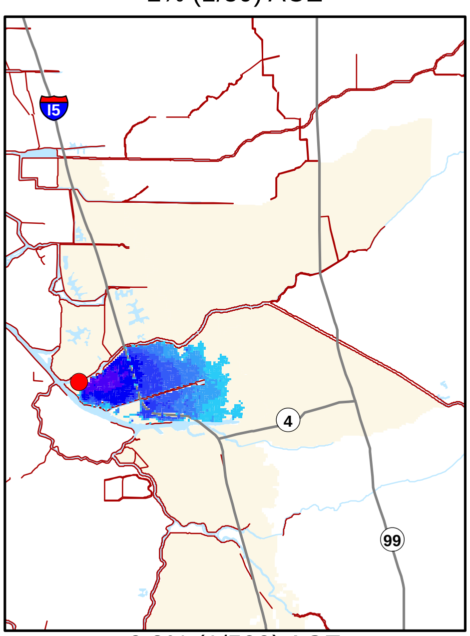
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Breach Location B-D5

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

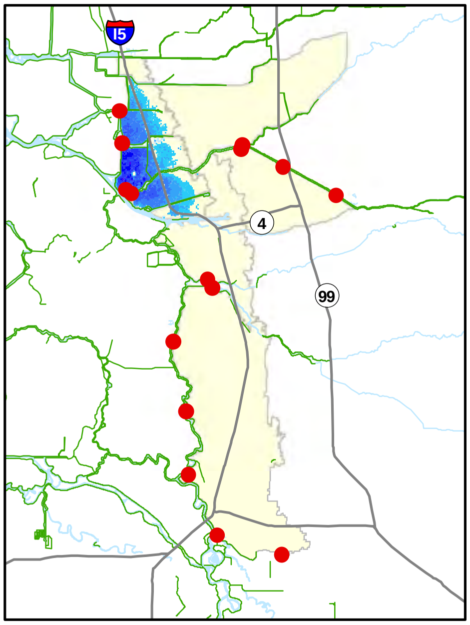
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-D5

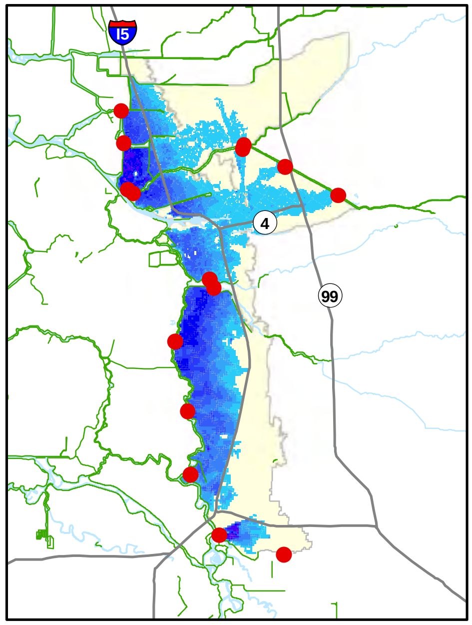
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

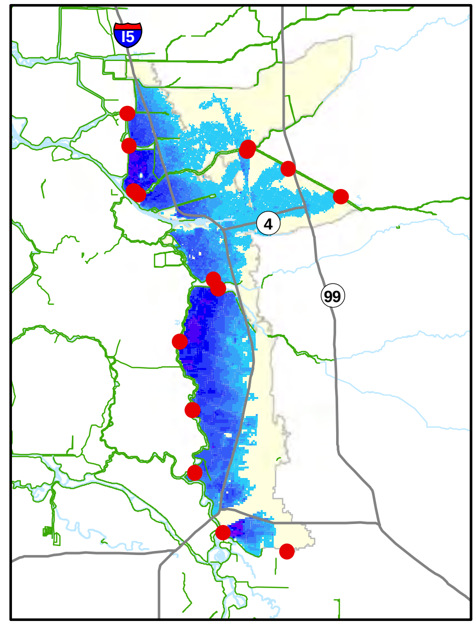
JUN 2014



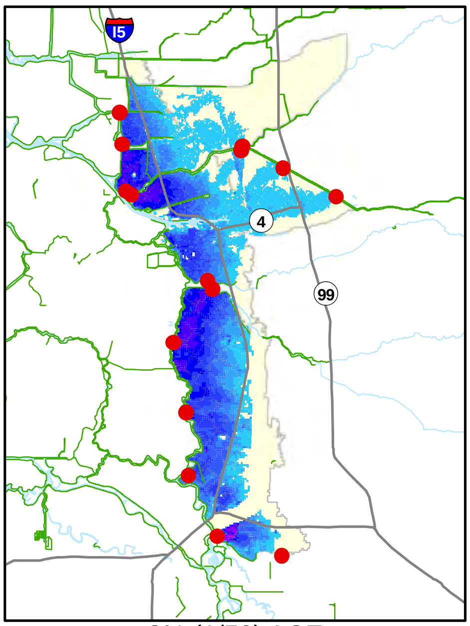
50% (1/2) ACE



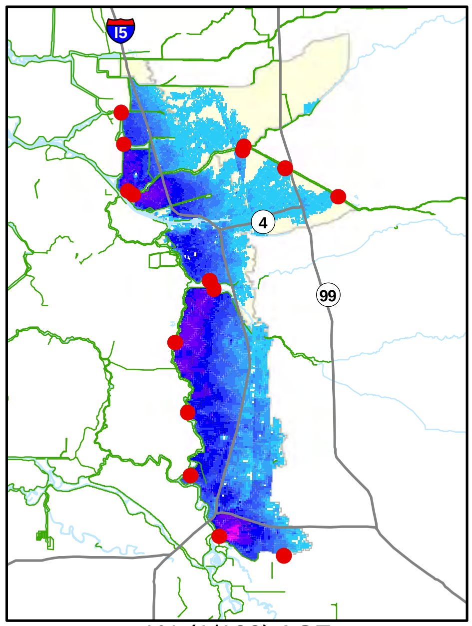
10% (1/10) ACE



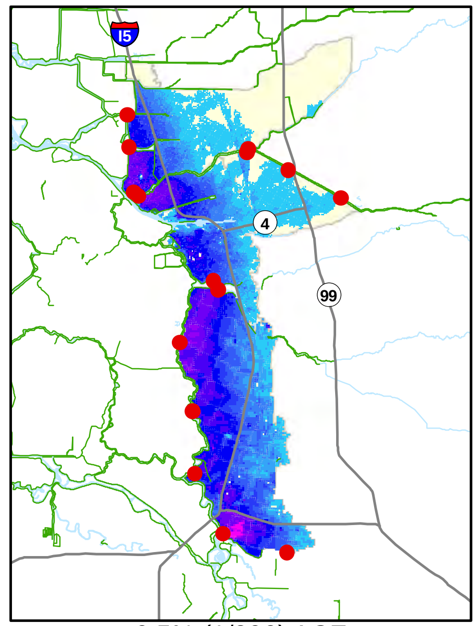
4% (1/25) ACE



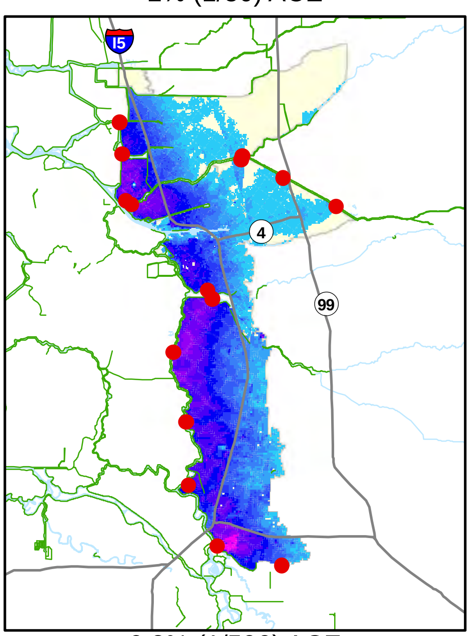
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Levee Breach Included

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- > 22.0
- Levees (CLD, NLD)
- LSJ Study Extent

Miles
0 5

N

NOTE: All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within Study Extent

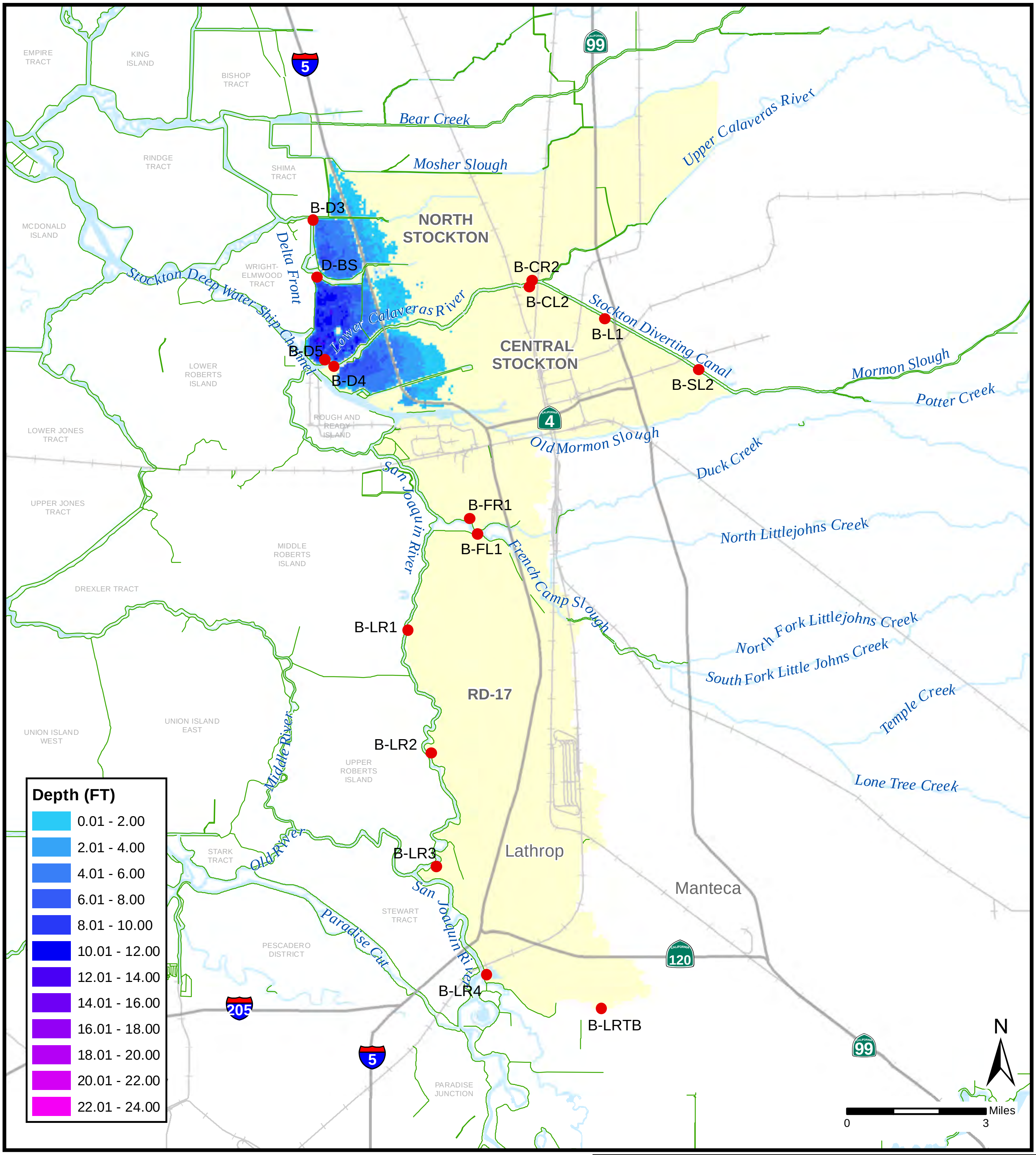
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE - 1
NO ACTION

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

OCT 2015



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

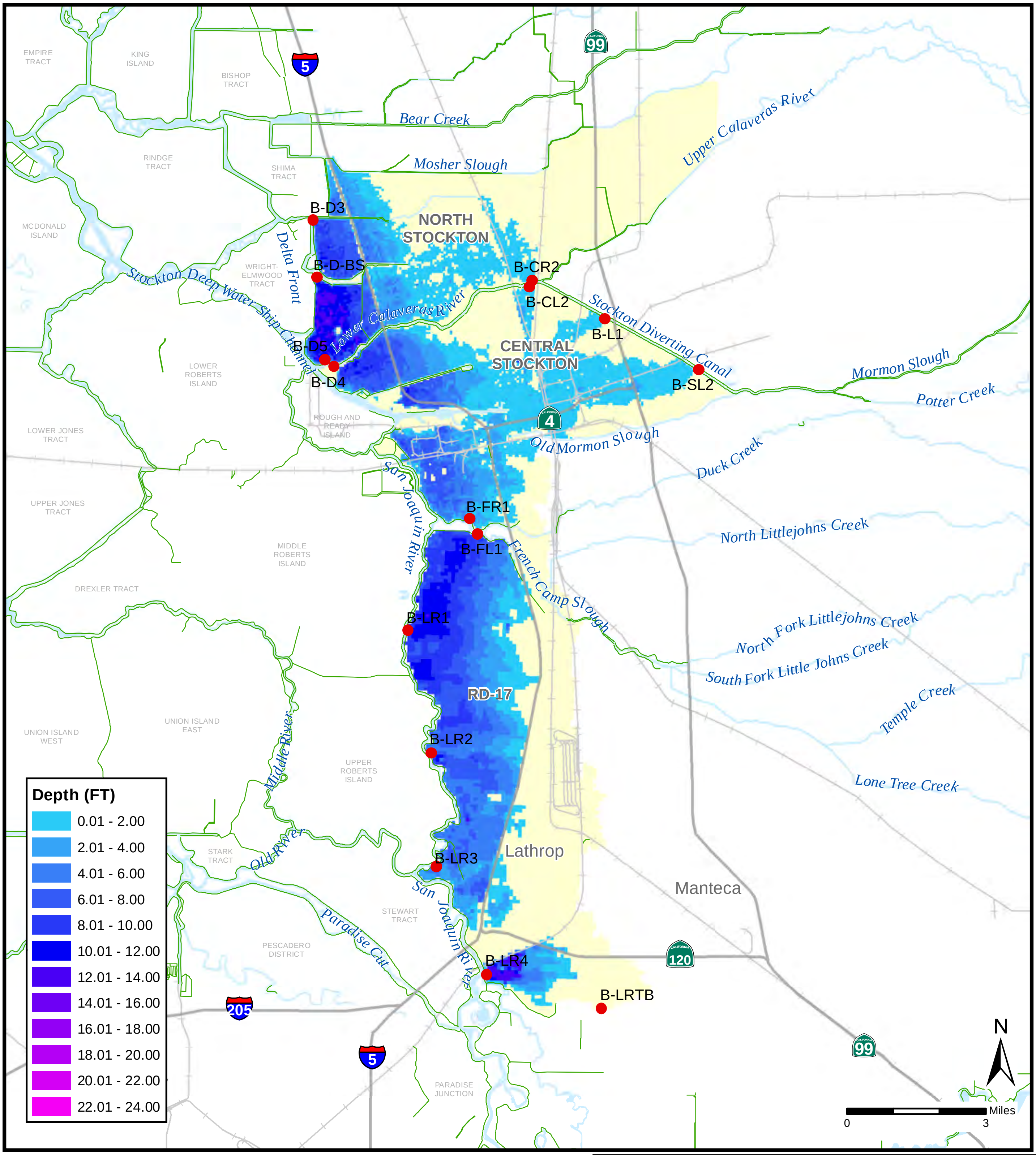
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

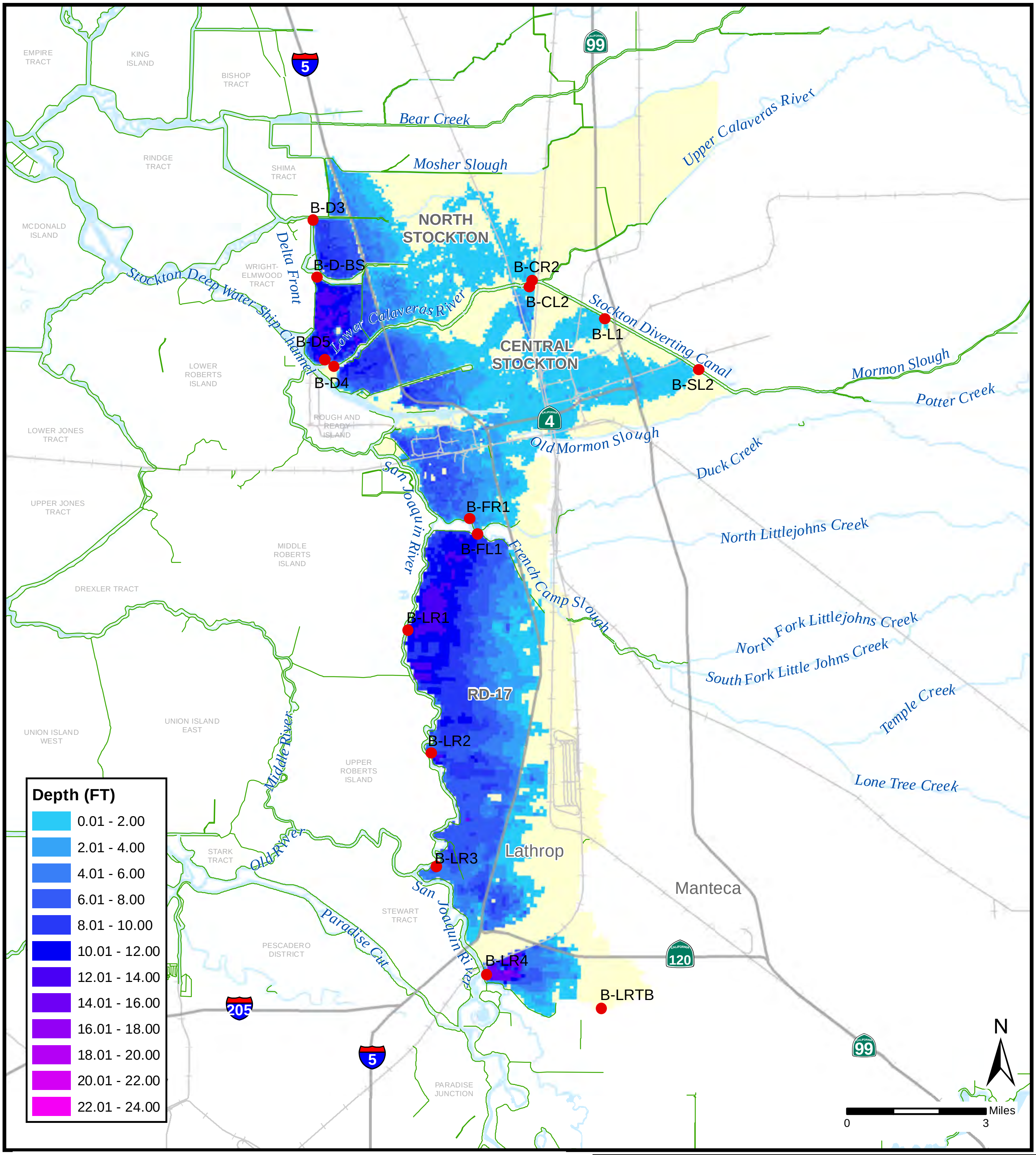
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN ALTERNATIVE 1 - NO ACTION 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

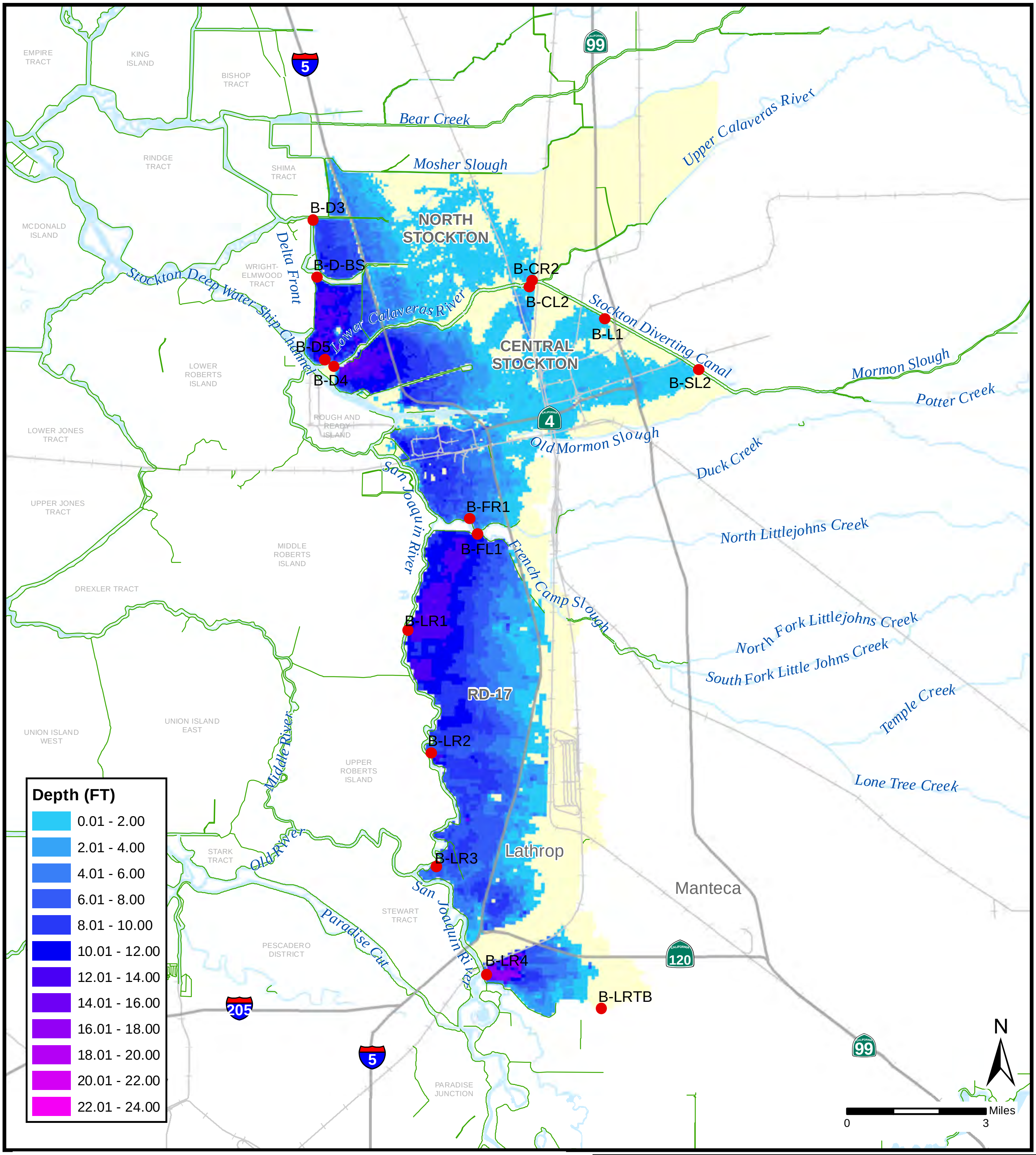
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN ALTERNATIVE 1 - NO ACTION 4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

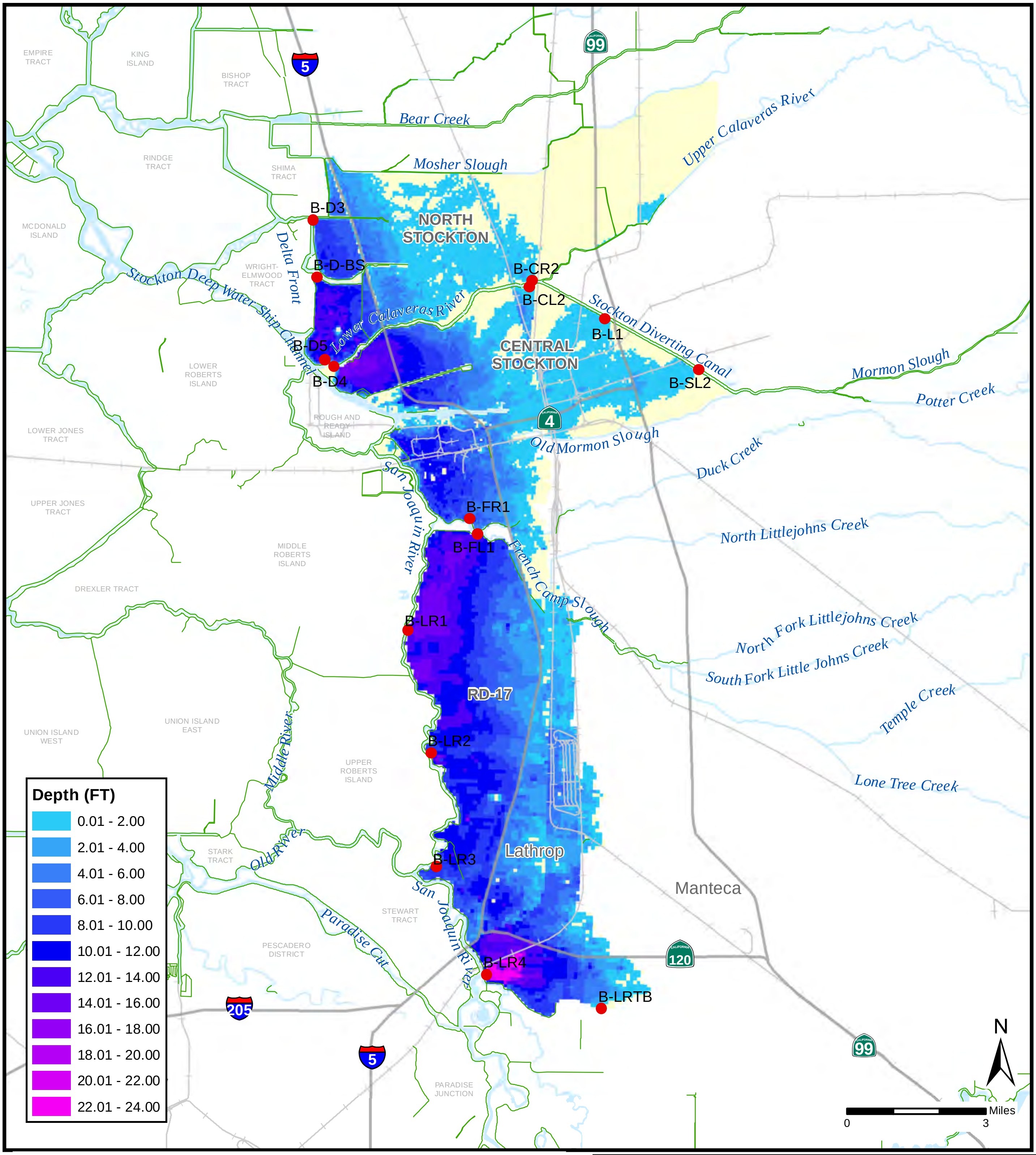
Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

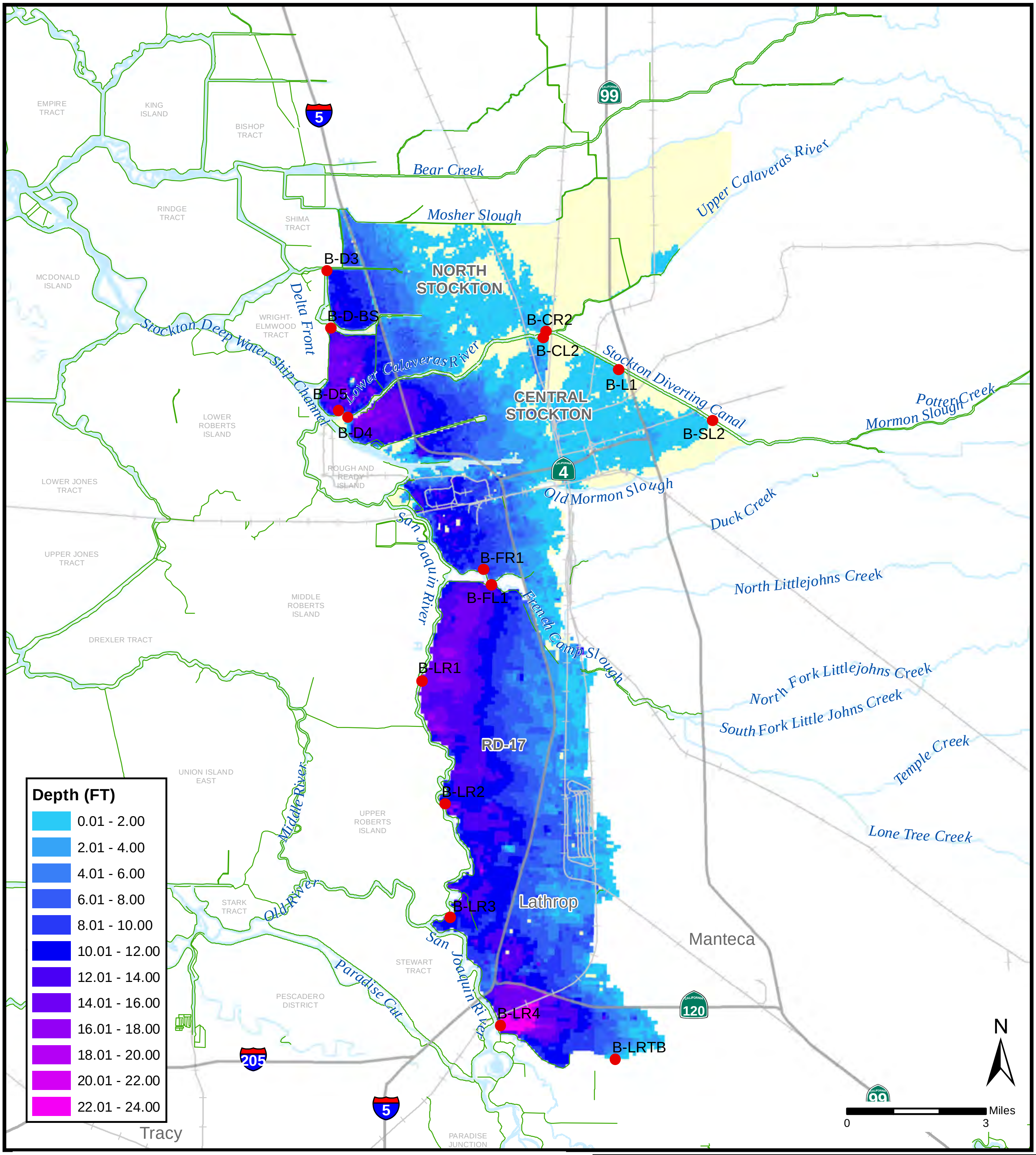
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

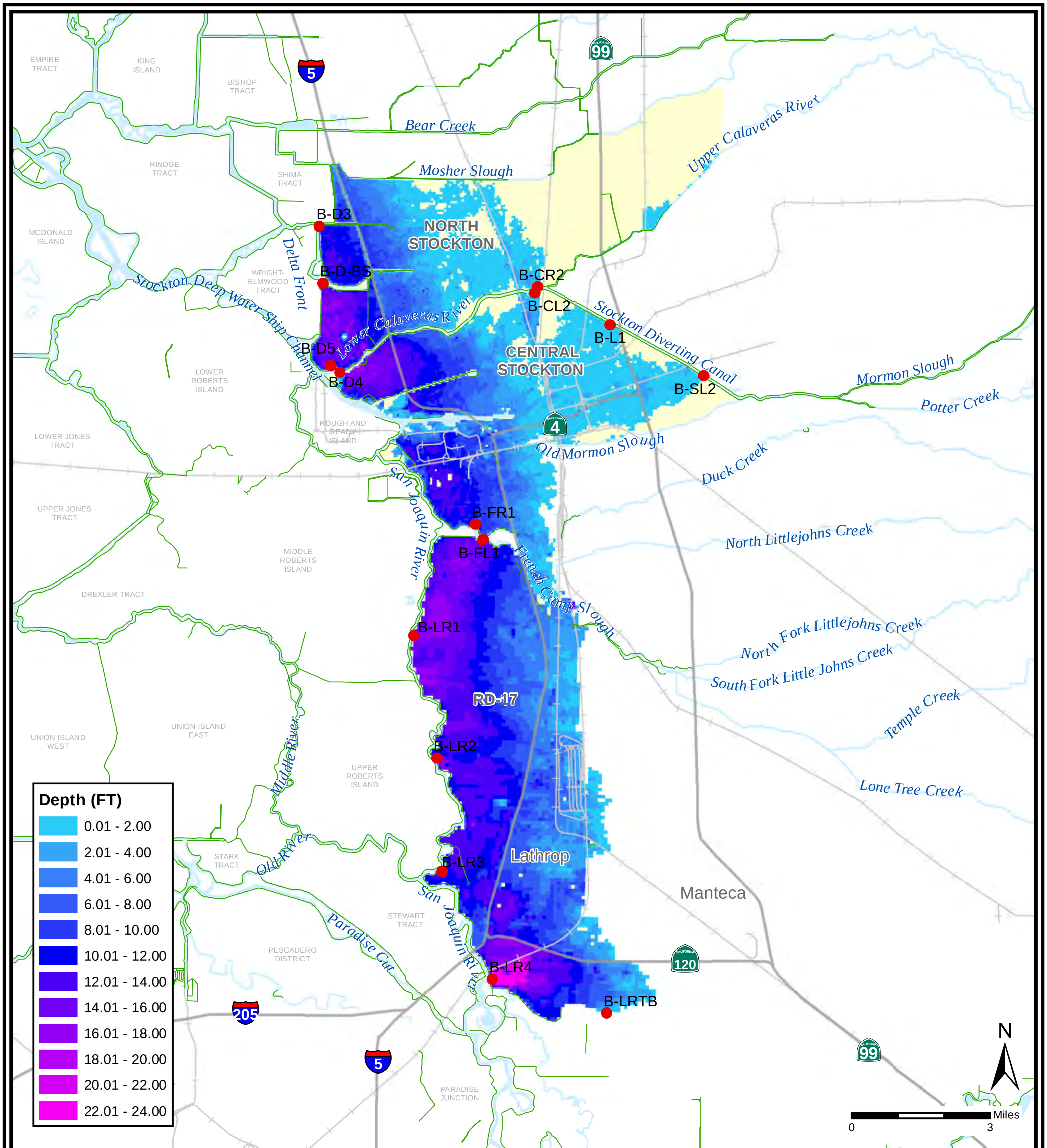
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

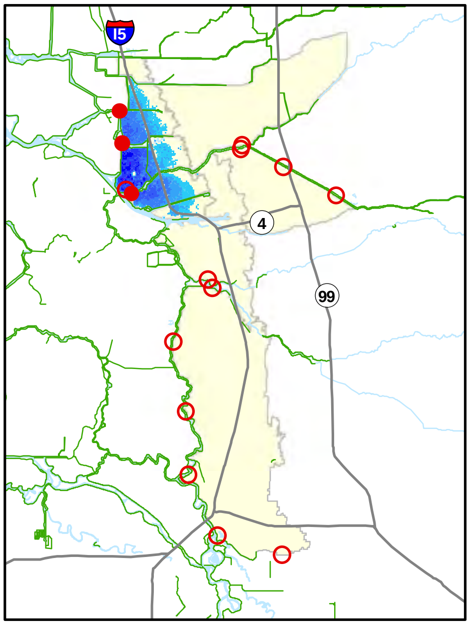
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
0.2% (1/500) ACE

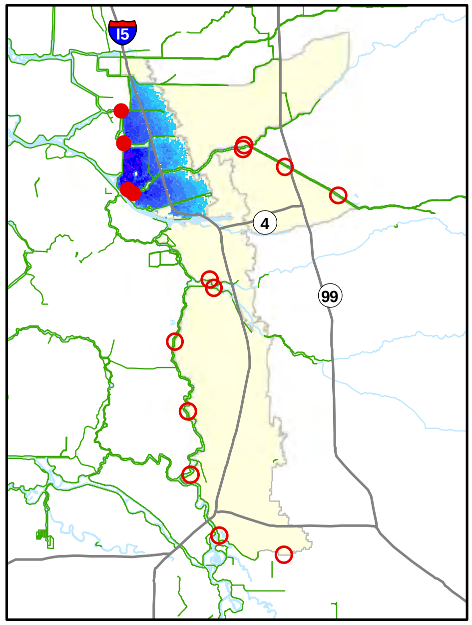
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

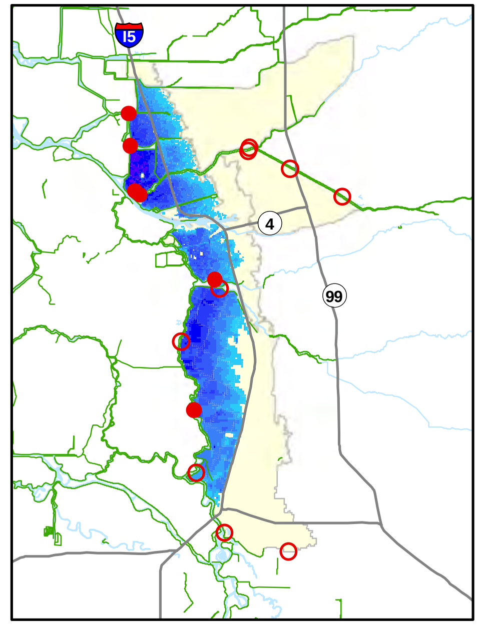
DEC 2015



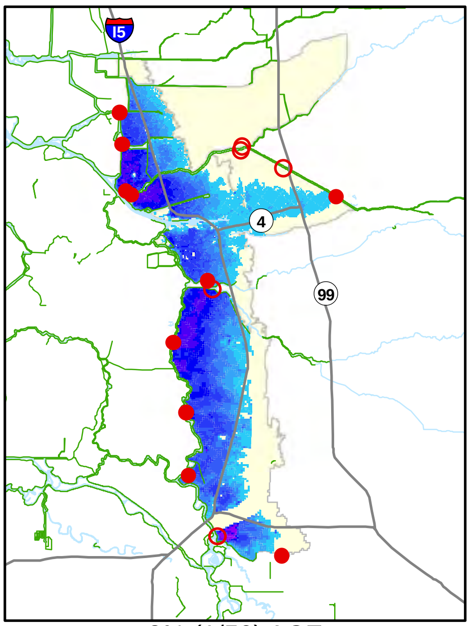
50% (1/2) ACE



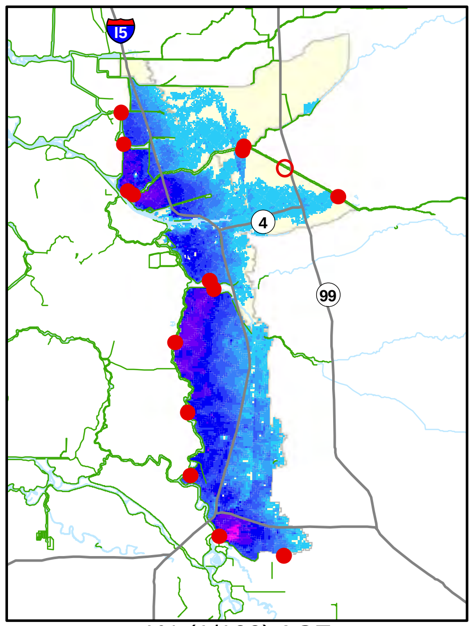
10% (1/10) ACE



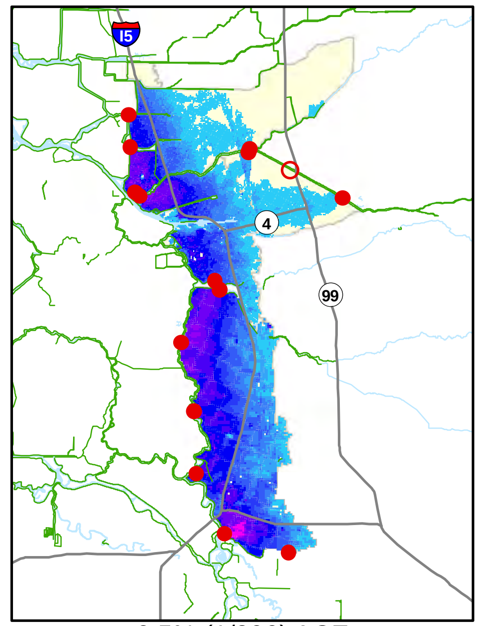
4% (1/25) ACE



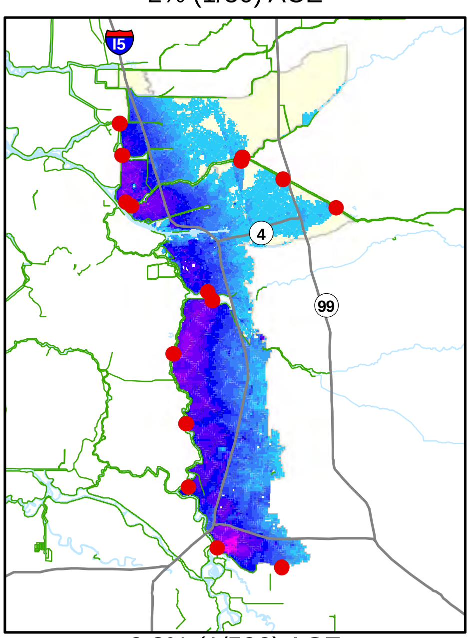
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

- Fails R&U Criteria
- Meets R&U Criteria

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Study Extent

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

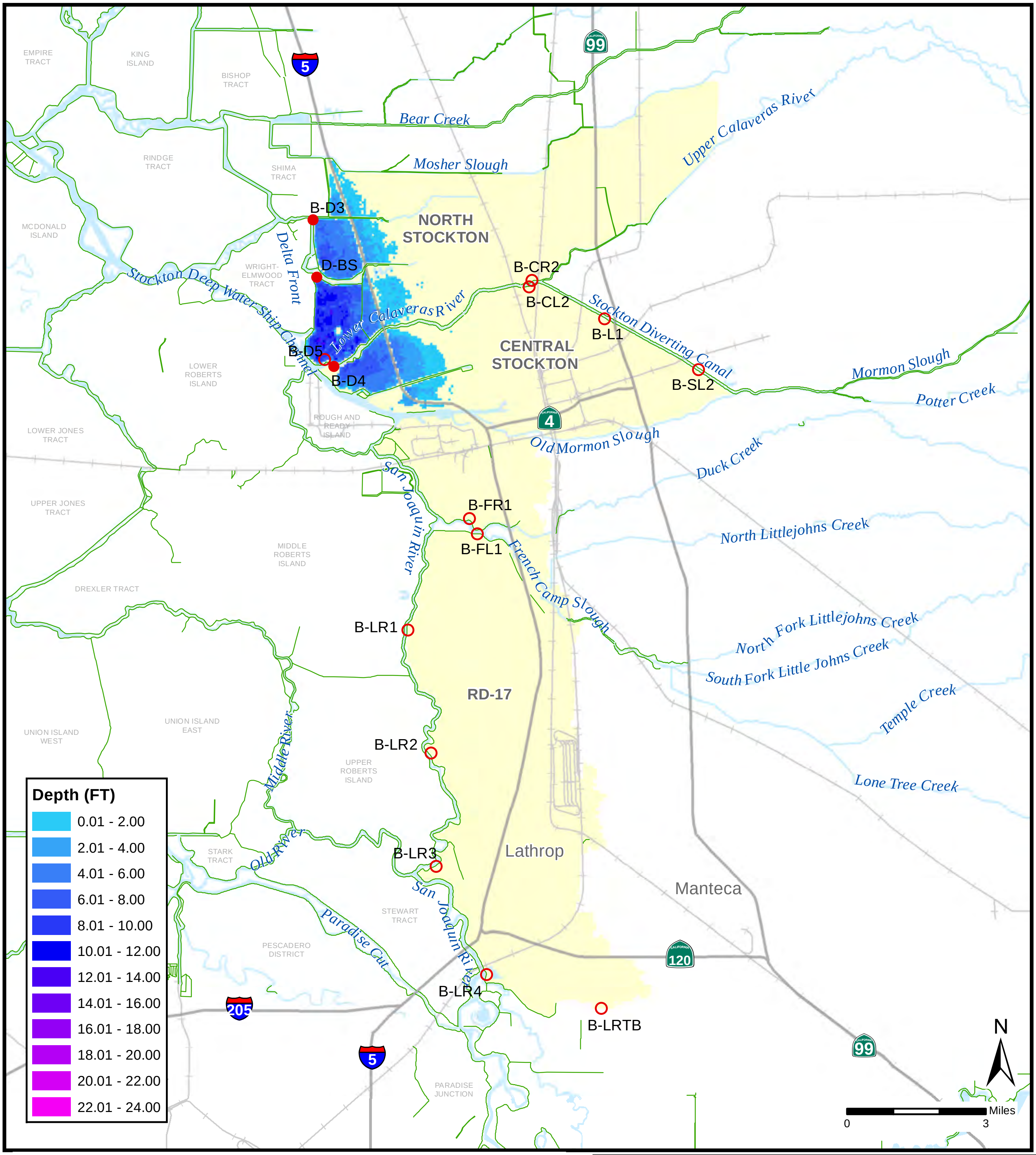
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 1 NO ACTION

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

DEC 2015



Legend

- Levees (CLD, NLD)
- Highway
- Railroads
- Fails R&U Criteria
- Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

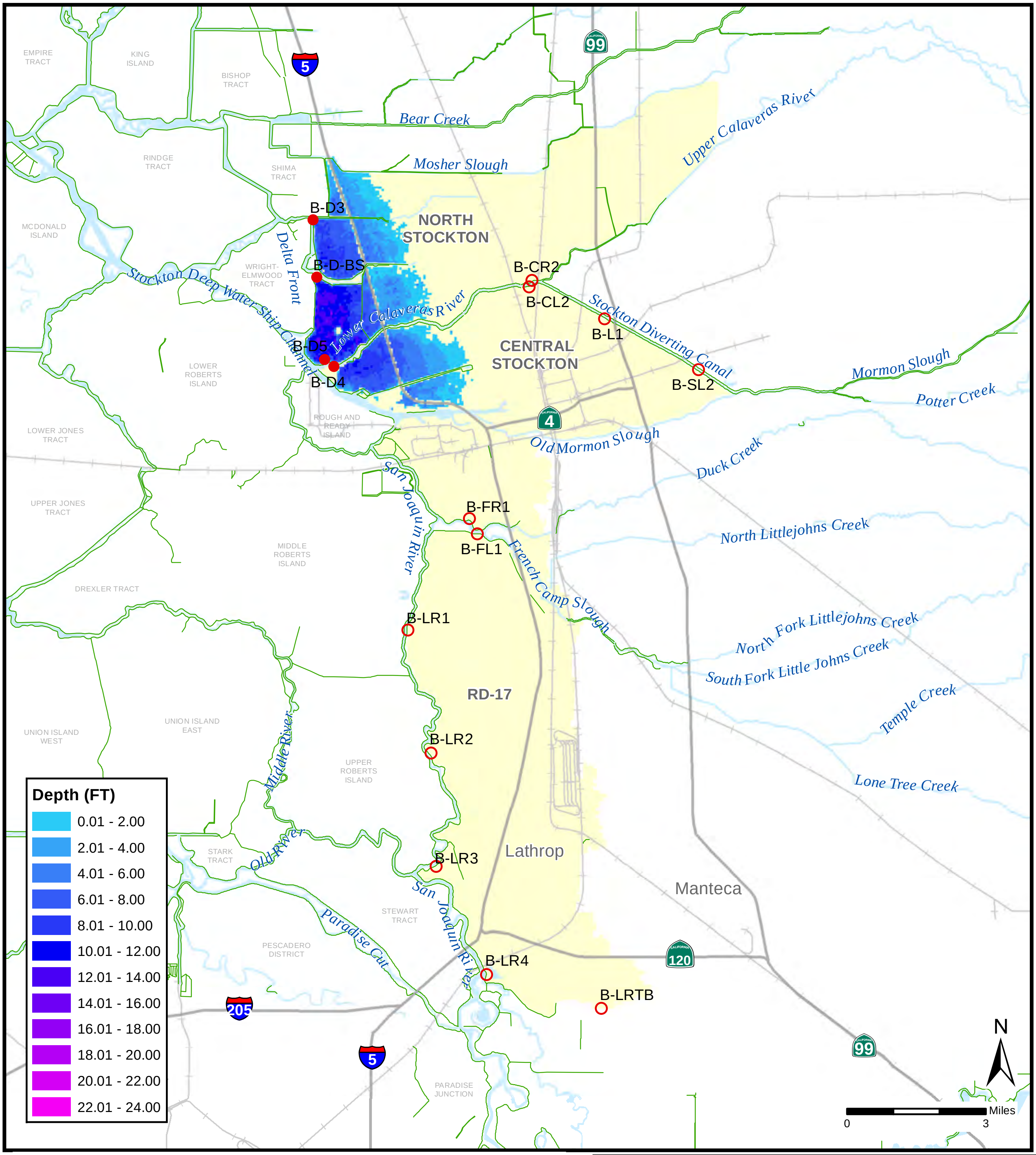
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



Legend

- Levees (CLD, NLD)
- Highway
- Railroads
- Fails R&U Criteria
- Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

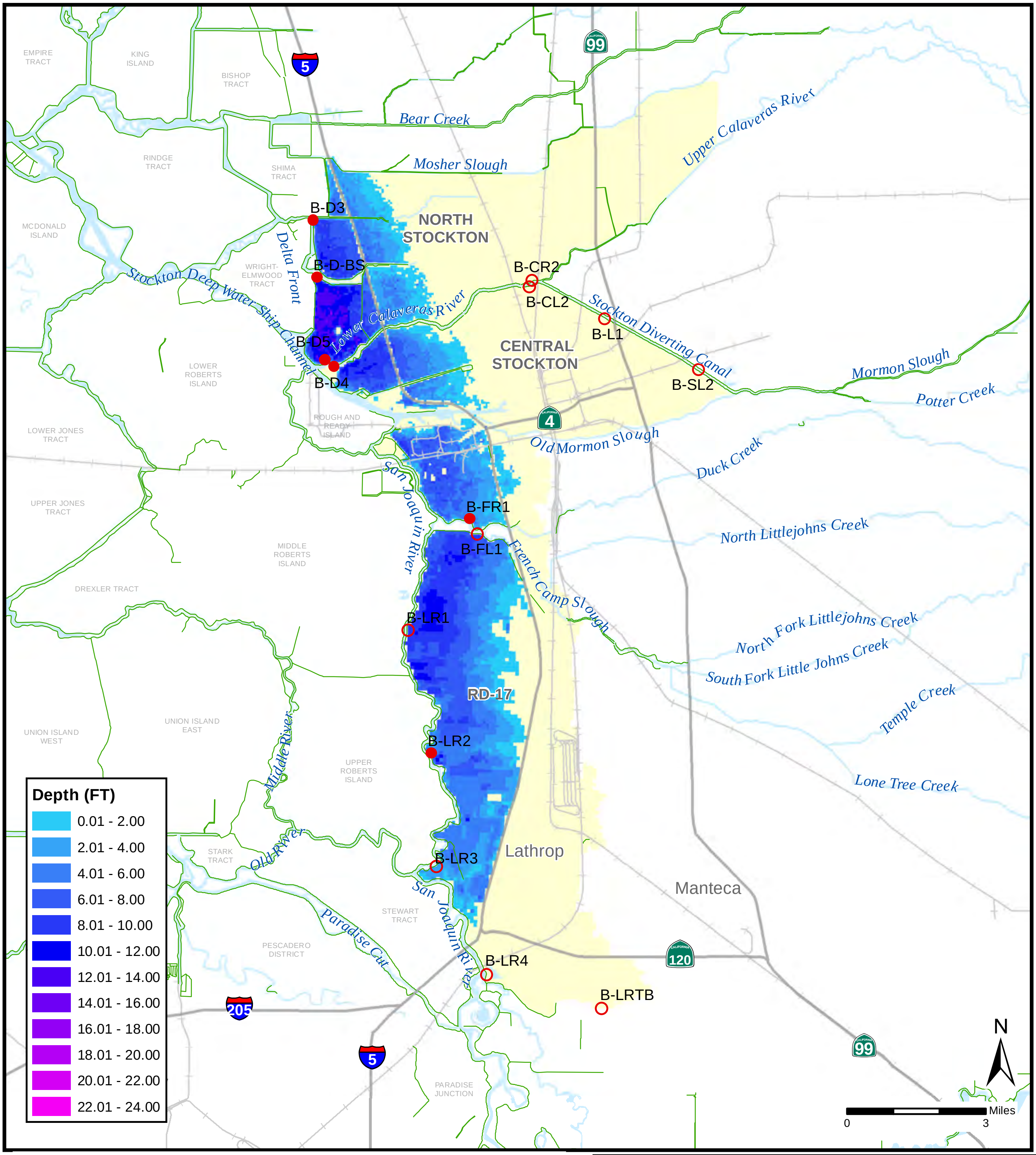
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



Legend

- Levees (CLD, NLD)
- Highway
- Railroads
- Fails R&U Criteria
- Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

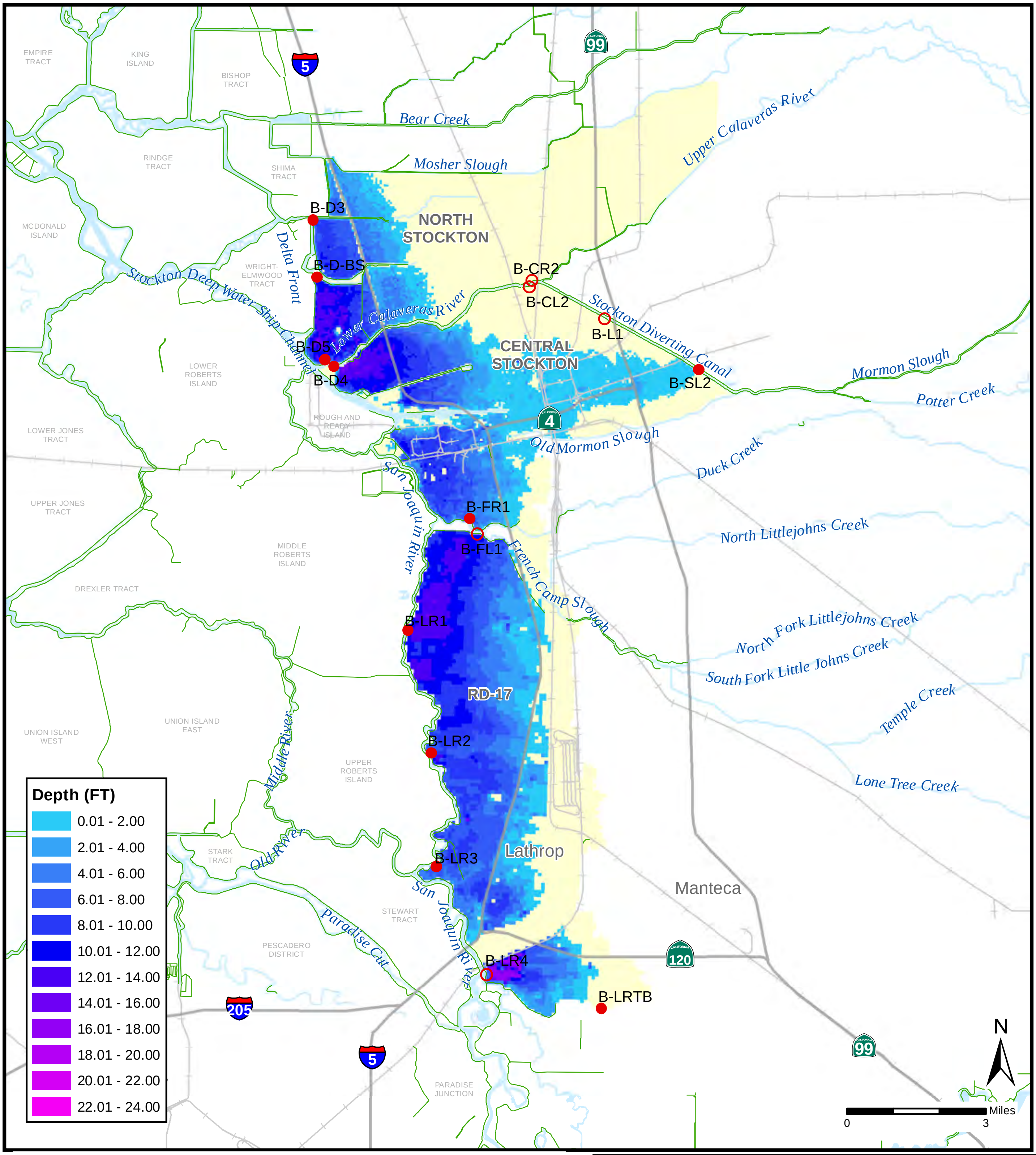
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



Legend

Levees (CLD, NLD)
Highway
Railroads

Fails R&U Criteria
Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

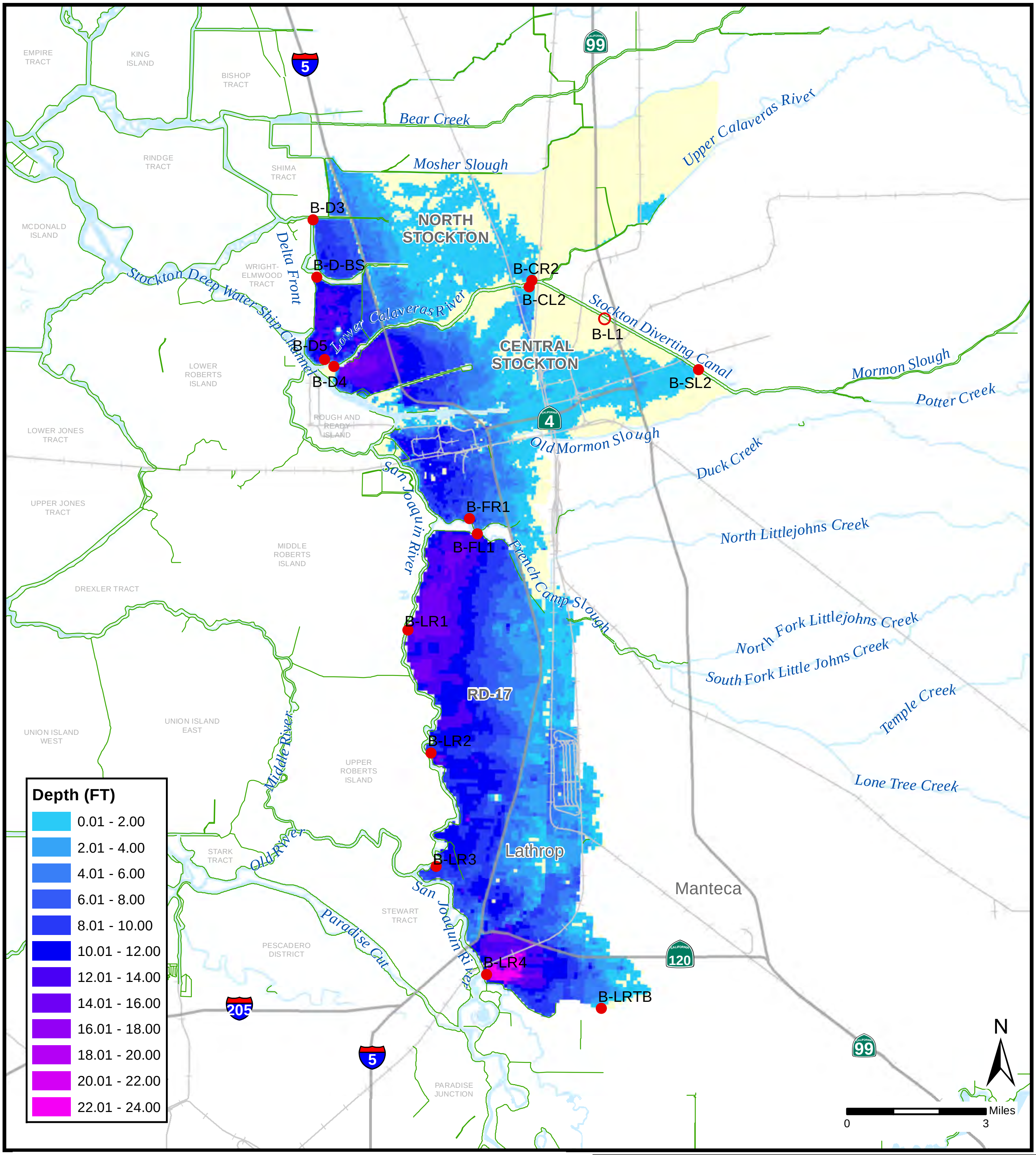
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



Legend

- Levees (CLD, NLD)
- Highway
- Railroads
- Fails R&U Criteria
- Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

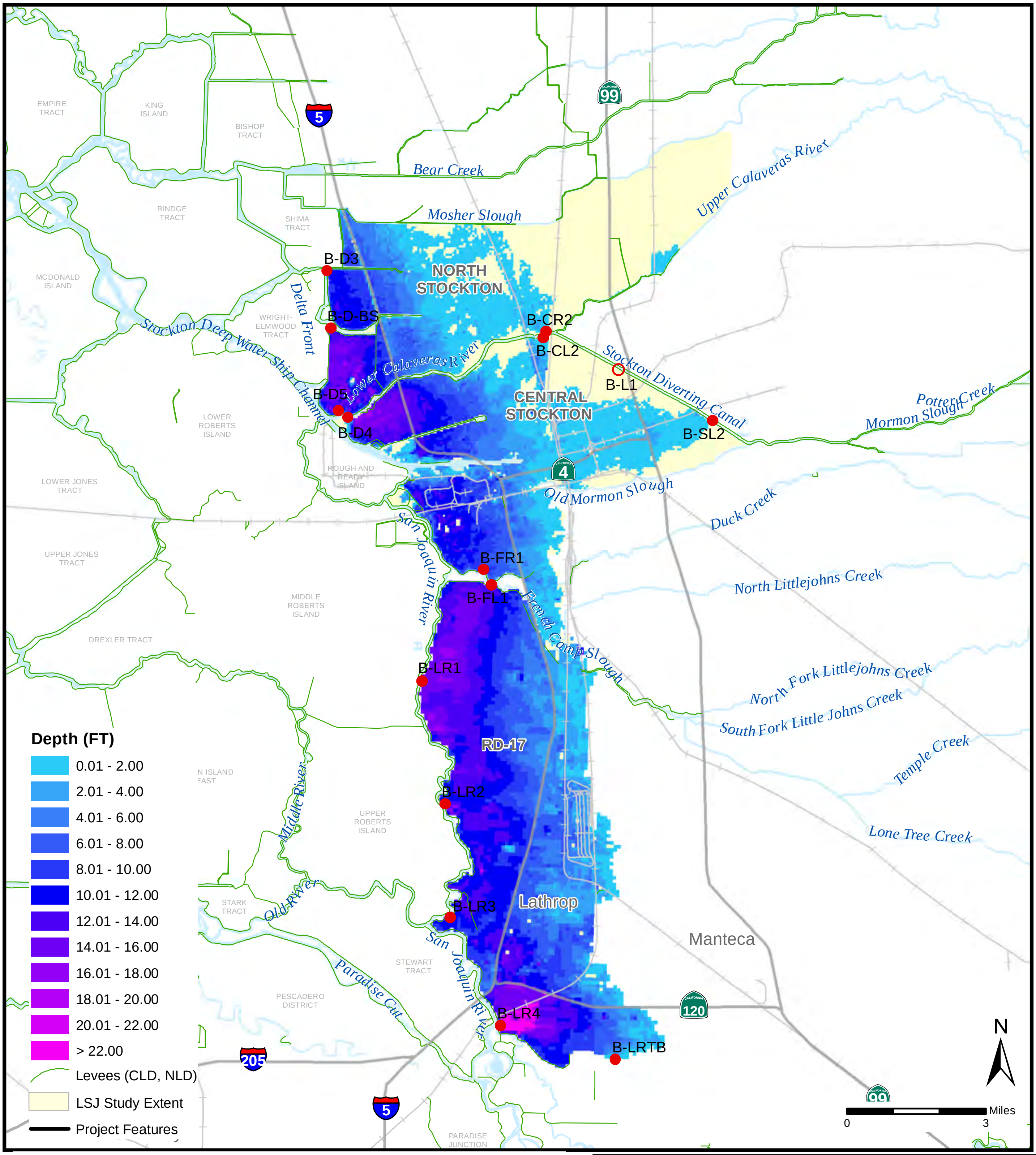
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE 1 - NO ACTION 1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



Legend

Levees (CLD, NLD)

Highway

Railroads

Fails R&U Criteria

Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

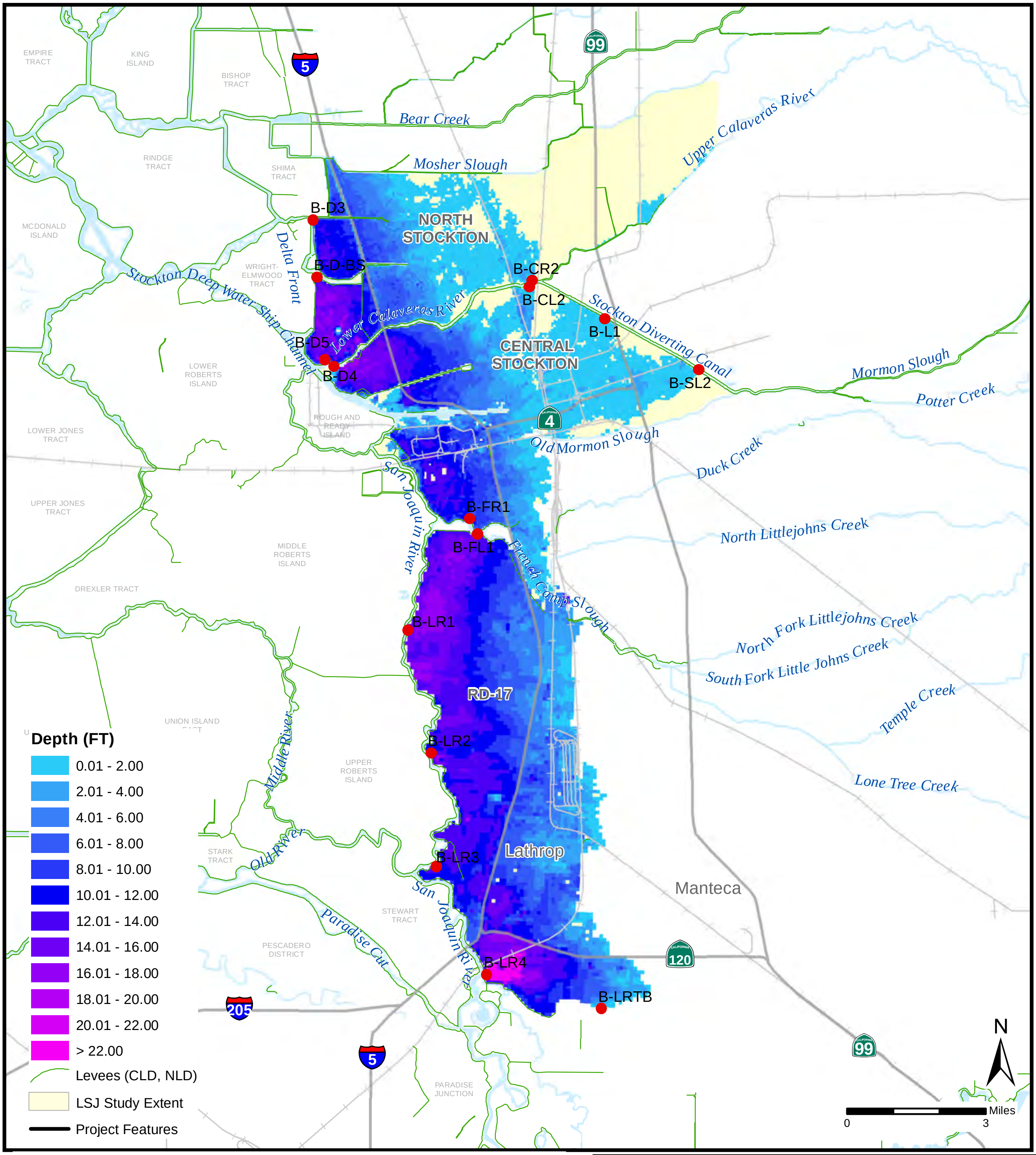
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE 1 - NO ACTION 0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

OCT 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

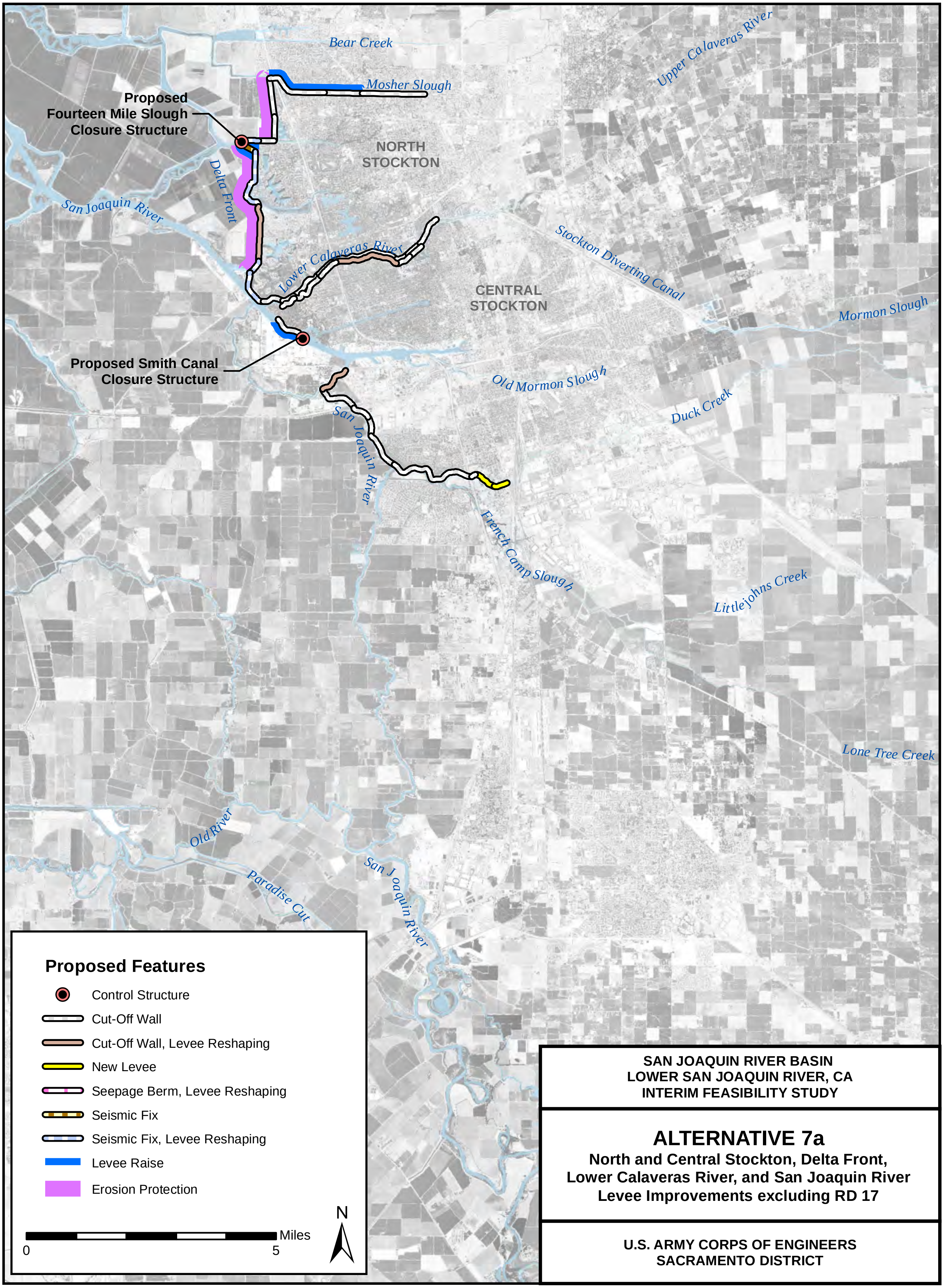
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE 1 - NO ACTION 0.2% (1/500) ACE

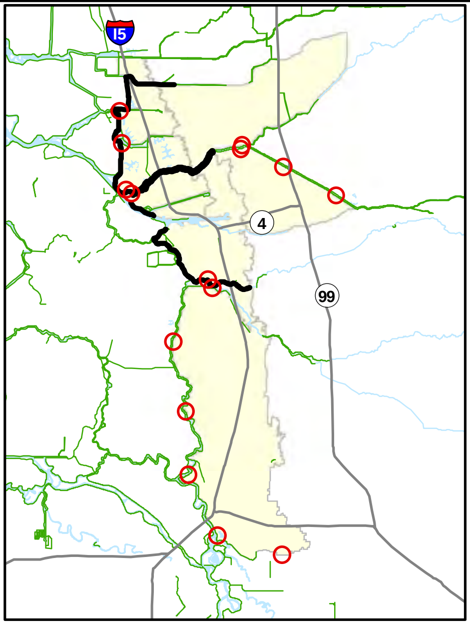
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

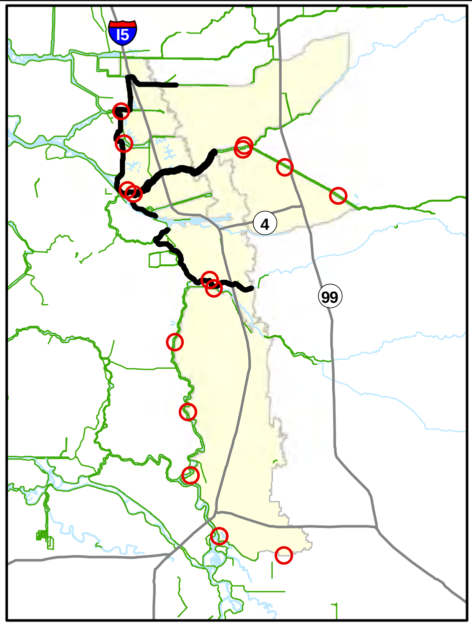
OCT 2014


Bear Creek
Mosher Slough
Upper Calaveras River
Proposed
Fourteen Mile Slough
Closure Structure
NORTH
STOCKTON
Delta Front
San Joaquin River
Lower Calaveras River
Stockton Diverting Canal
CENTRAL
STOCKTON
Mormon Slough
Proposed Smith Canal
Closure Structure
Old Mormon Slough
Duck Creek
San Joaquin River
French Camp Slough
Littlejohns Creek
Lone Tree Creek
Old River
Paradise Cut
San Joaquin River
Proposed Features
Control Structure
Cut-Off Wall
Cut-Off Wall, Levee Reshaping
New Levee
Seepage Berm, Levee Reshaping
Seismic Fix
Seismic Fix, Levee Reshaping
Levee Raise
Erosion Protection
0
5
Miles
N
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY
ALTERNATIVE 7a
North and Central Stockton, Delta Front,
Lower Calaveras River, and San Joaquin River
Levee Improvements excluding RD 17
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

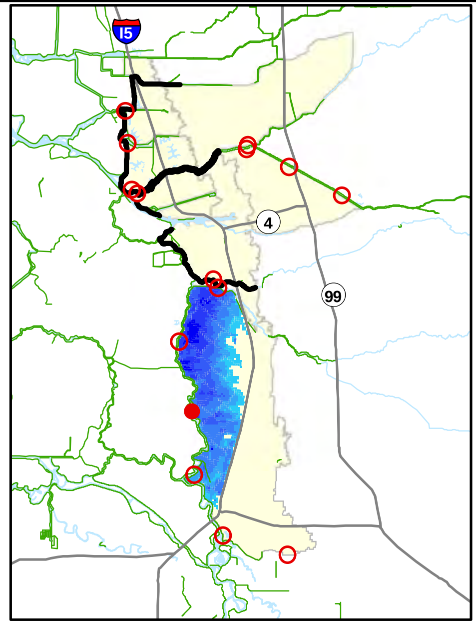
JUN 2014



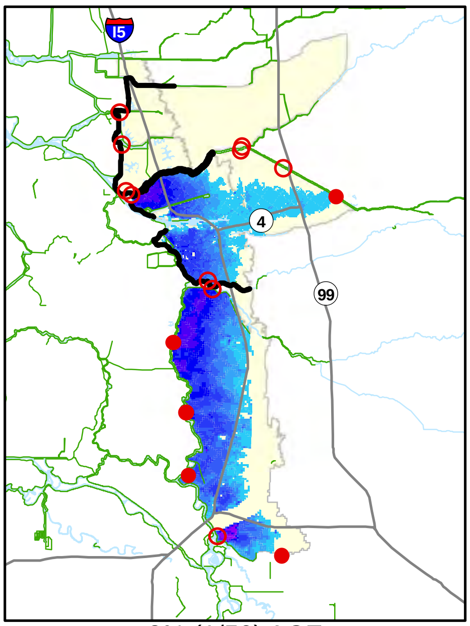
50% (1/2) ACE



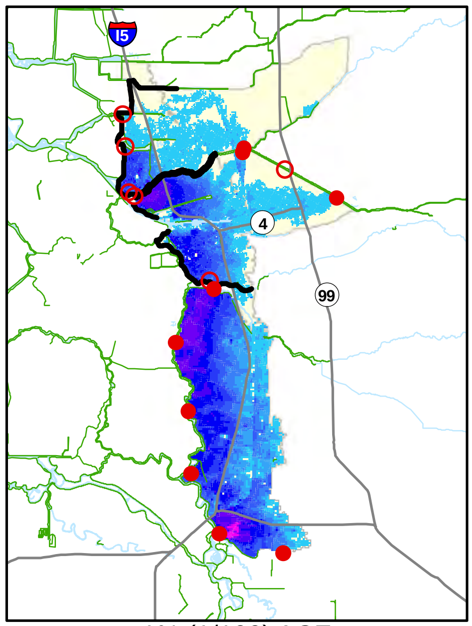
10% (1/10) ACE



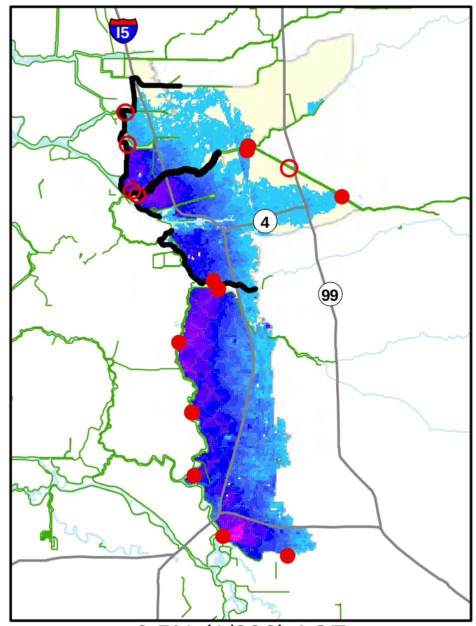
4% (1/25) ACE



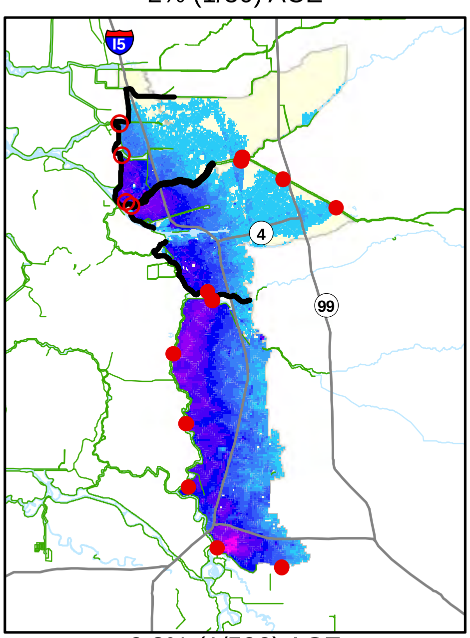
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Fails R&U Criteria
Meets R&U Criteria

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- > 22.0
- Levees (CLD, NLD)
- LSJ Study Extent

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

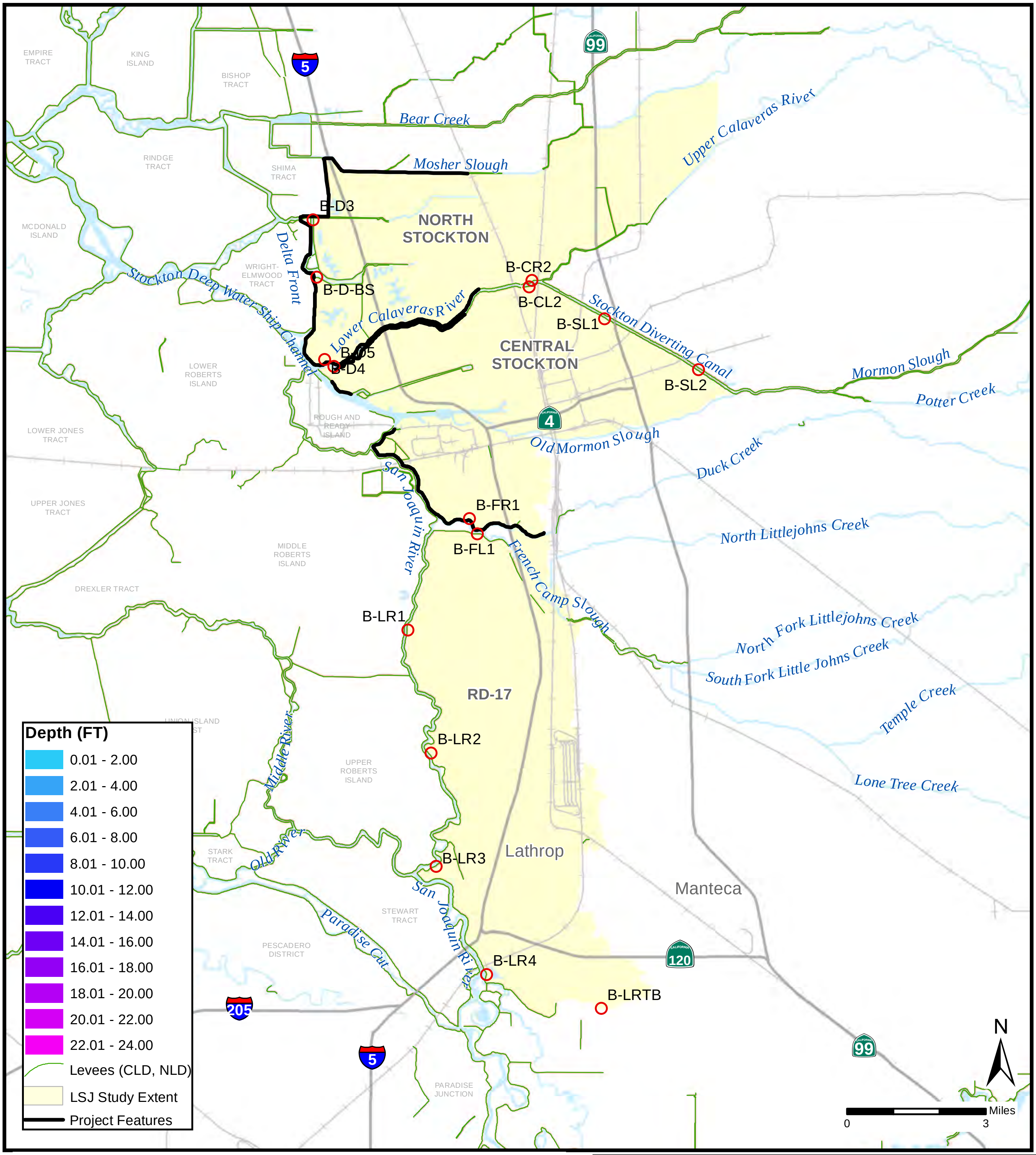
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7A

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

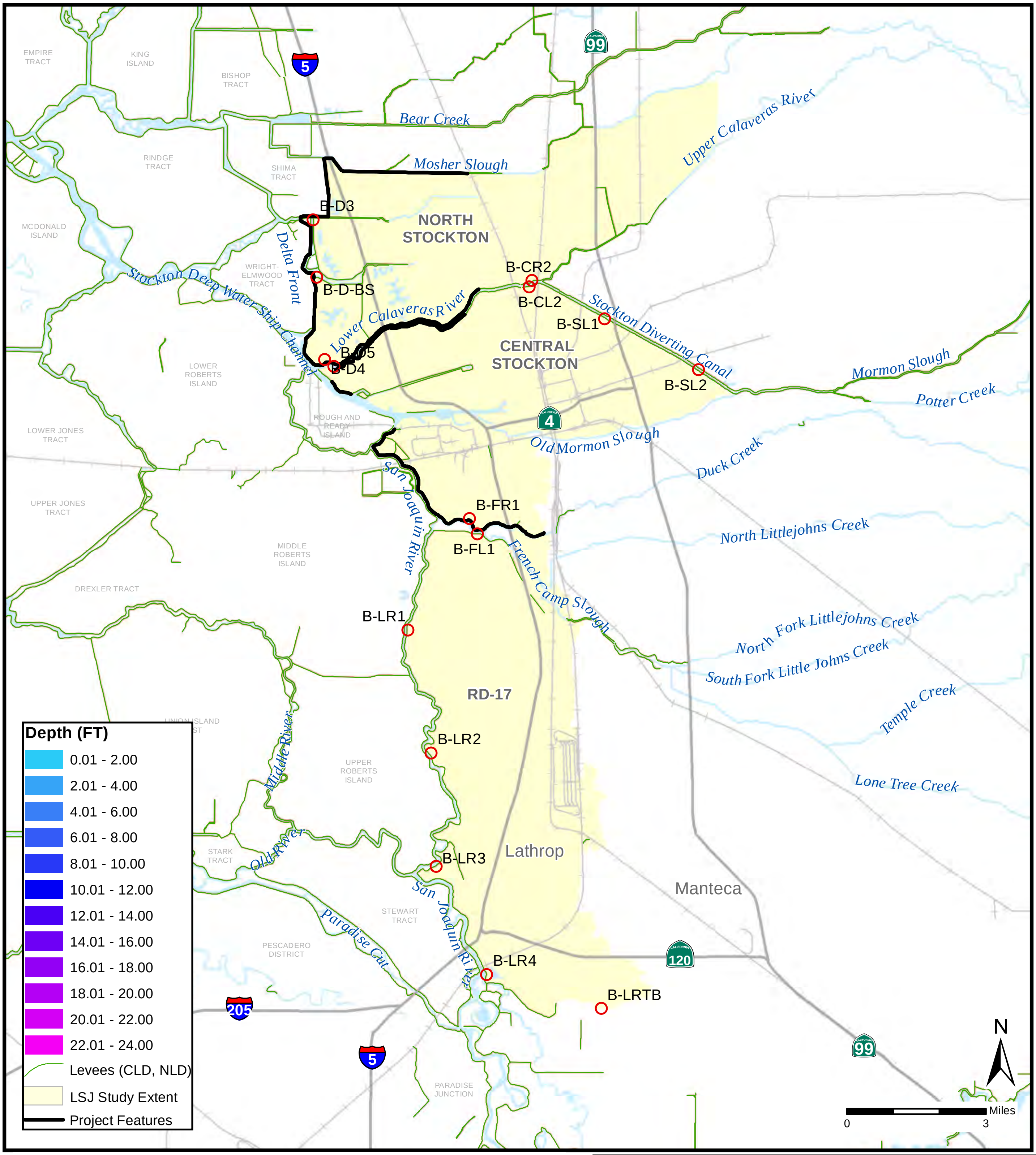
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7A 50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

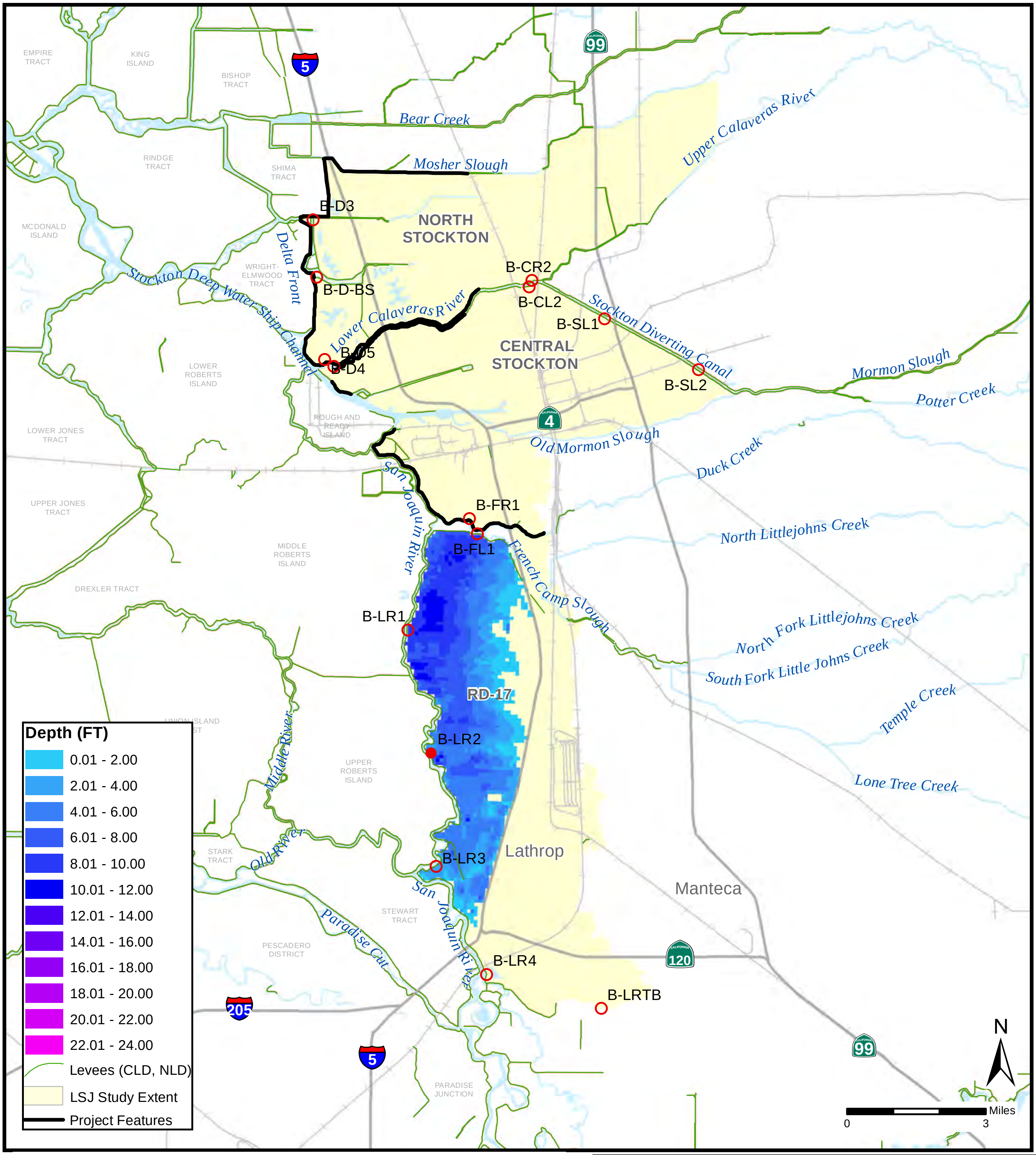
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7A 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

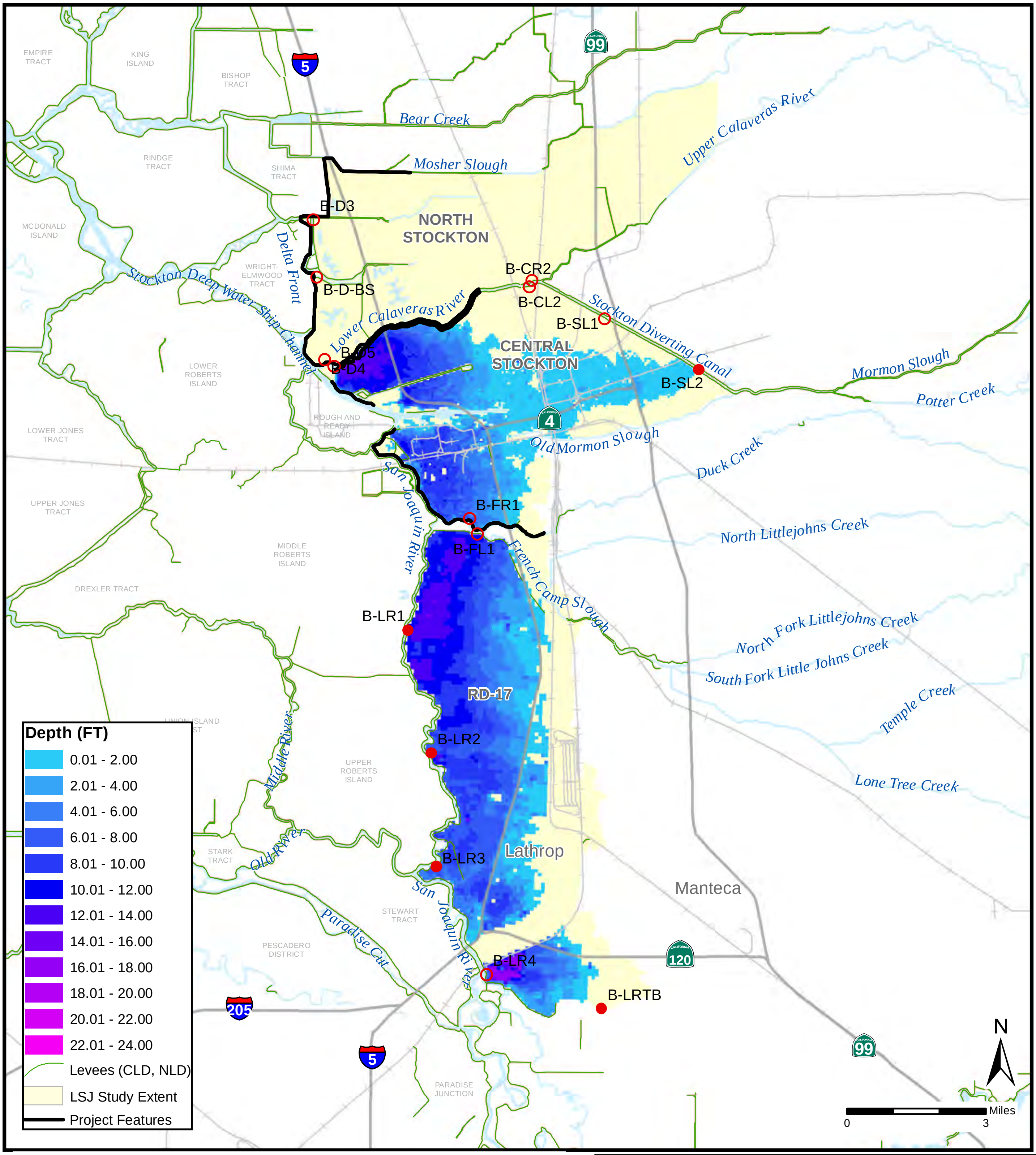
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7A
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

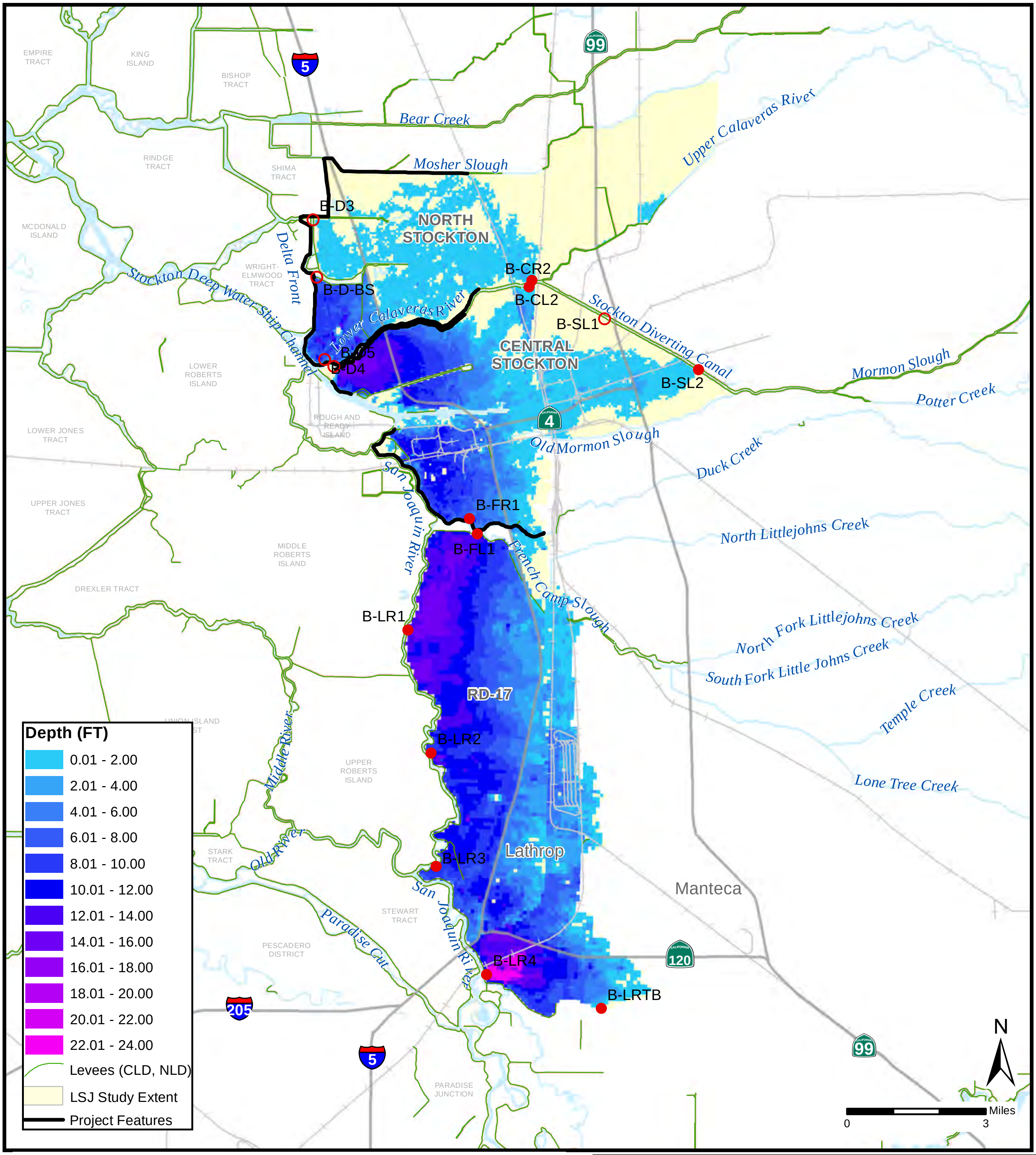
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7A 2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

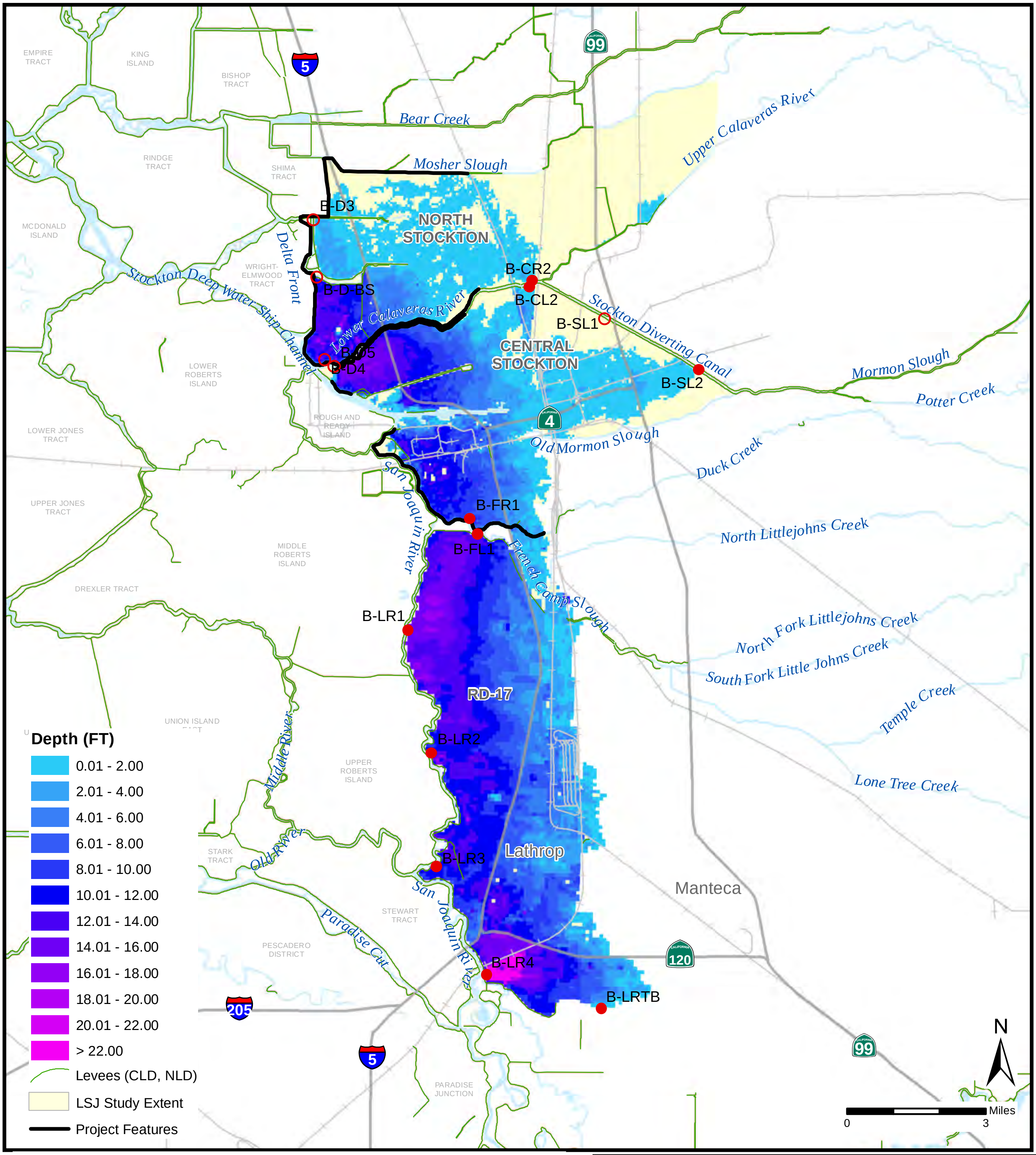
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7A 1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

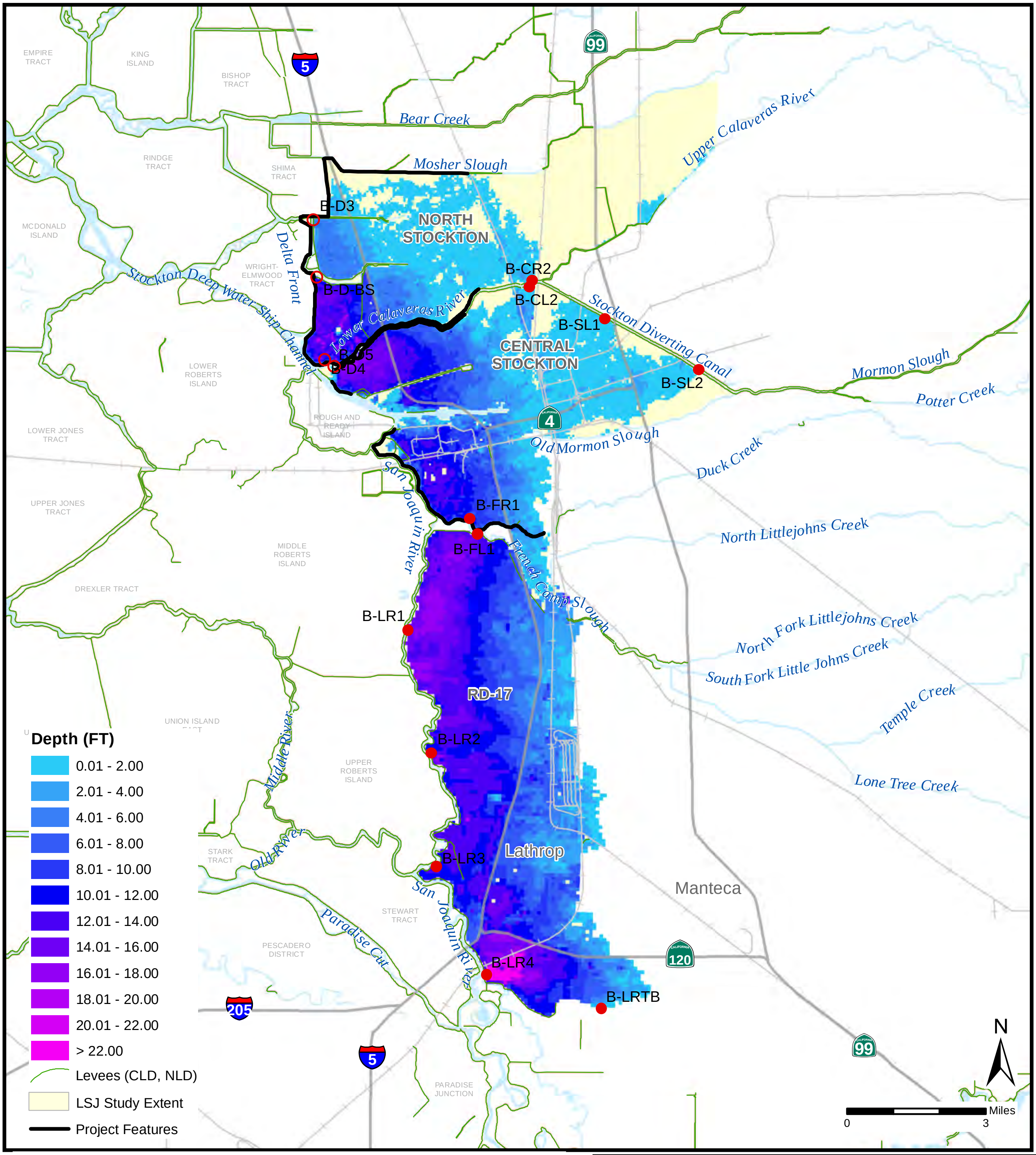
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7A 0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

OCT 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- ● Fails R&U Criteria
- ○ Meets R&U Criteria

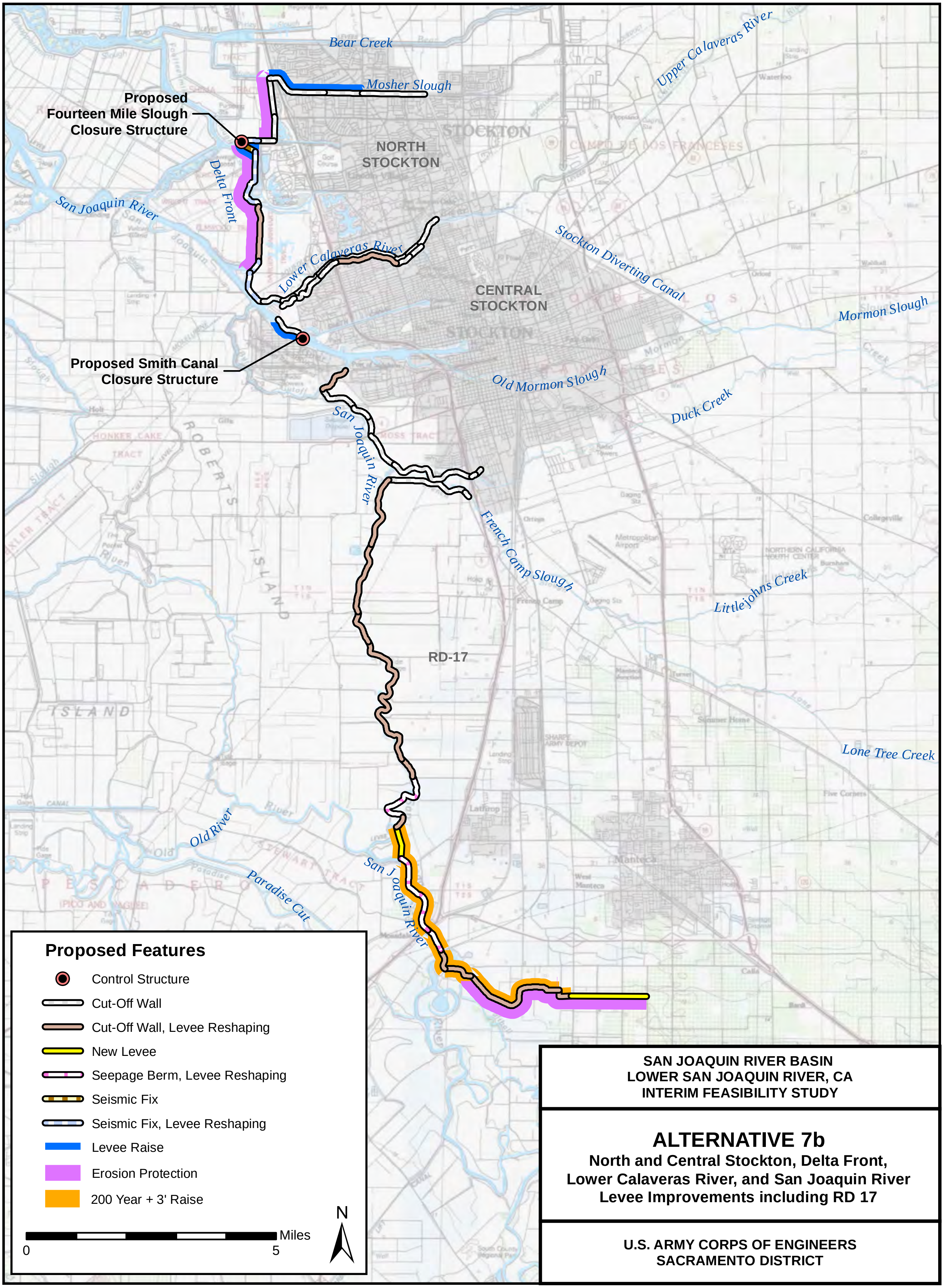
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7A 0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

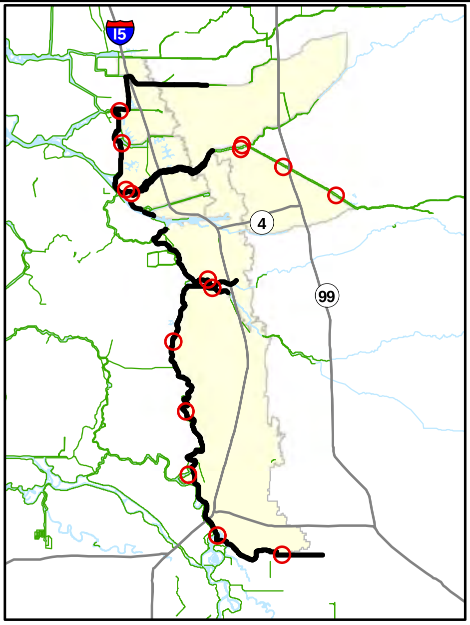
OCT 2014



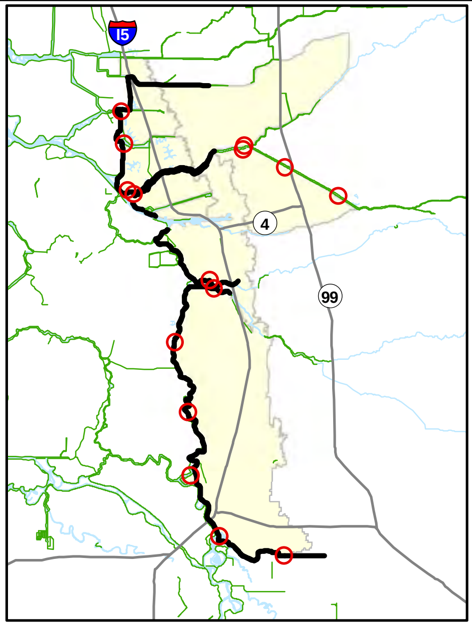
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

ALTERNATIVE 7b
North and Central Stockton, Delta Front, Lower Calaveras River, and San Joaquin River Levee Improvements including RD 17

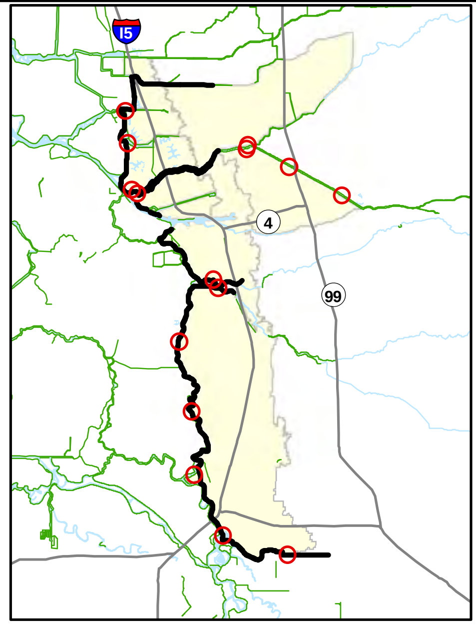
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



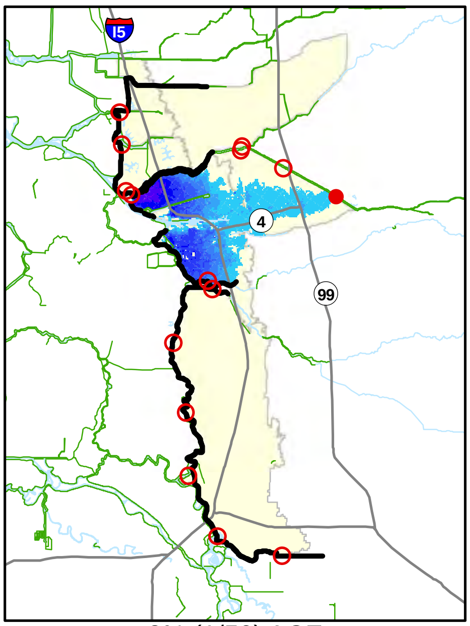
50% (1/2) ACE



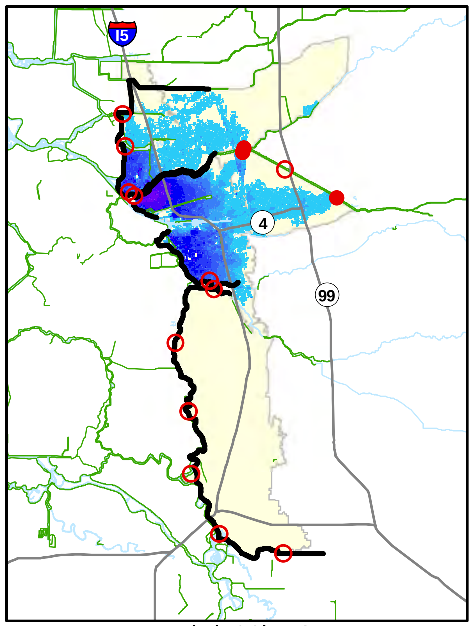
10% (1/10) ACE



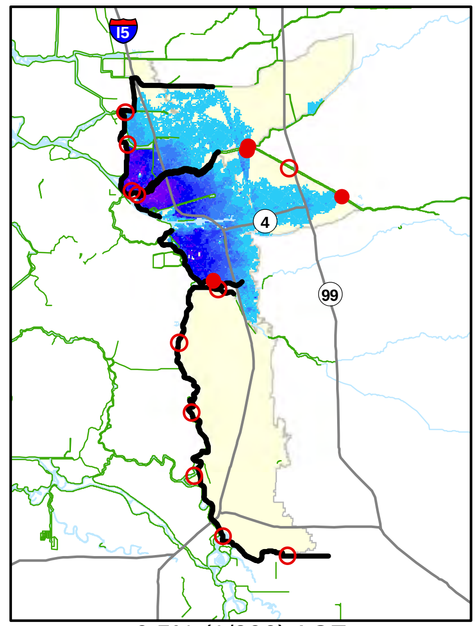
4% (1/25) ACE



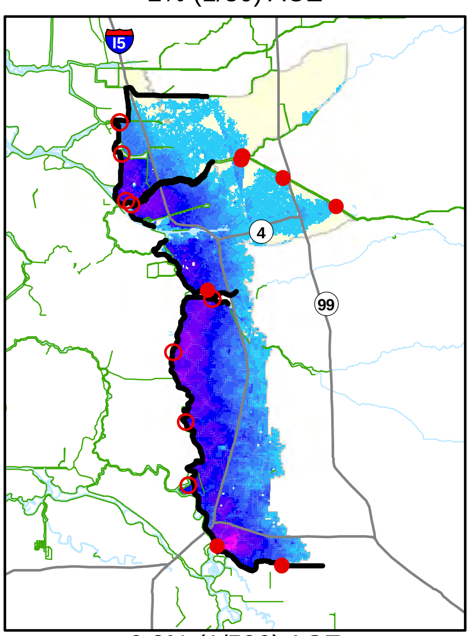
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Fails R&U Criteria

Meets R&U Criteria

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- > 22.0
- Levees (CLD, NLD)
- LSJ Study Extent

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

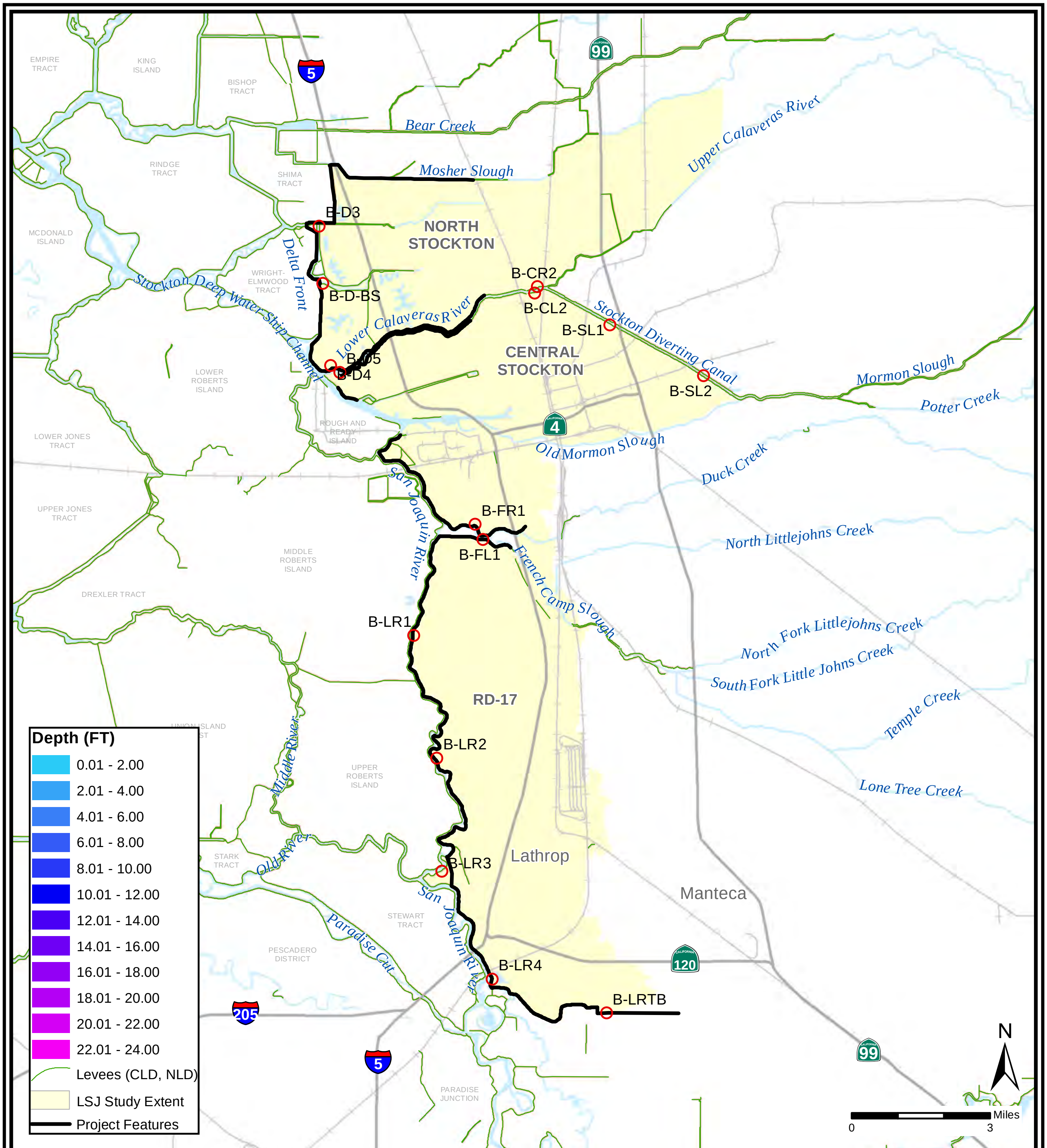
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

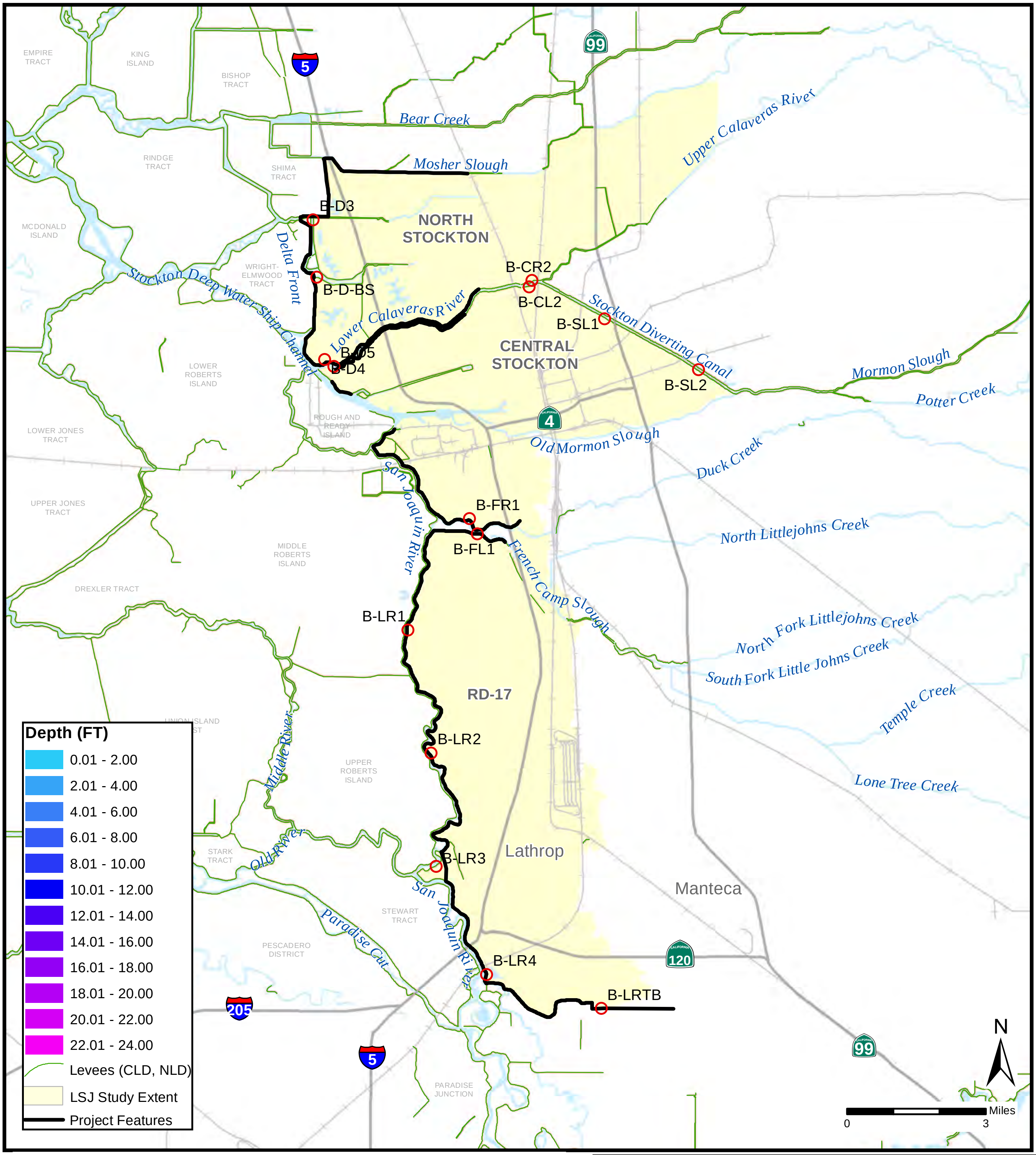
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B 50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 62



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

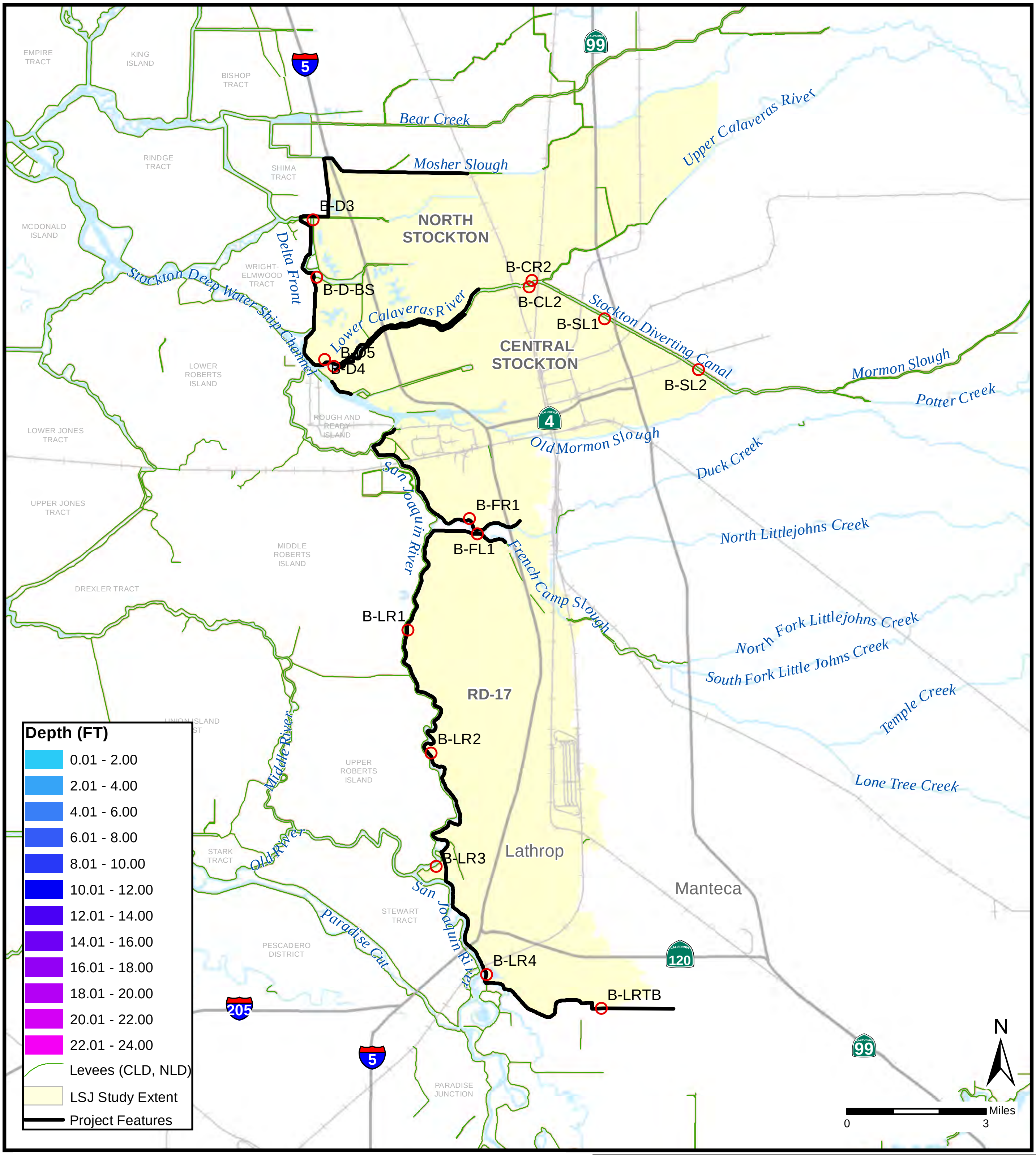
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

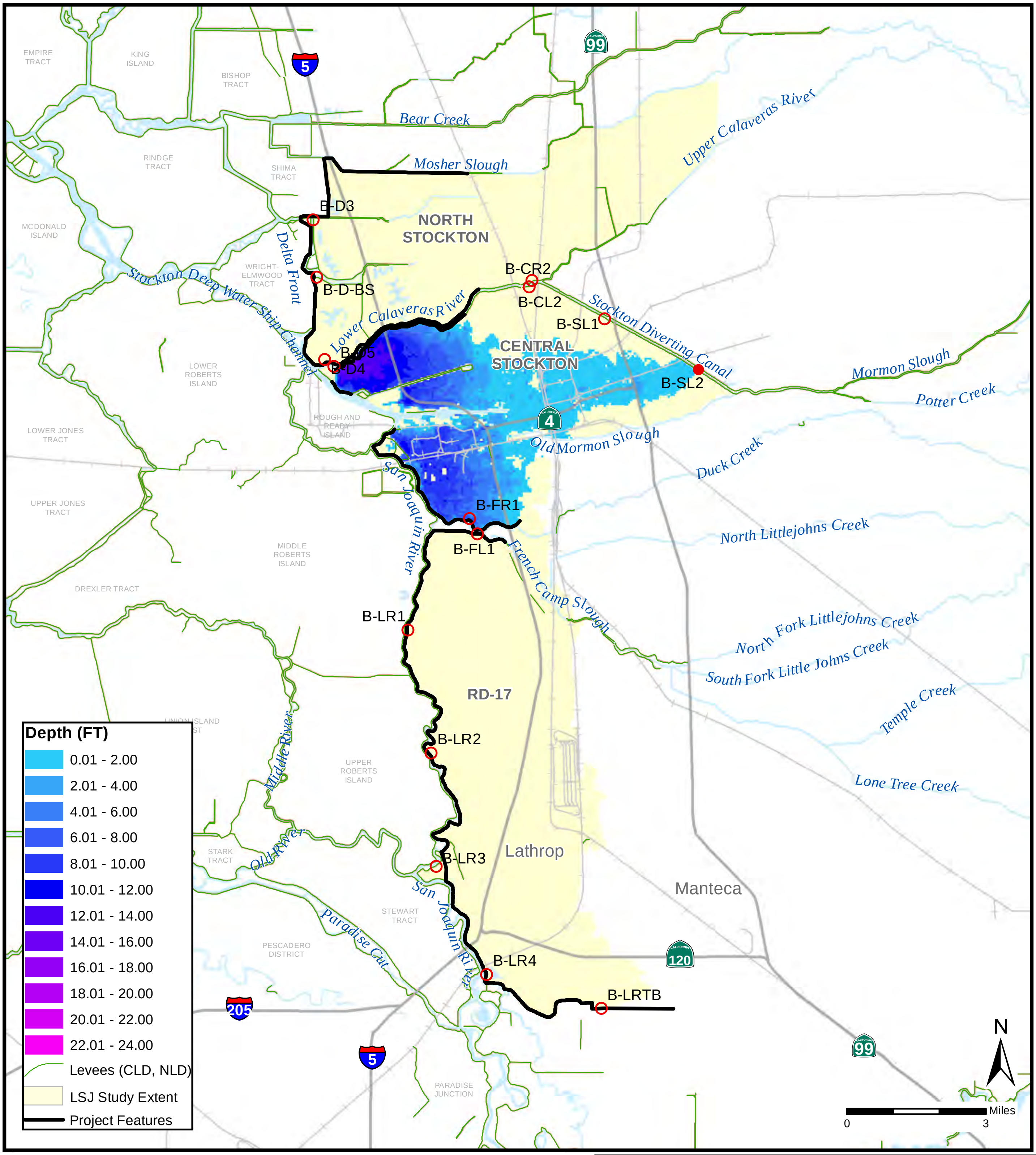
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B 4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

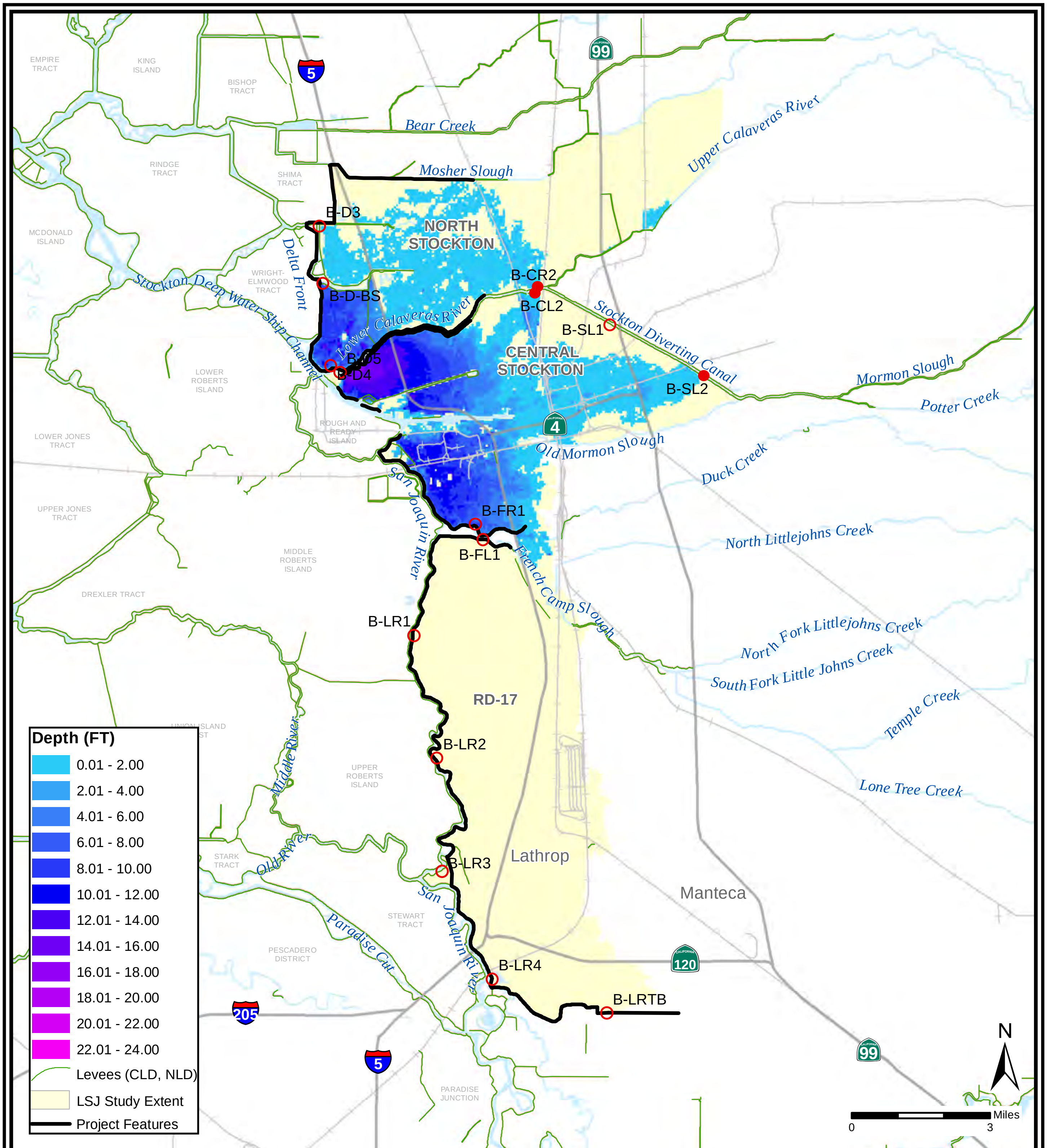
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B 2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

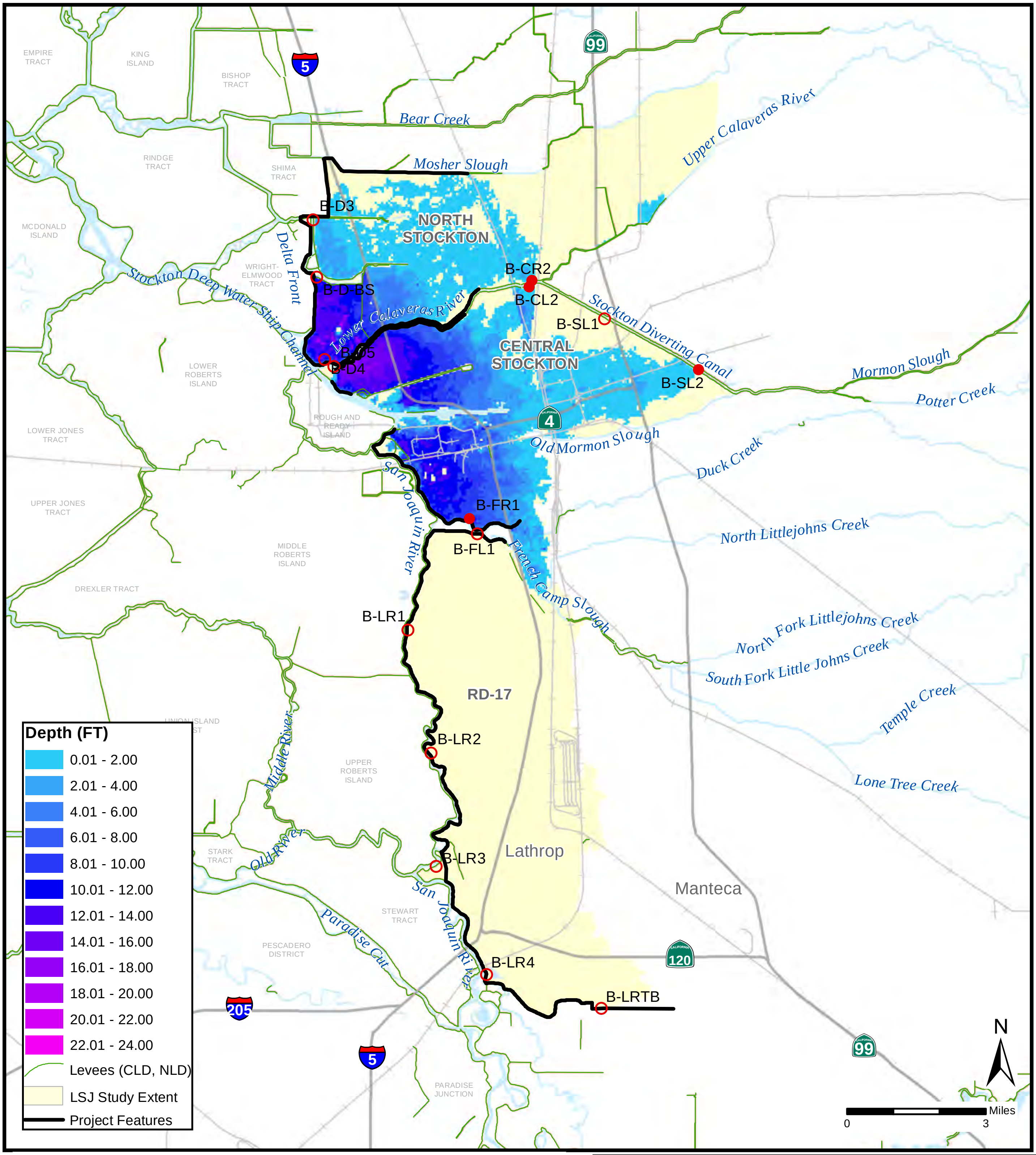
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B 1% (1/100)ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

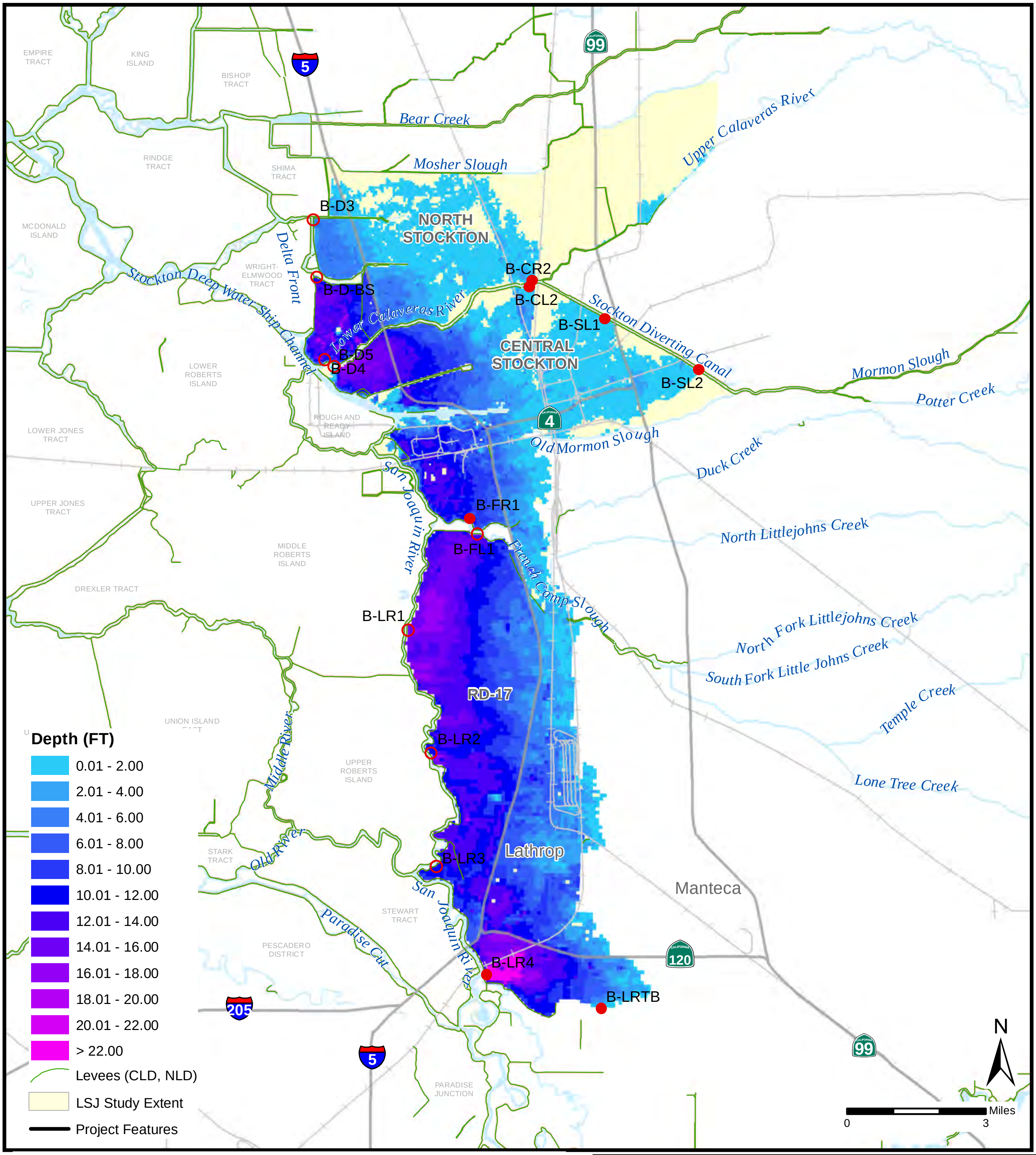
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B 0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

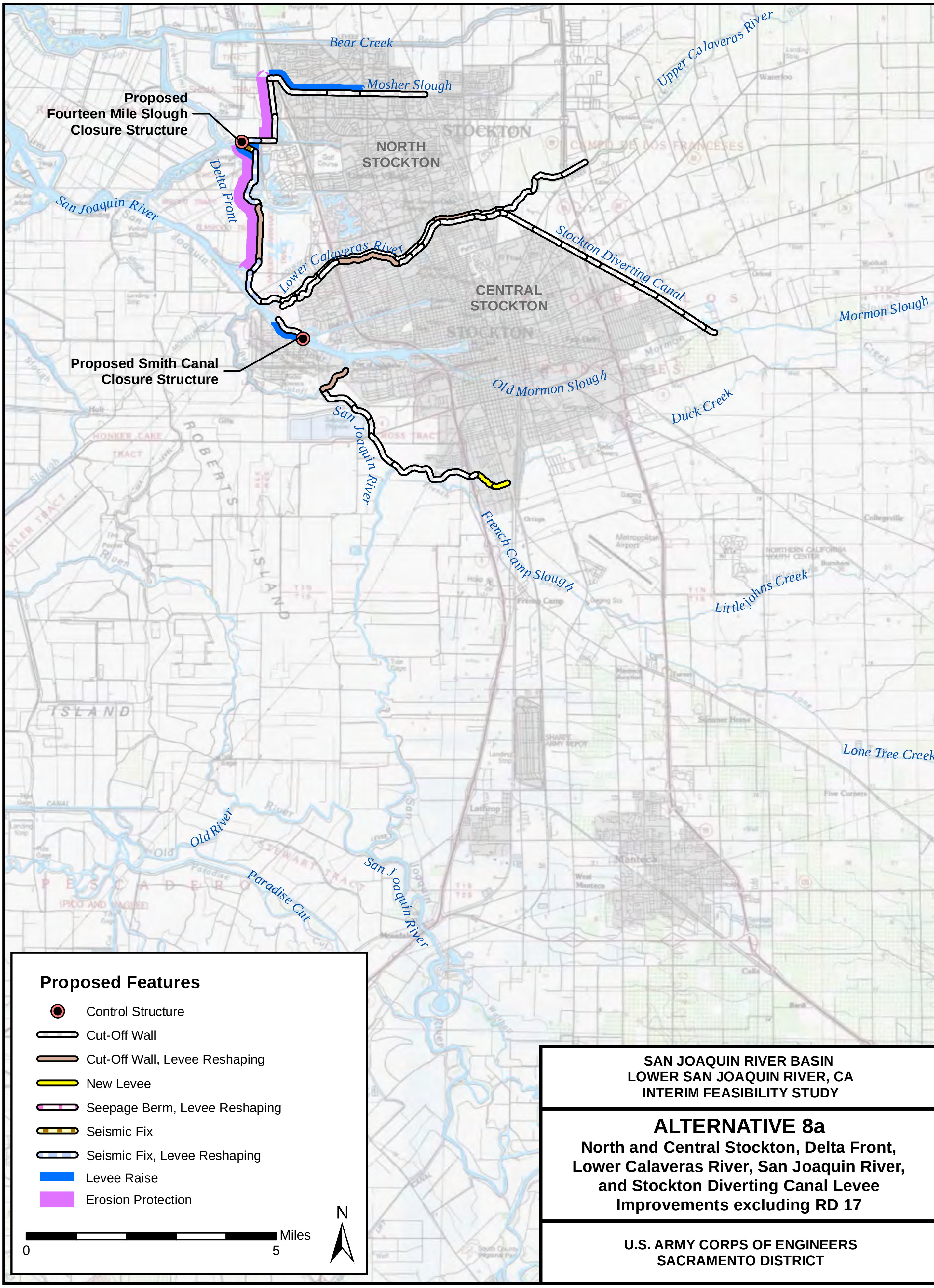
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B 0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

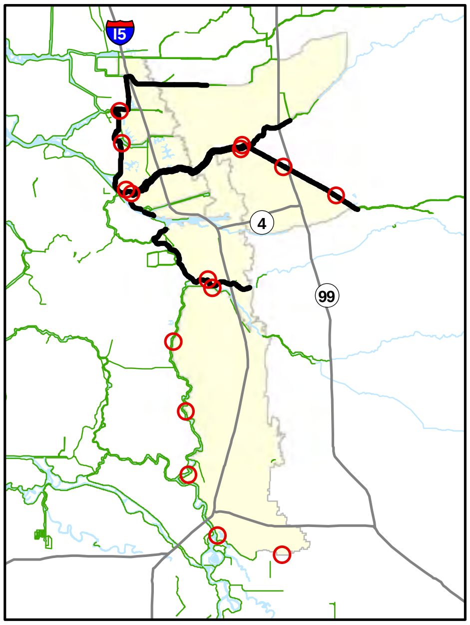
OCT 2014



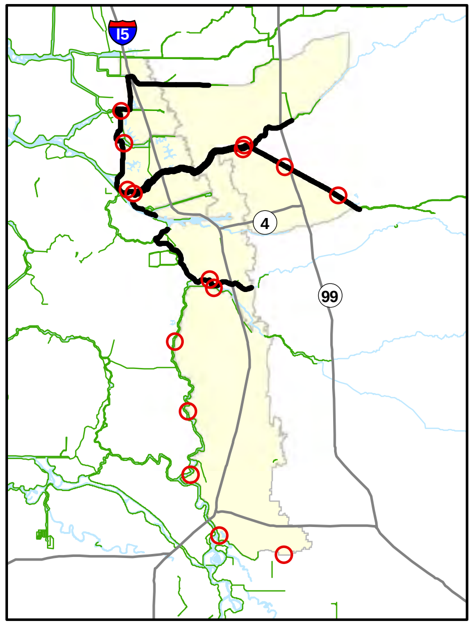
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

ALTERNATIVE 8a
North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River, and Stockton Diverting Canal Levee Improvements excluding RD 17

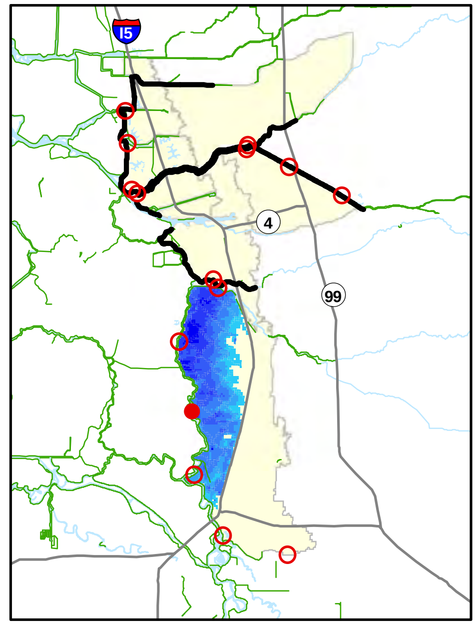
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



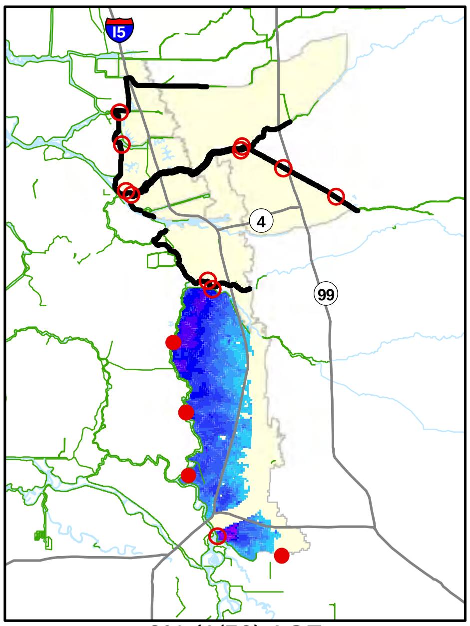
50% (1/2) ACE



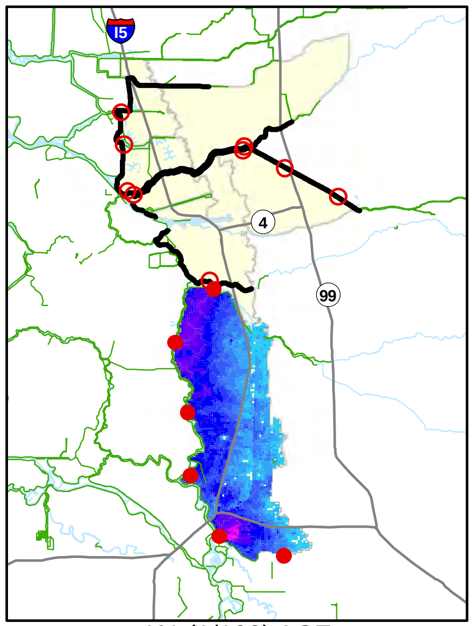
10% (1/10) ACE



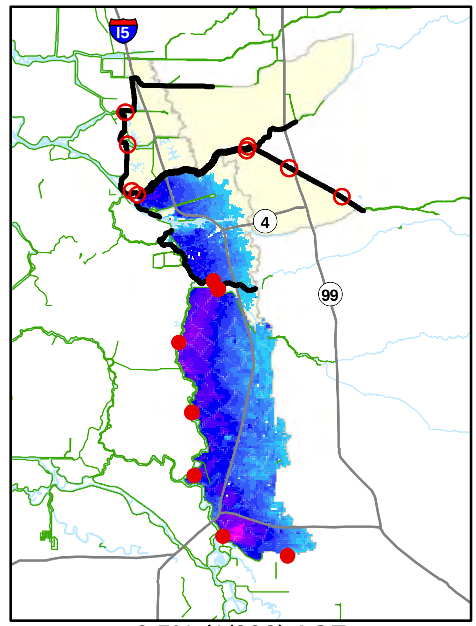
4% (1/25) ACE



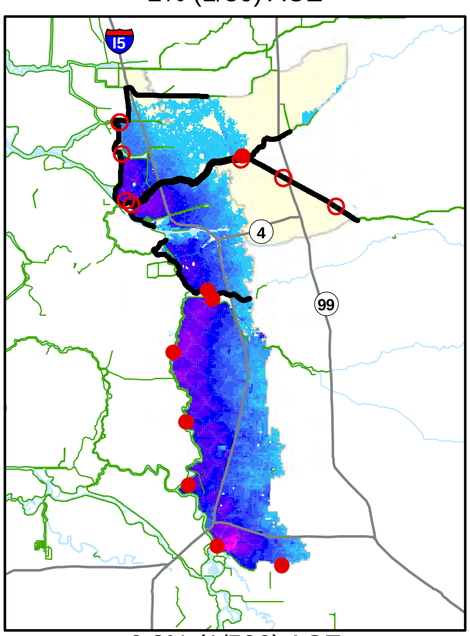
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Fails R&U Criteria
Meets R&U Criteria

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- > 22.0
- Levees (CLD, NLD)
- LSJ Study Extent

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

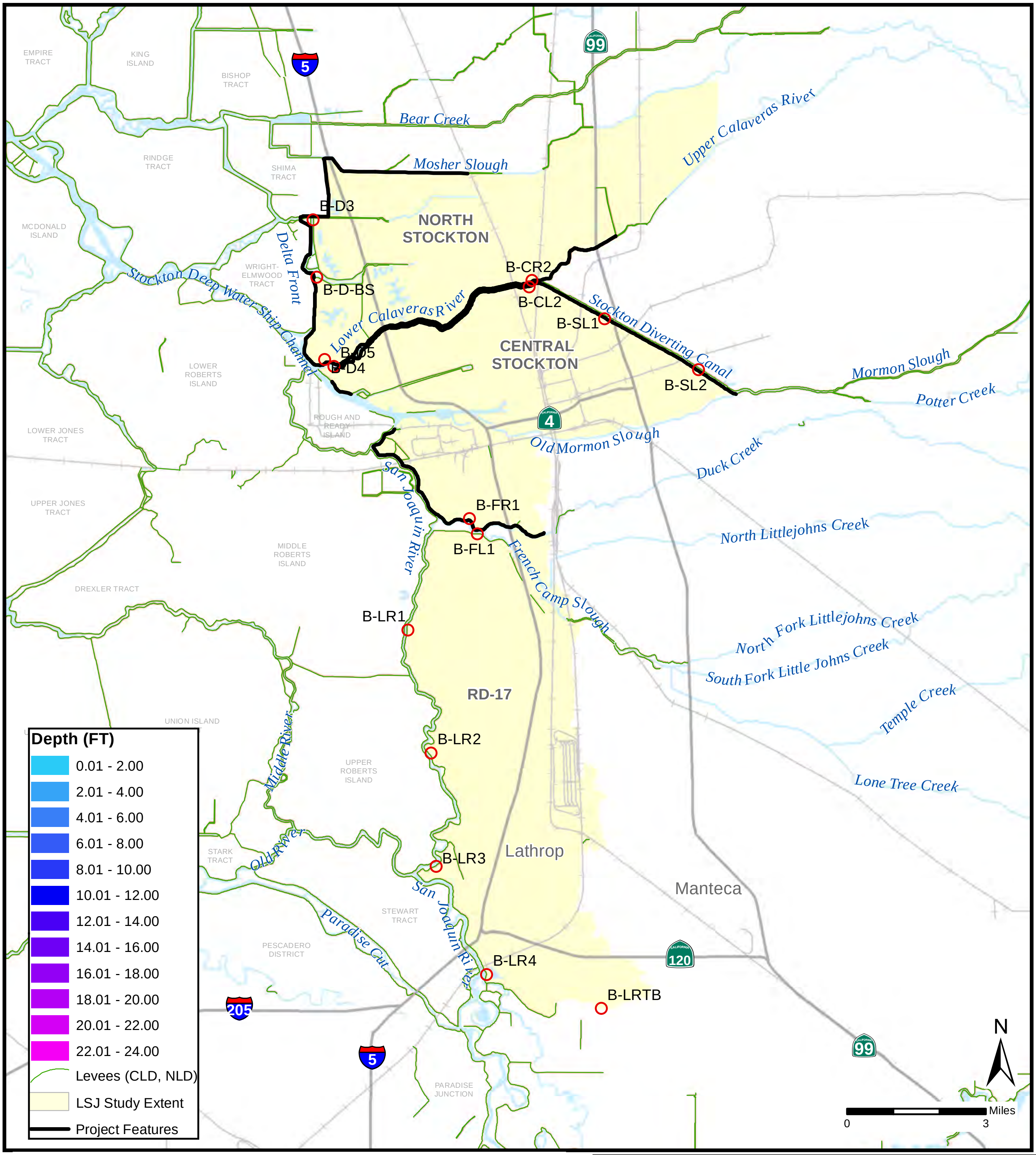
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8A

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

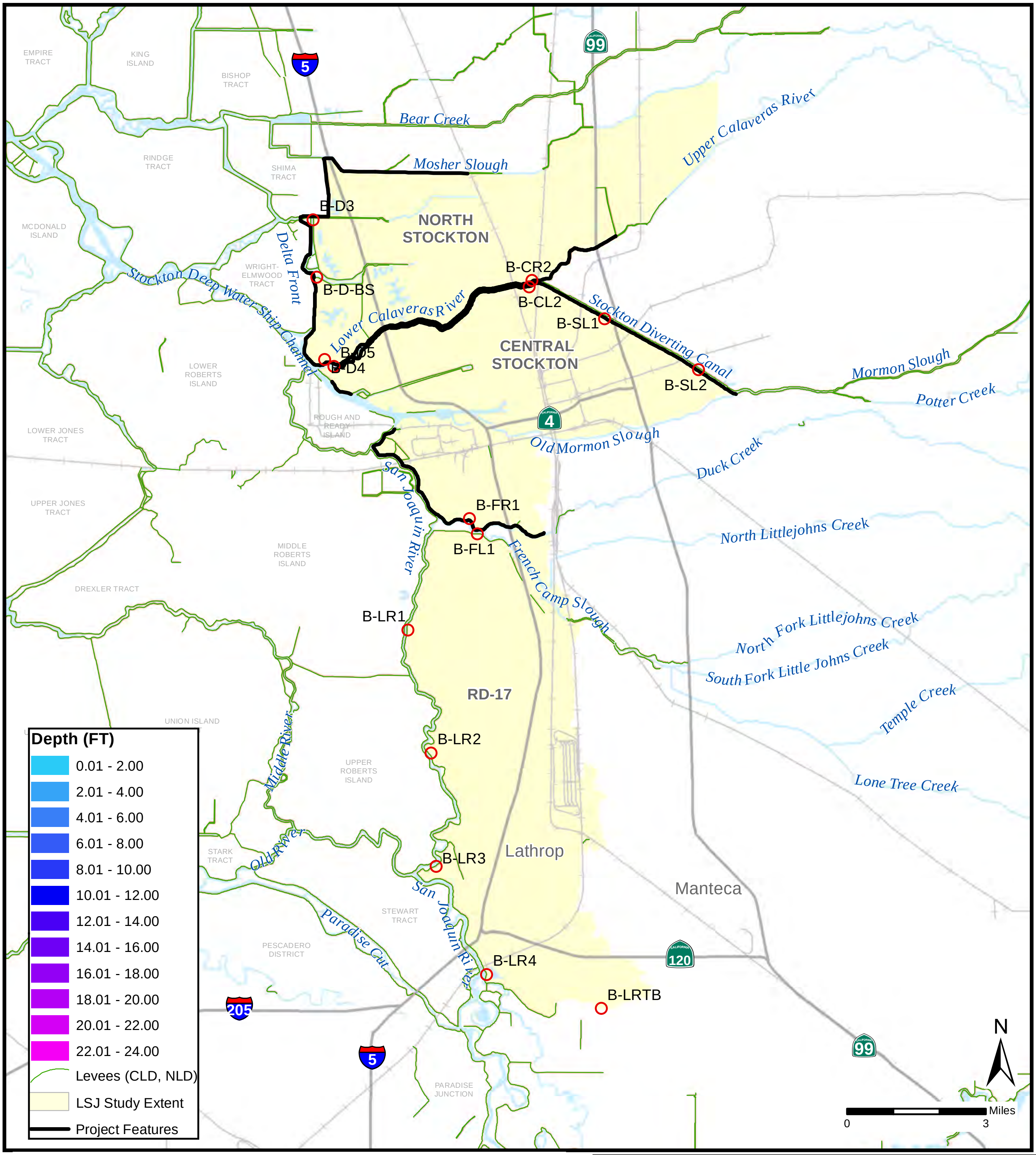
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8A 50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 71



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

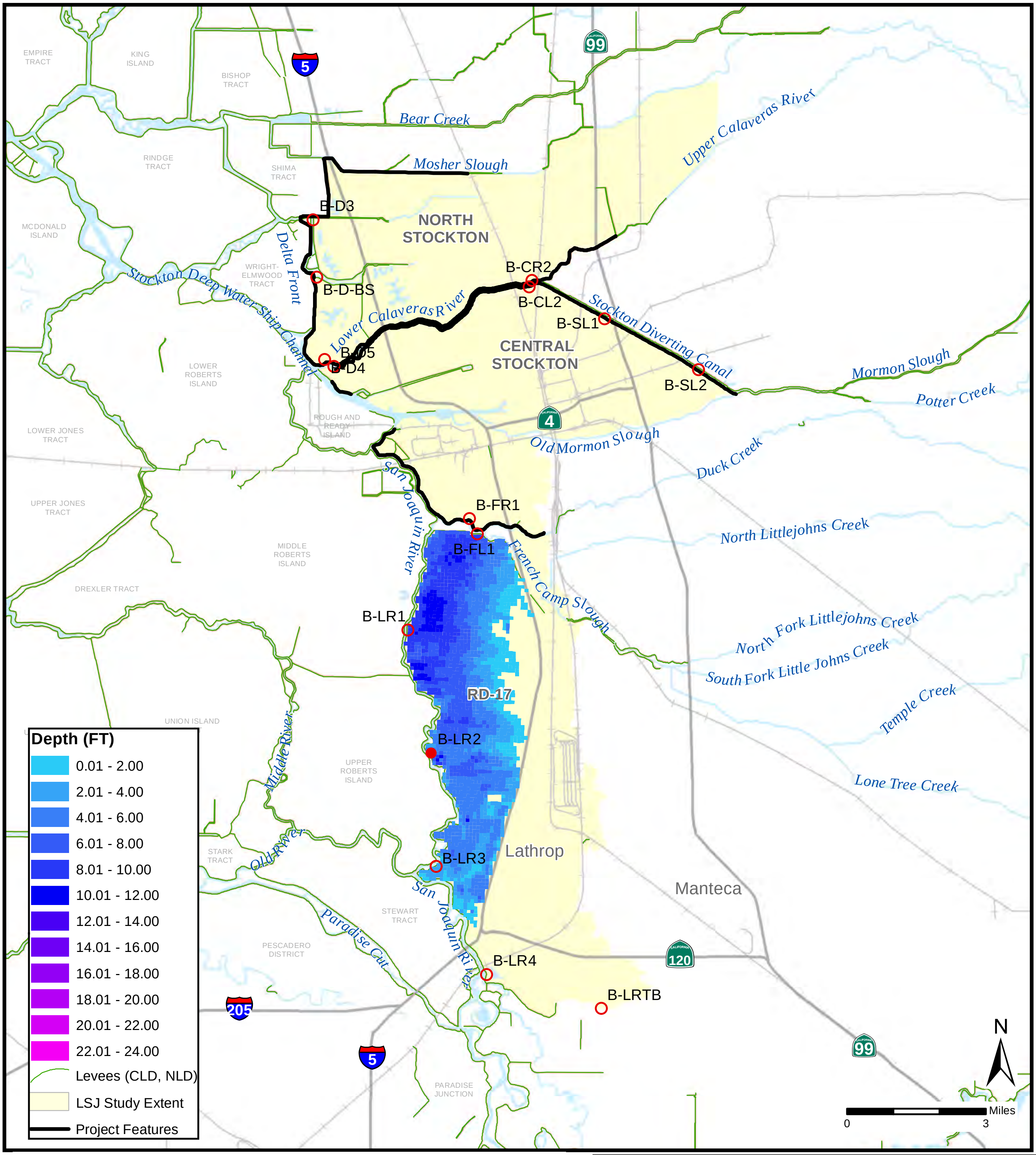
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

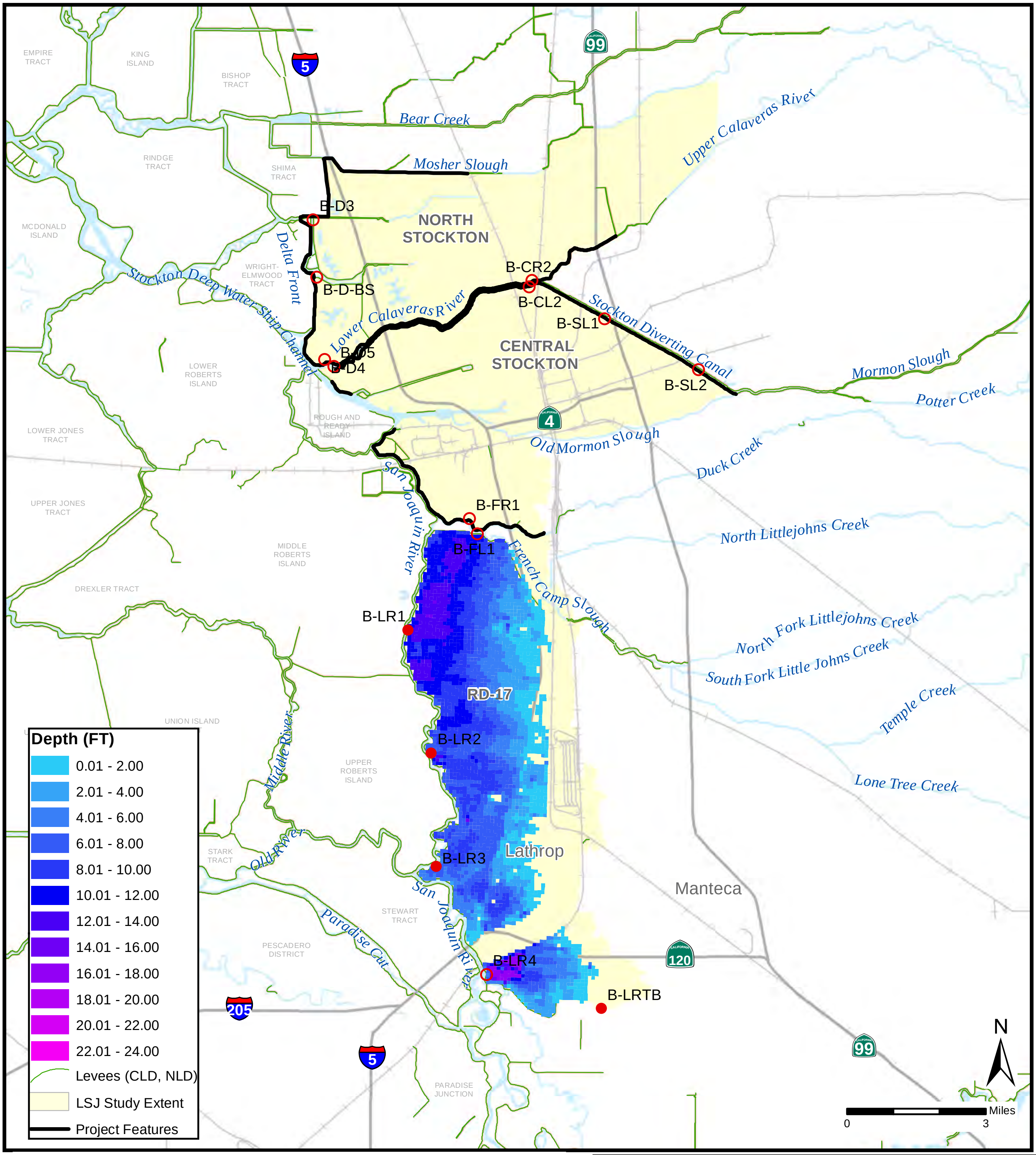
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8A
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

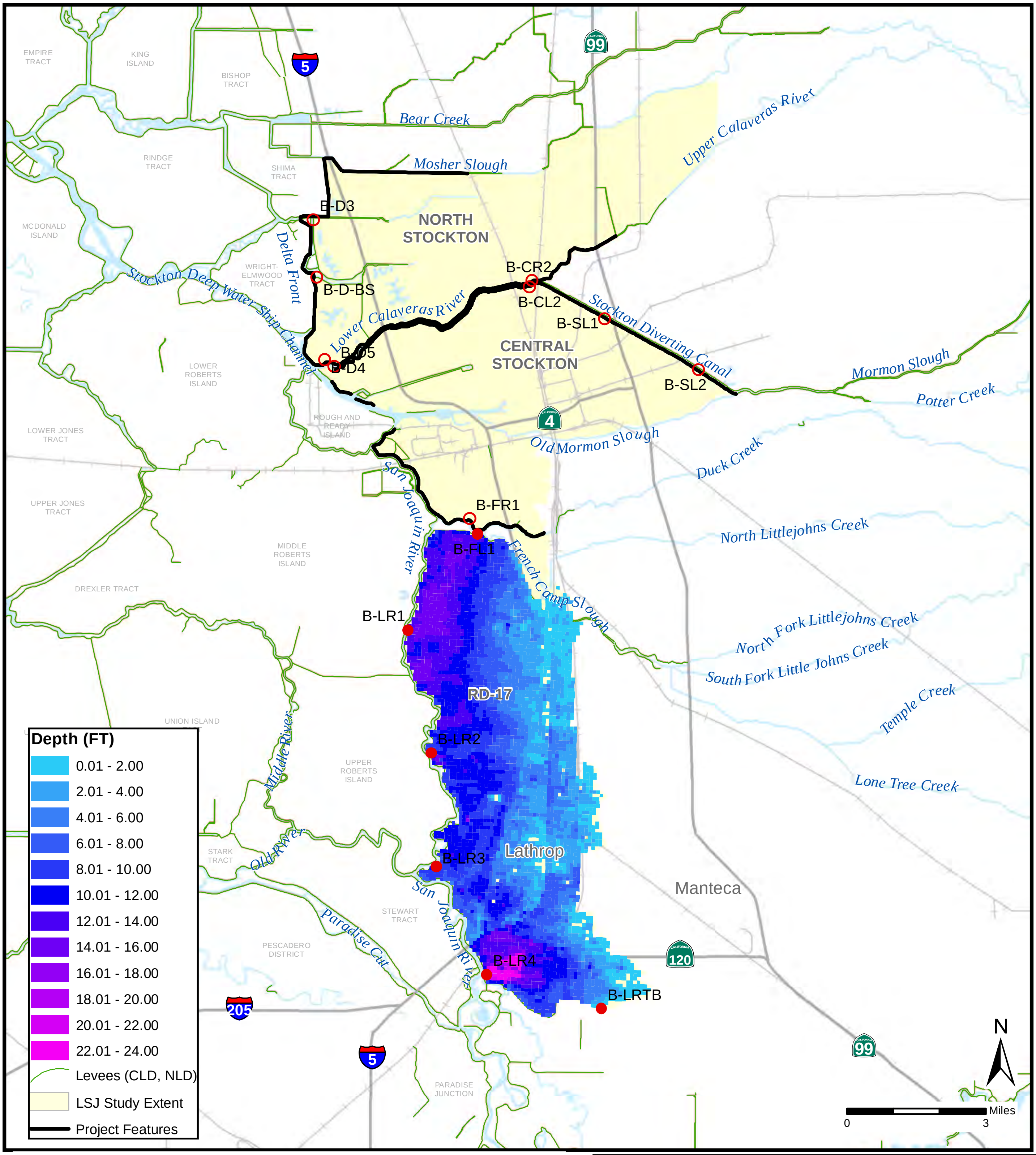
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- ● Fails R&U Criteria
- ○ Meets R&U Criteria

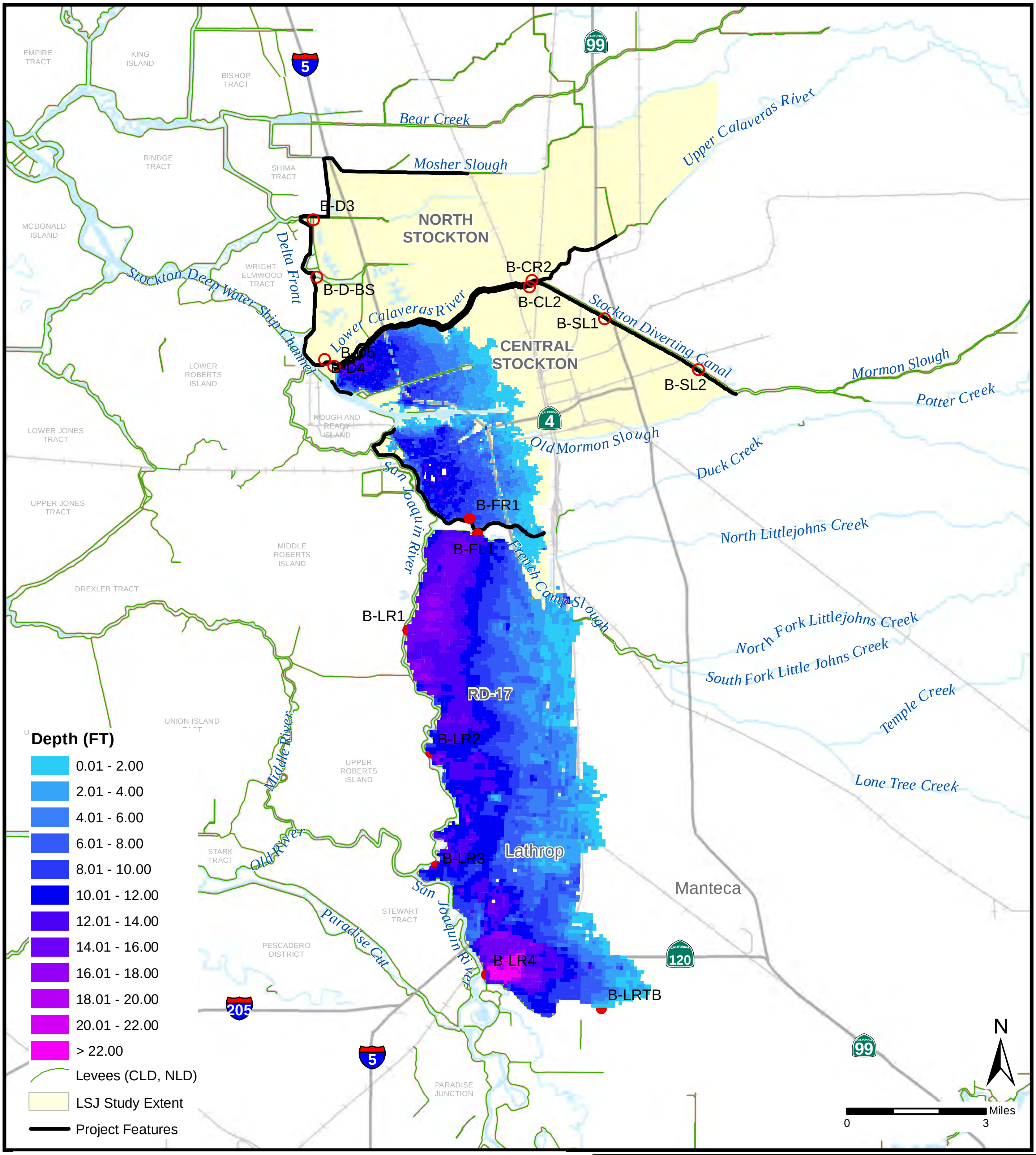
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8A 1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

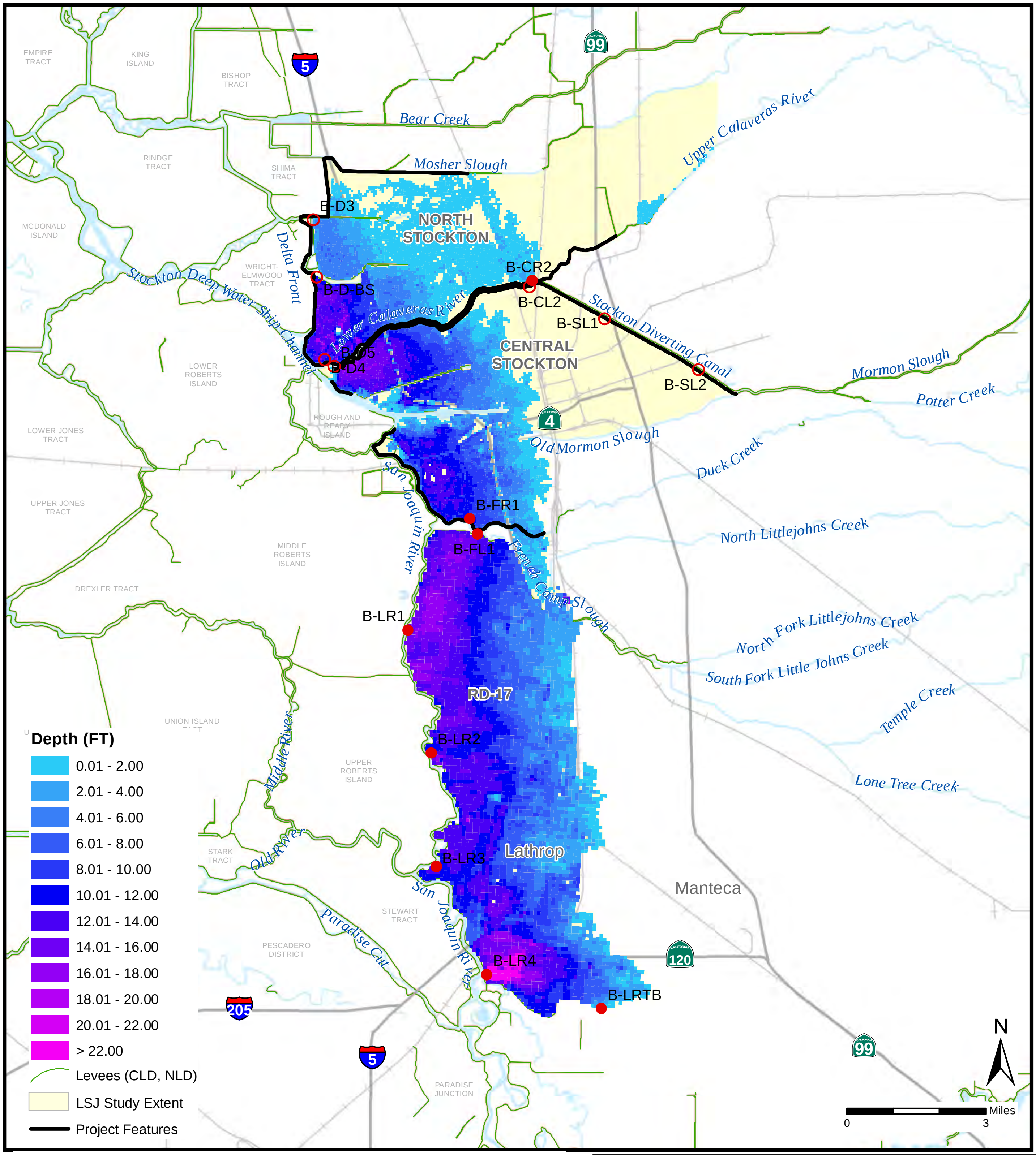
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8A 0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

OCT 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- ● Fails R&U Criteria
- ○ Meets R&U Criteria

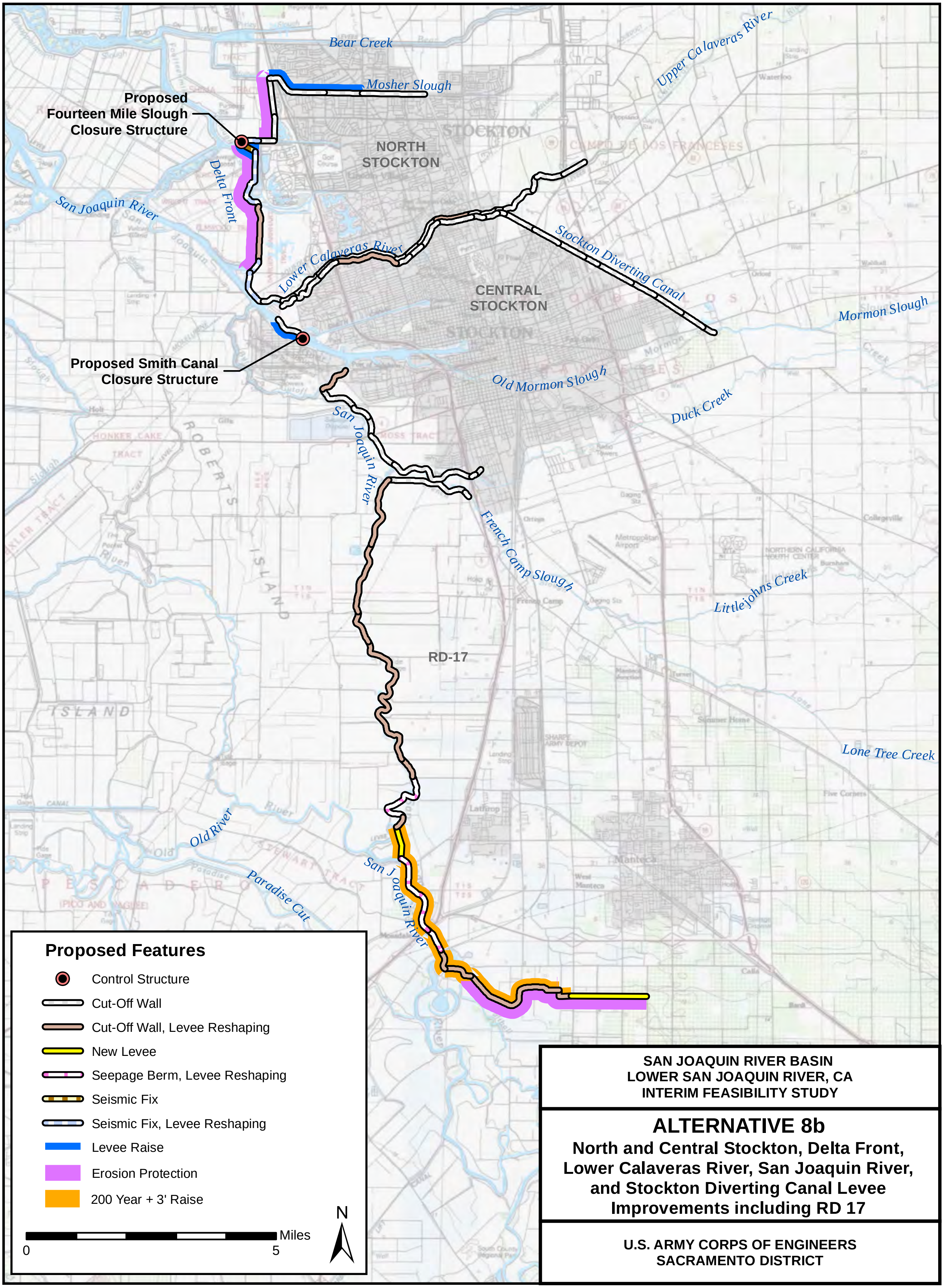
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8A 0.2% (1/500) ACE

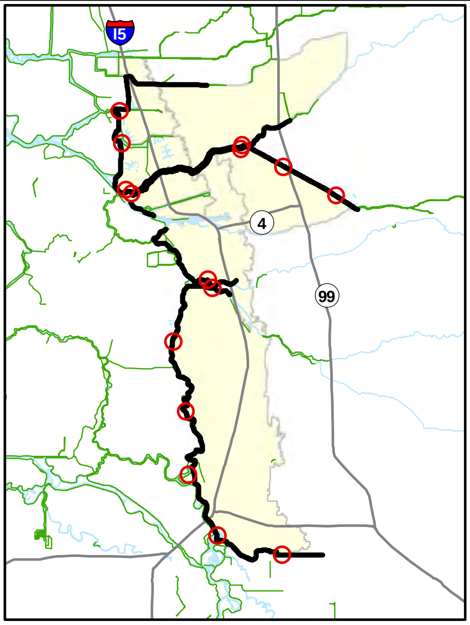
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

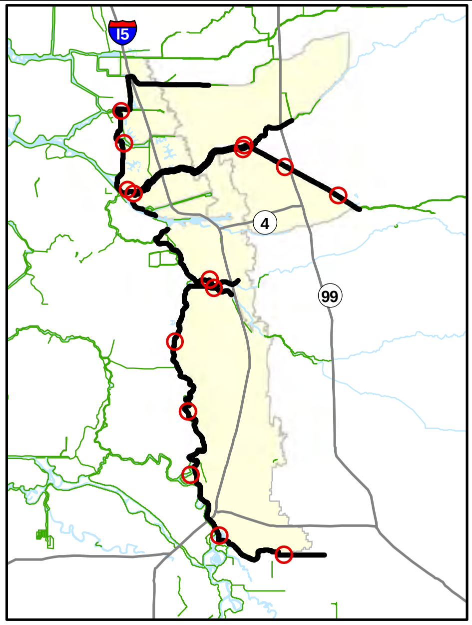
OCT 2014



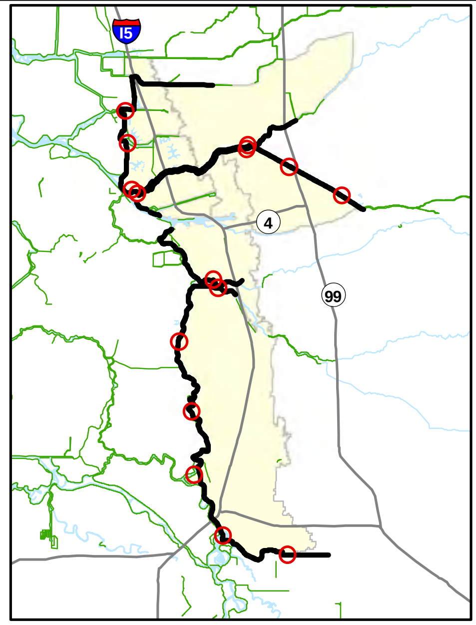
Bear Creek
Mosher Slough
Upper Calaveras River
Proposed
Fourteen Mile Slough
Closure Structure
NORTH
STOCKTON
Delta Front
San Joaquin River
Lower Calaveras River
Stockton Diverting Canal
CENTRAL
STOCKTON
Mormon Slough
Proposed Smith Canal
Closure Structure
Old Mormon Slough
Duck Creek
San Joaquin River
French Camp Slough
Littlejohns Creek
RD-17
Lone Tree Creek
Old River
Paradise Cut
San Joaquin River
Proposed Features
Control Structure
Cut-Off Wall
Cut-Off Wall, Levee Reshaping
New Levee
Seepage Berm, Levee Reshaping
Seismic Fix
Seismic Fix, Levee Reshaping
Levee Raise
Erosion Protection
200 Year + 3' Raise
0
5
Miles
N
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY
ALTERNATIVE 8b
North and Central Stockton, Delta Front,
Lower Calaveras River, San Joaquin River,
and Stockton Diverting Canal Levee
Improvements including RD 17
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



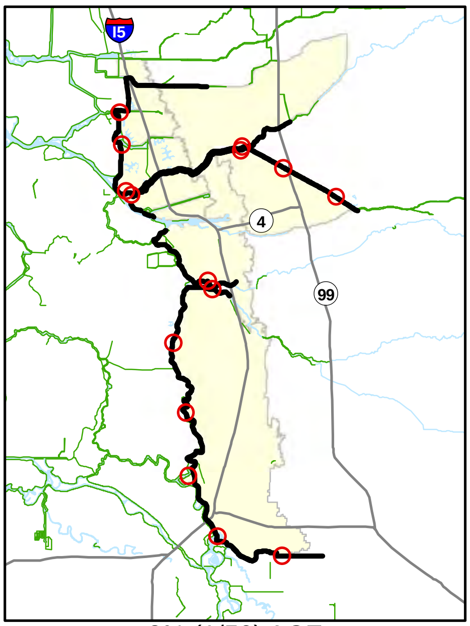
50% (1/2) ACE



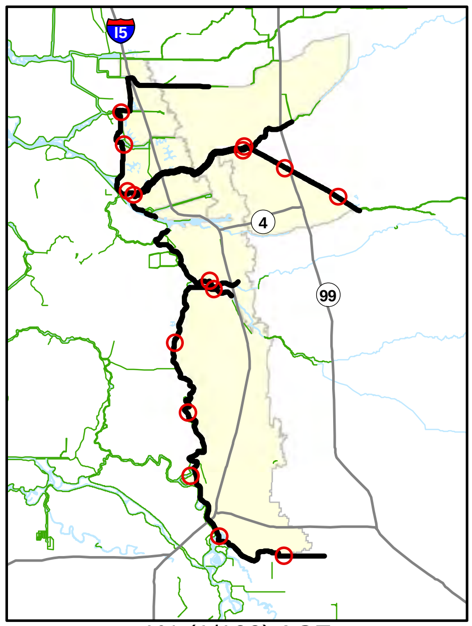
10% (1/10) ACE



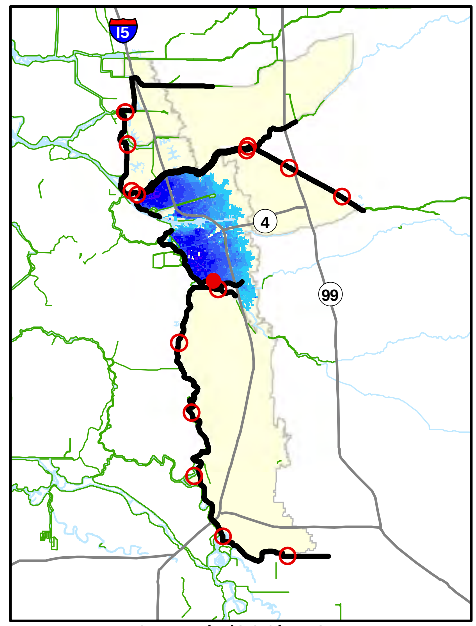
4% (1/25) ACE



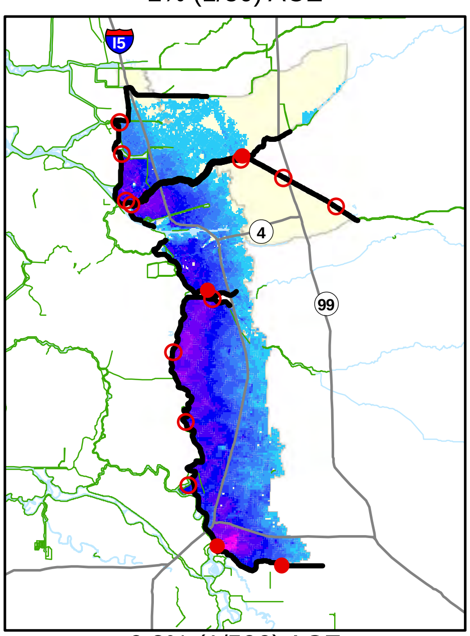
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Fails R&U Criteria
Meets R&U Criteria

Depth (FT)

0.01 - 2.00
2.01 - 4.00
4.01 - 6.00
6.01 - 8.00
8.01 - 10.00
10.01 - 12.00
12.01 - 14.00
14.01 - 16.00
16.01 - 18.00
18.01 - 20.00
20.01 - 22.00
> 22.0
Levees (CLD, NLD)
LSJ Study Extent

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

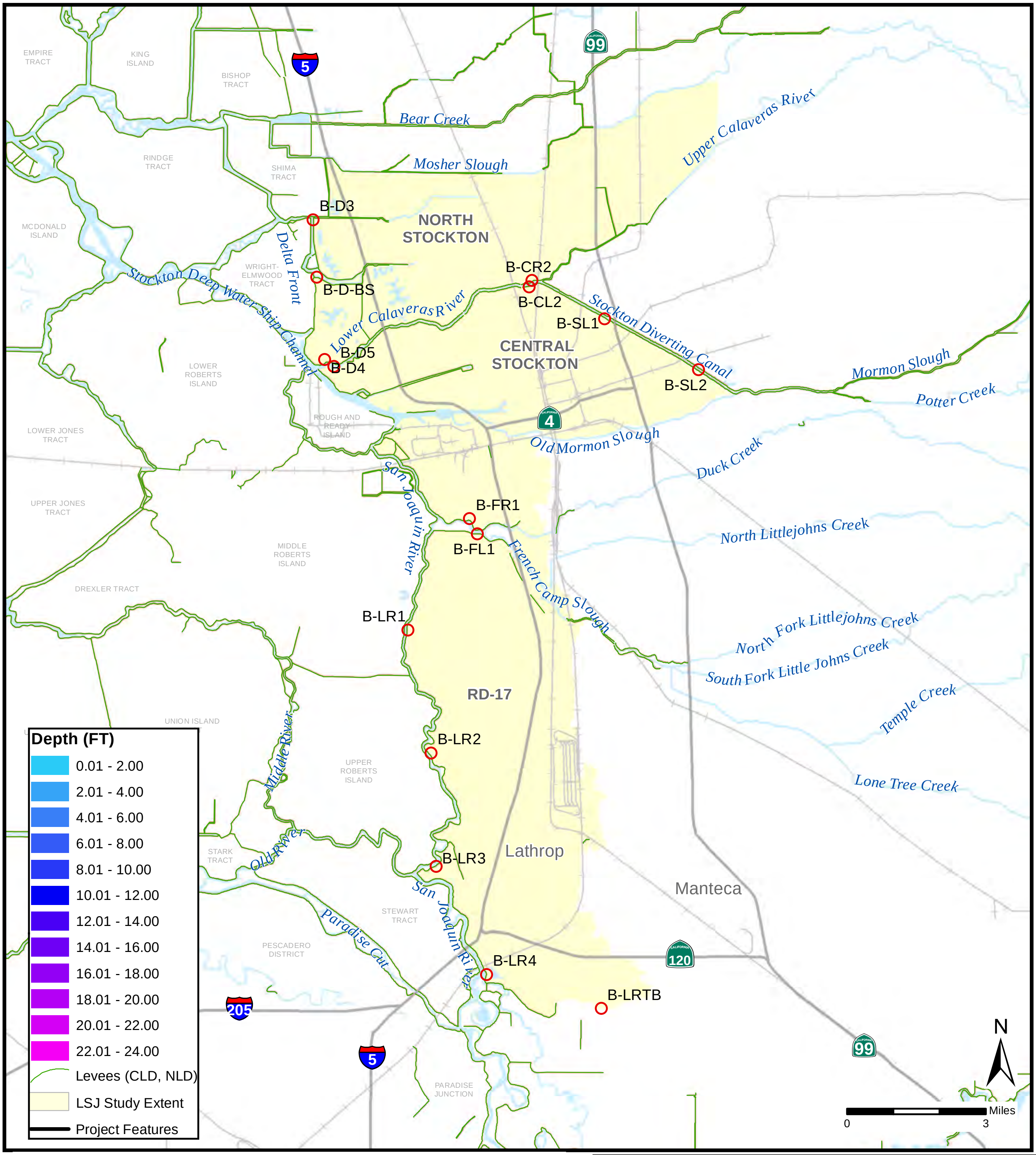
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

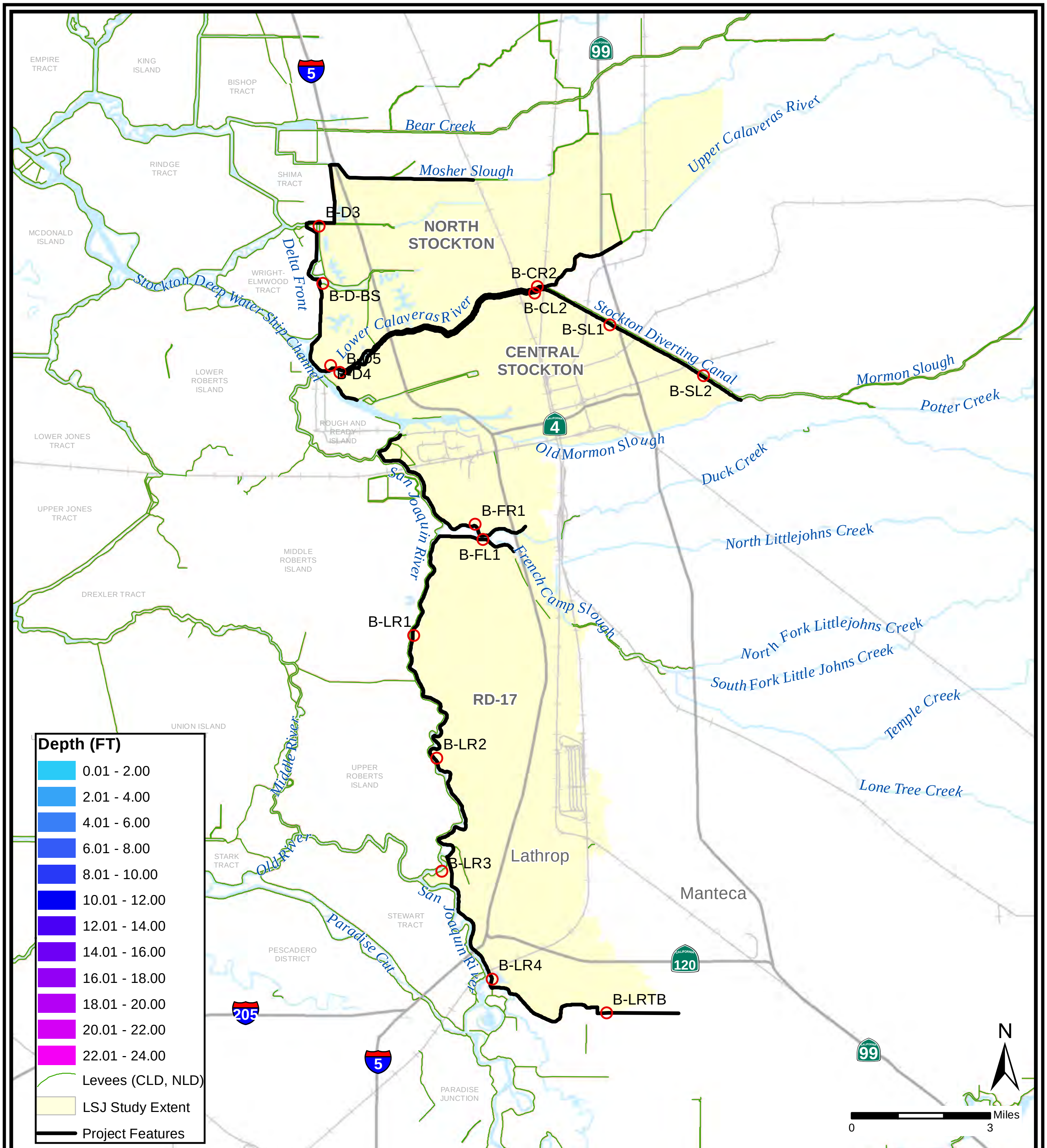
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 80



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- ● Fails R&U Criteria
- ○ Meets R&U Criteria

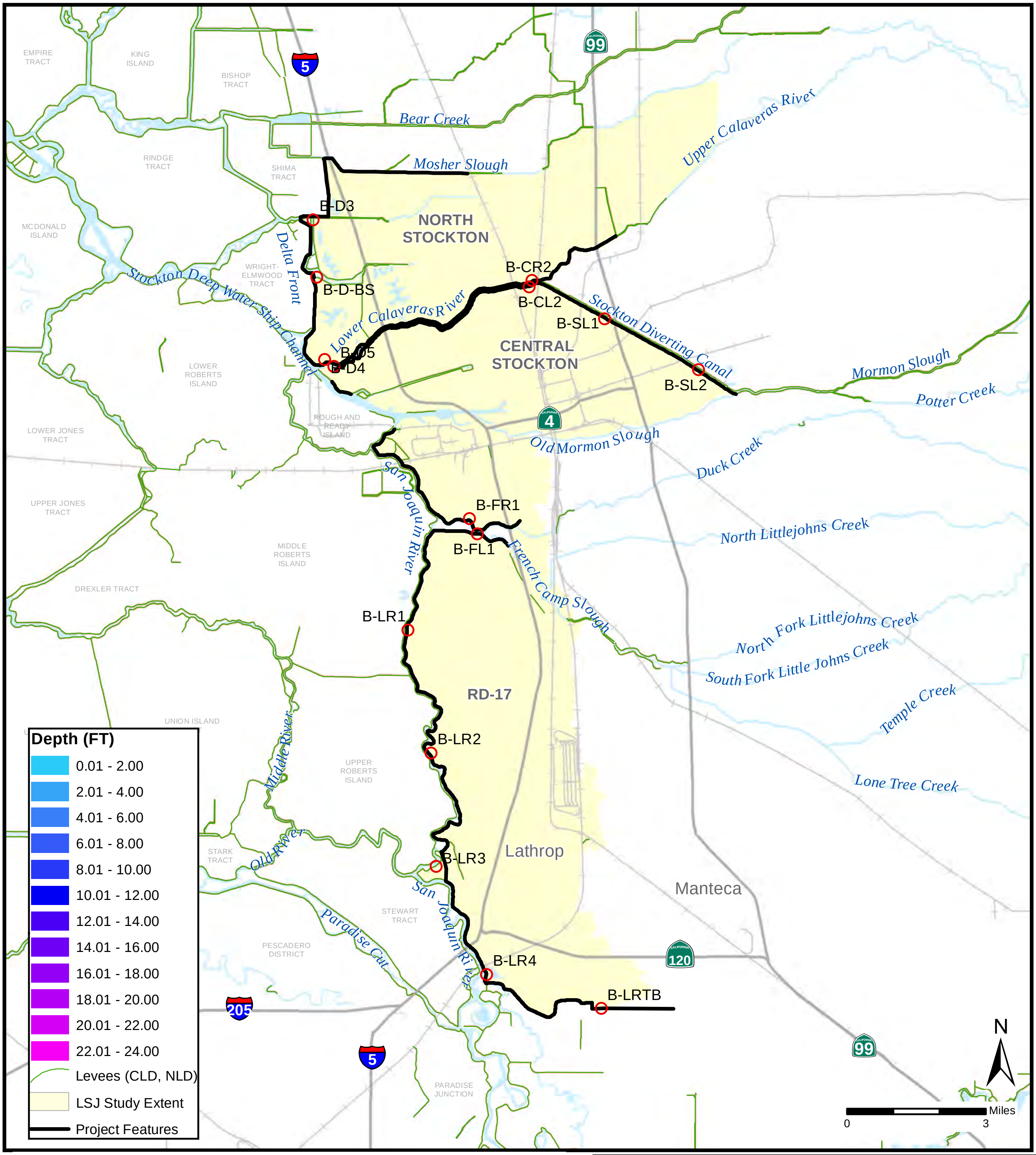
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

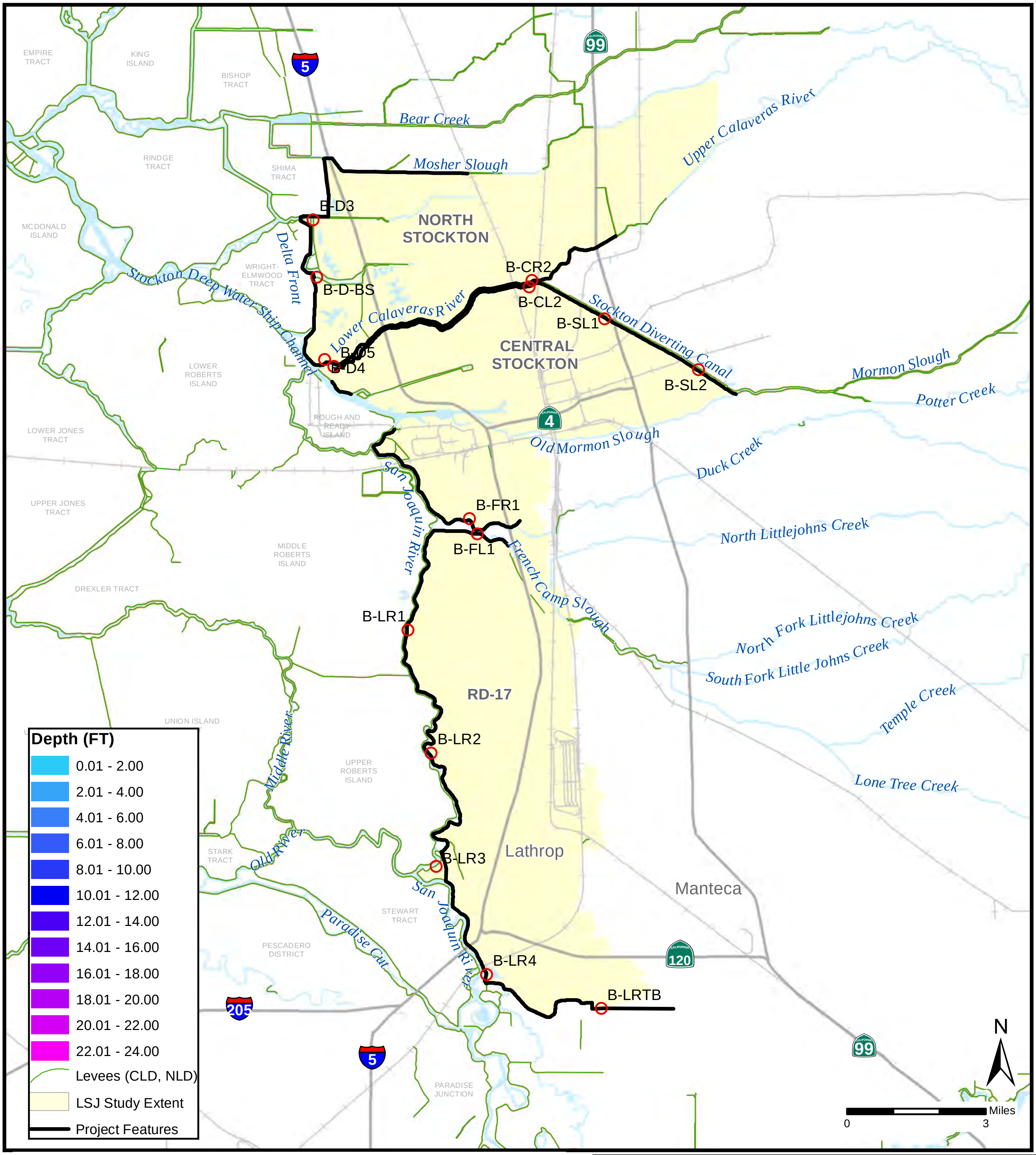
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

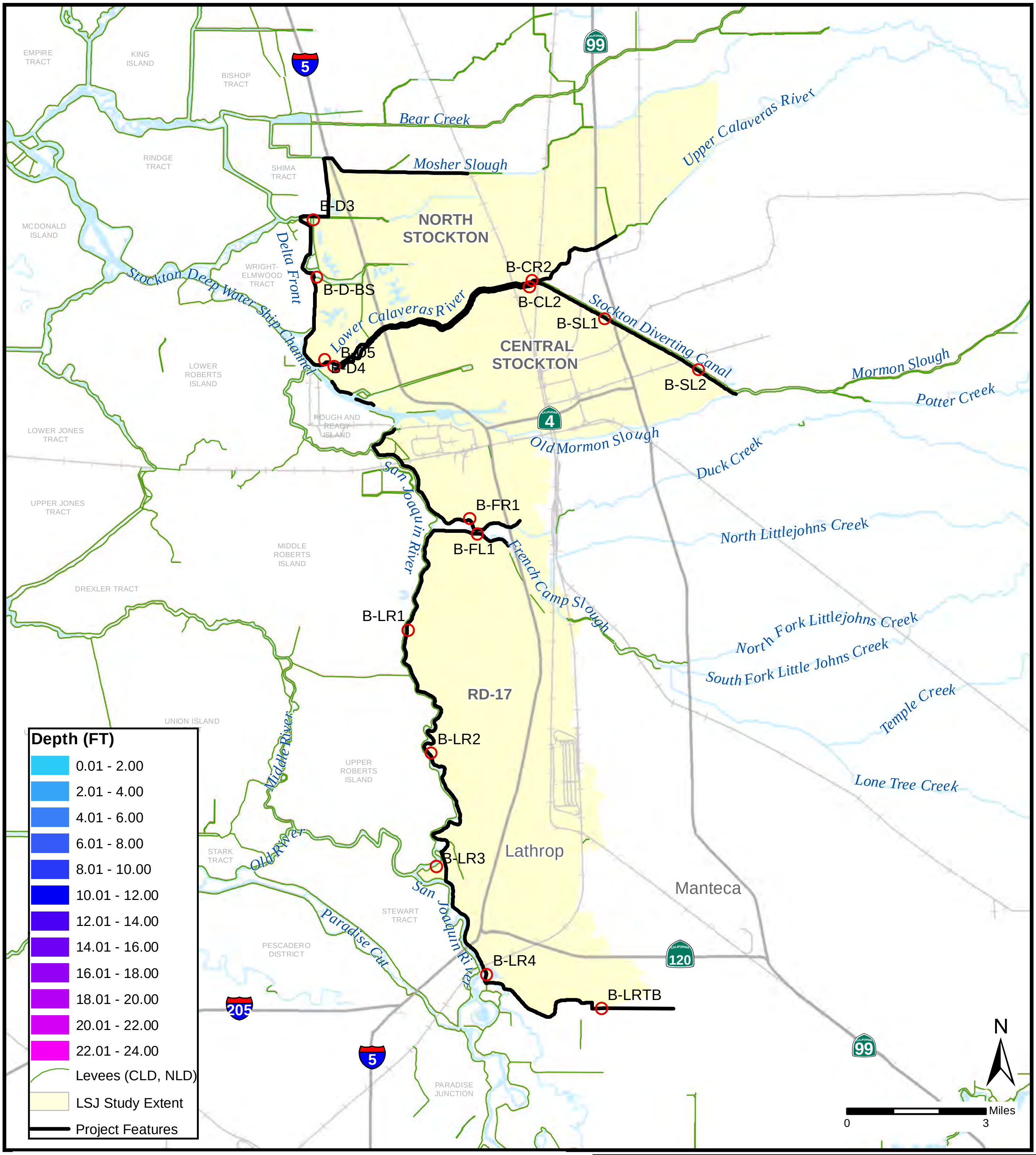
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

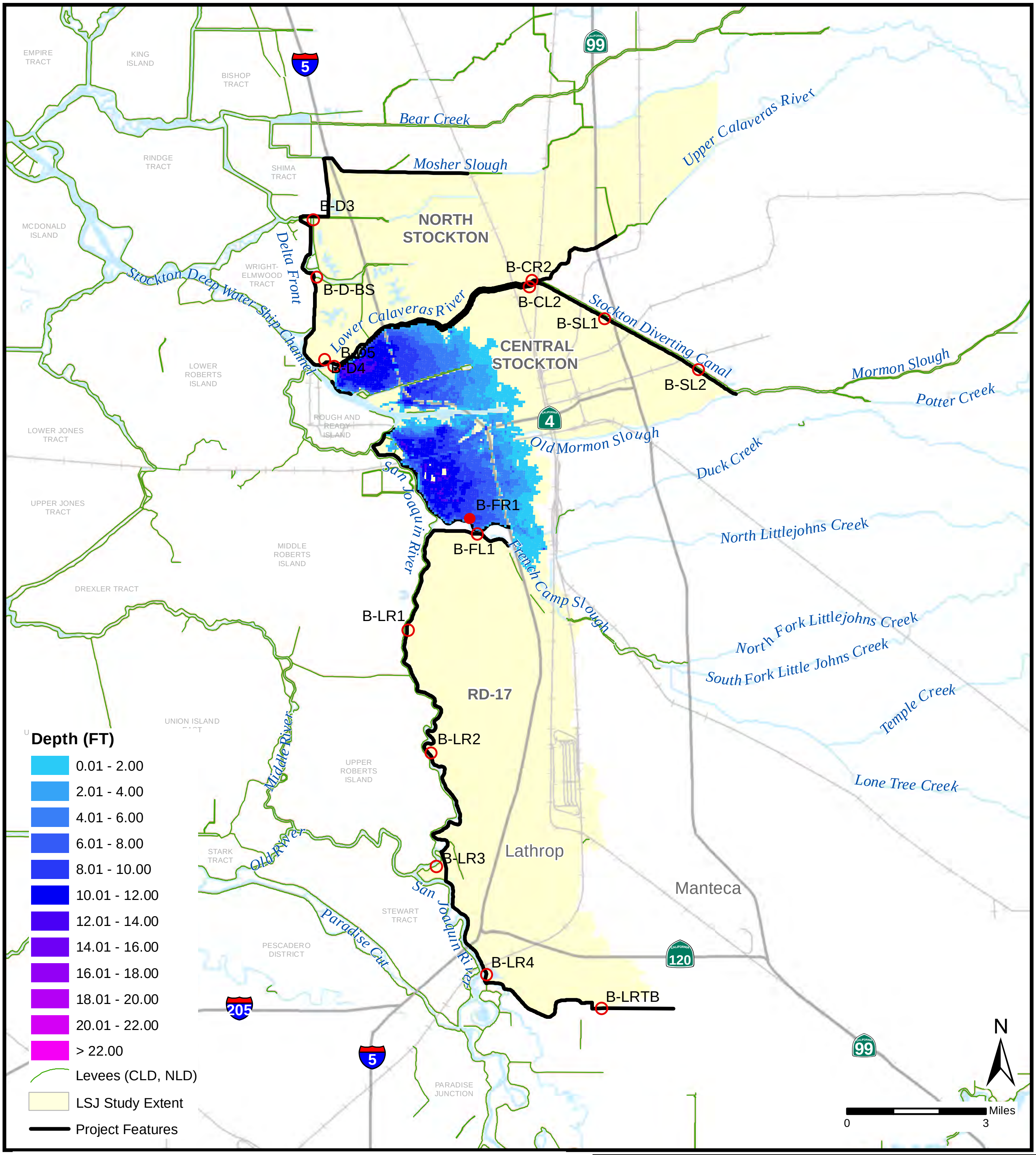
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

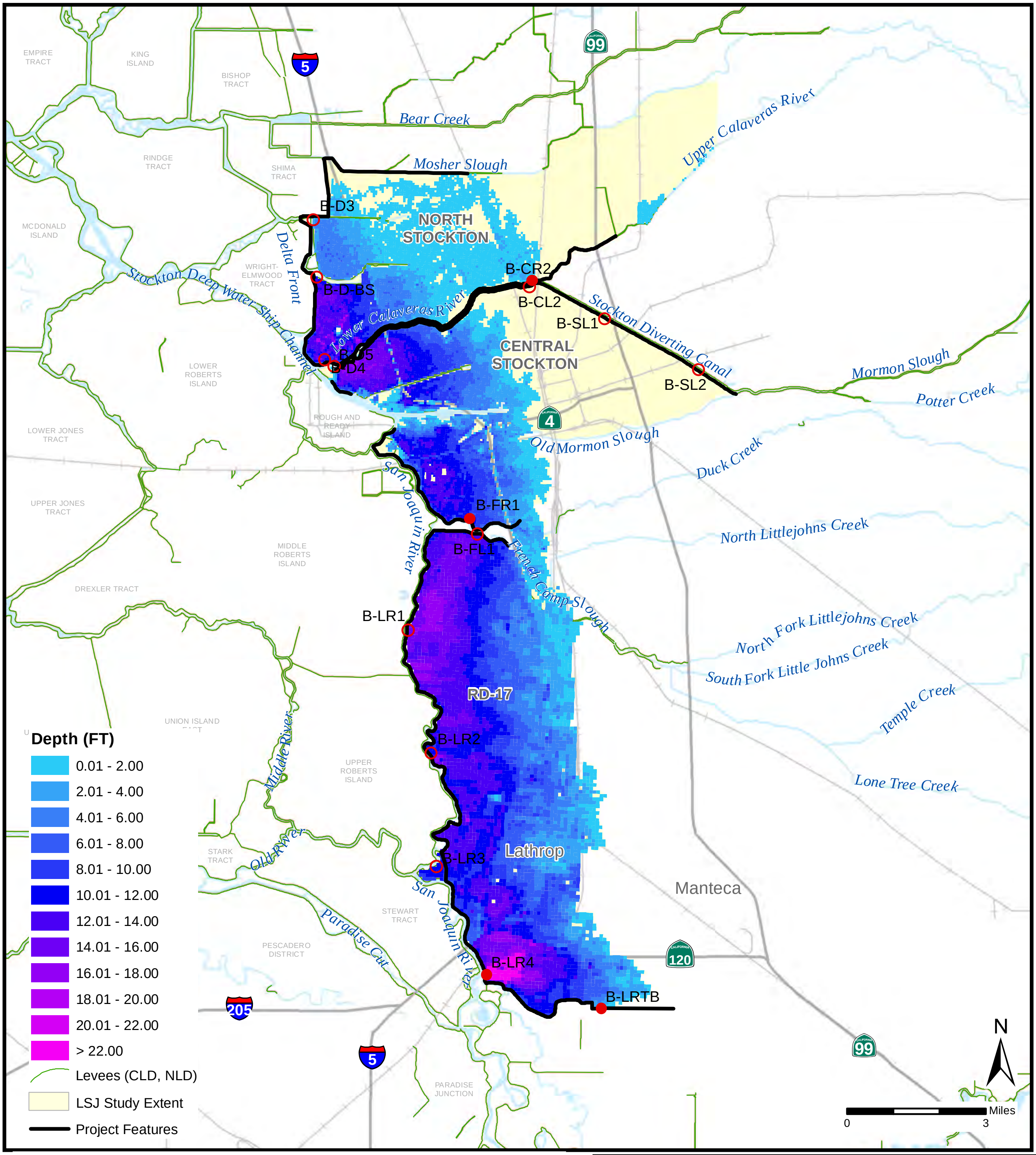
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

OCT 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- ● Fails R&U Criteria
- ○ Meets R&U Criteria

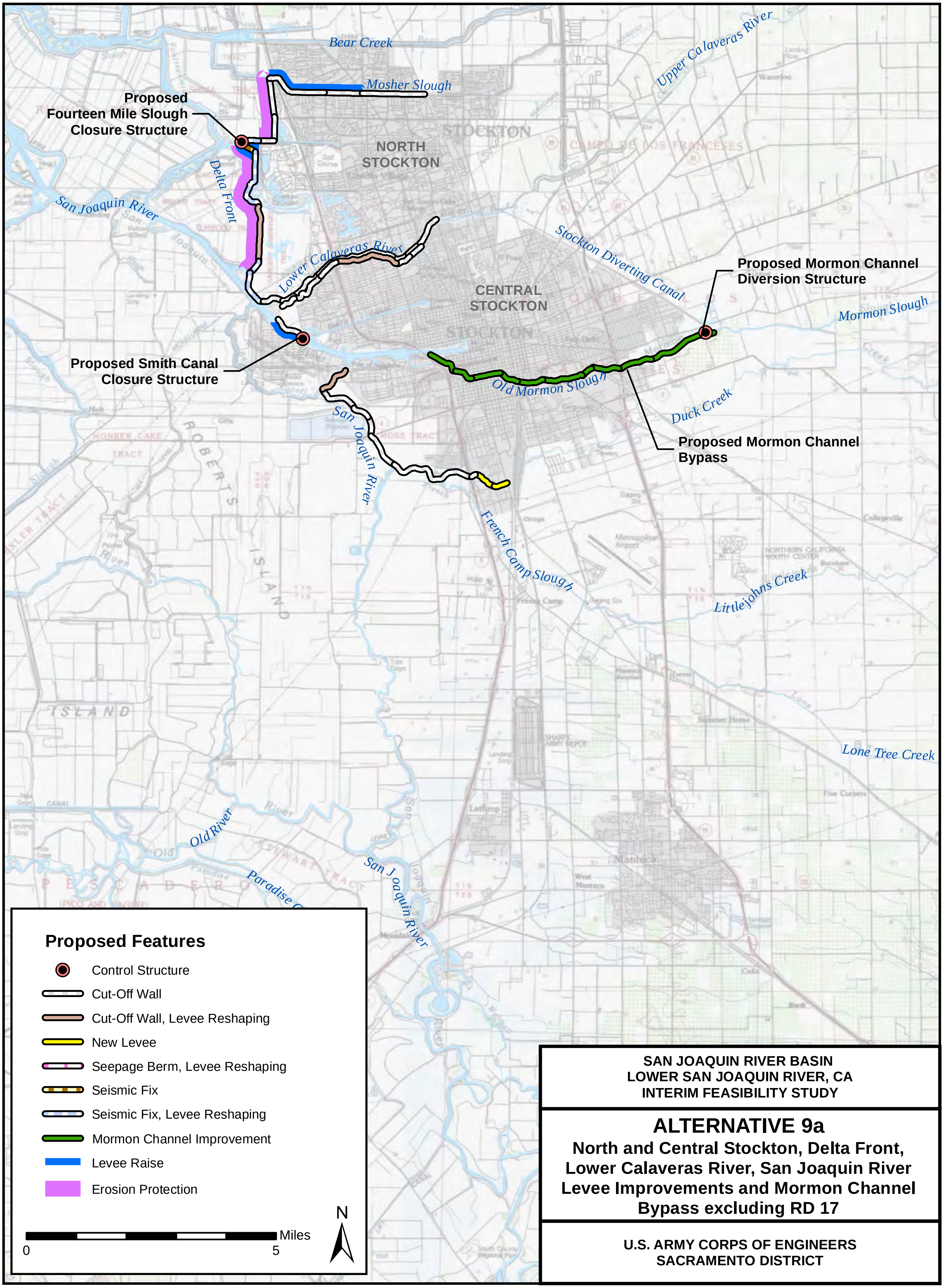
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

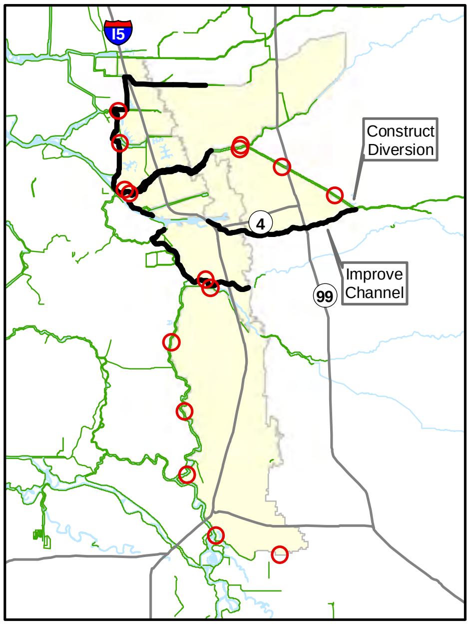
OCT 2014



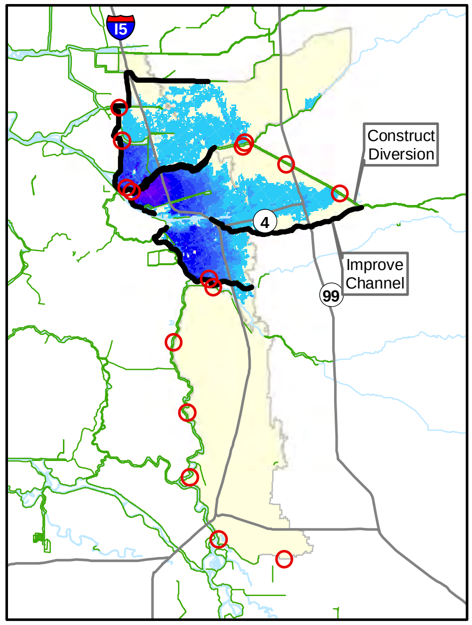
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

ALTERNATIVE 9a
North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River Levee Improvements and Mormon Channel Bypass excluding RD 17

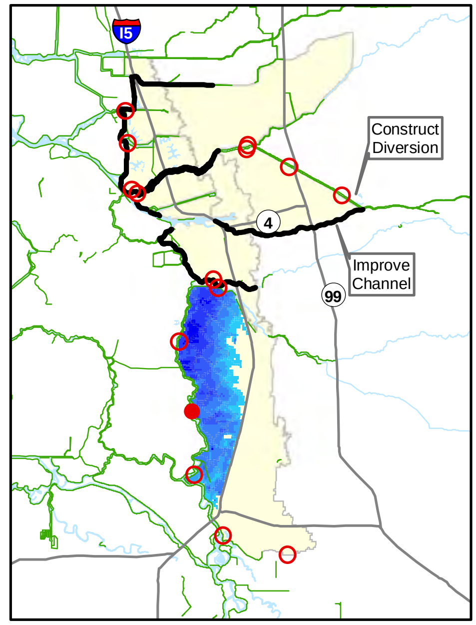
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



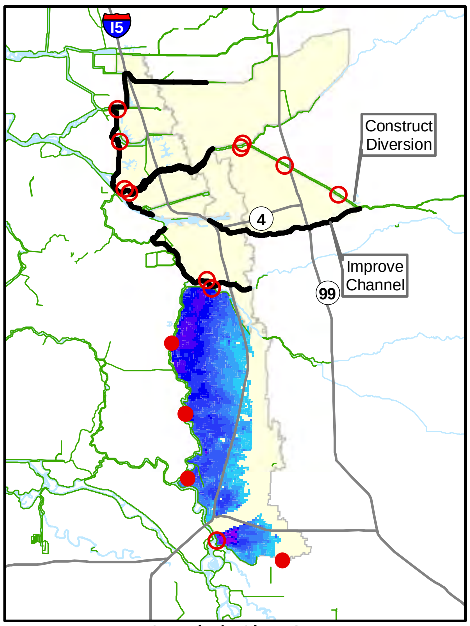
50% (1/2) ACE



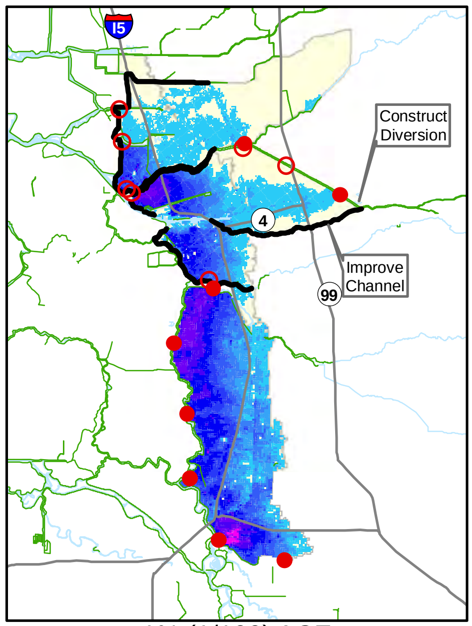
10% (1/10) ACE



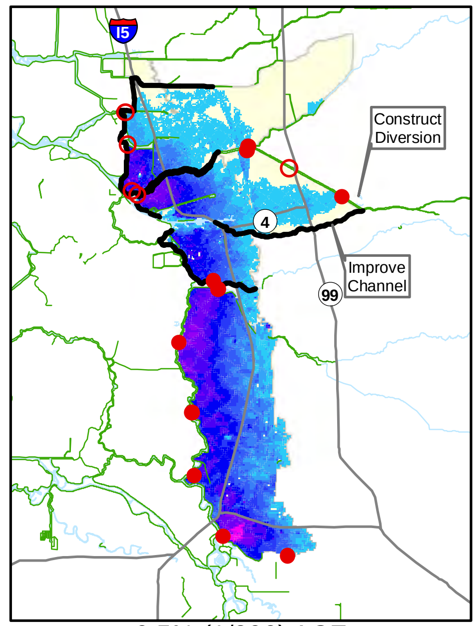
4% (1/25) ACE



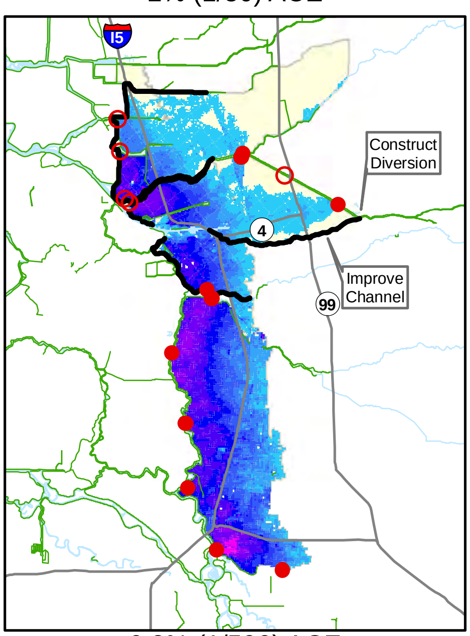
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

Fails R&U Criteria

Meets R&U Criteria

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- > 22.0
- Levees (CLD, NLD)
- LSJ Study Extent

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

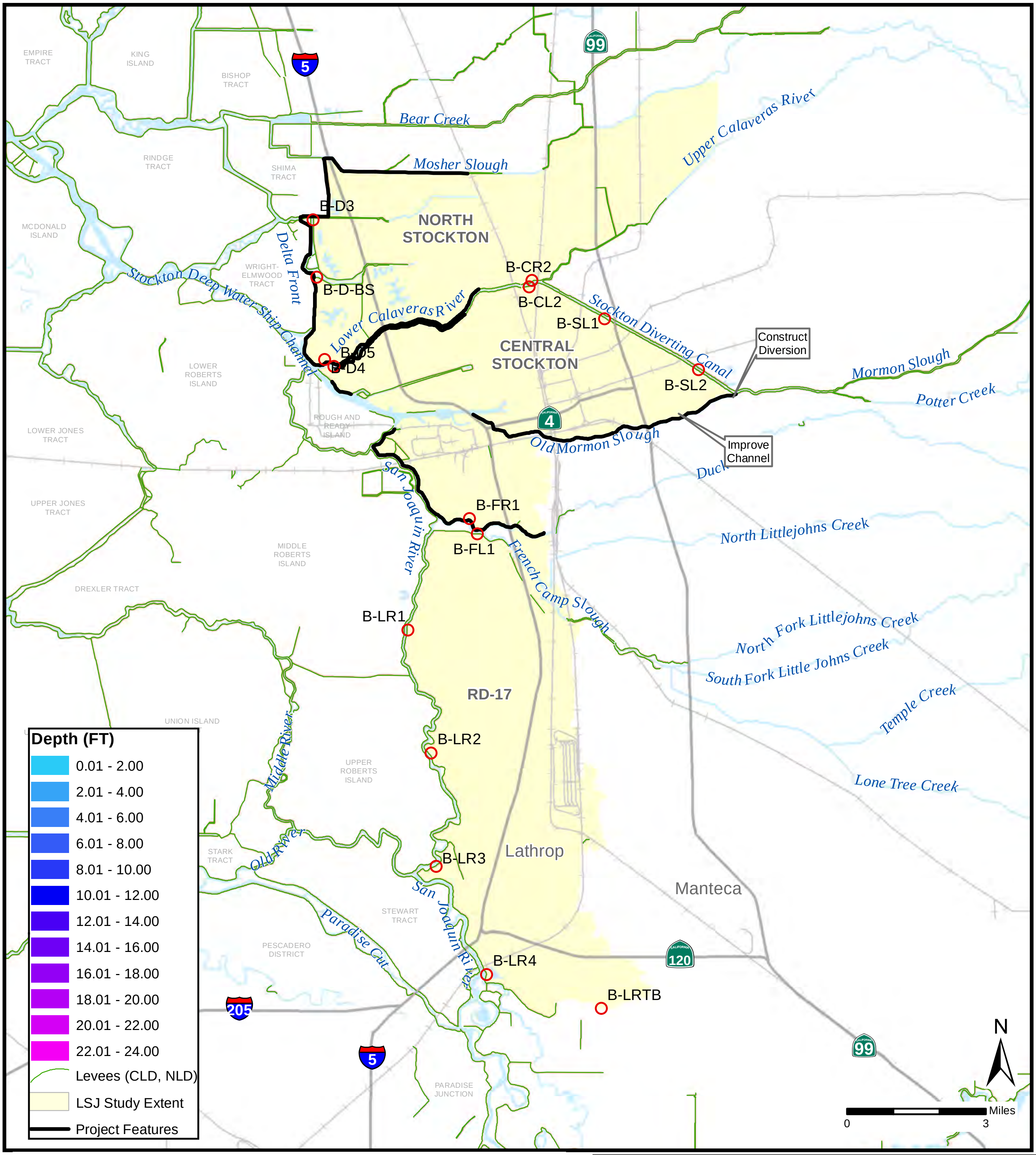
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9A

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

DEC 2015



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Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

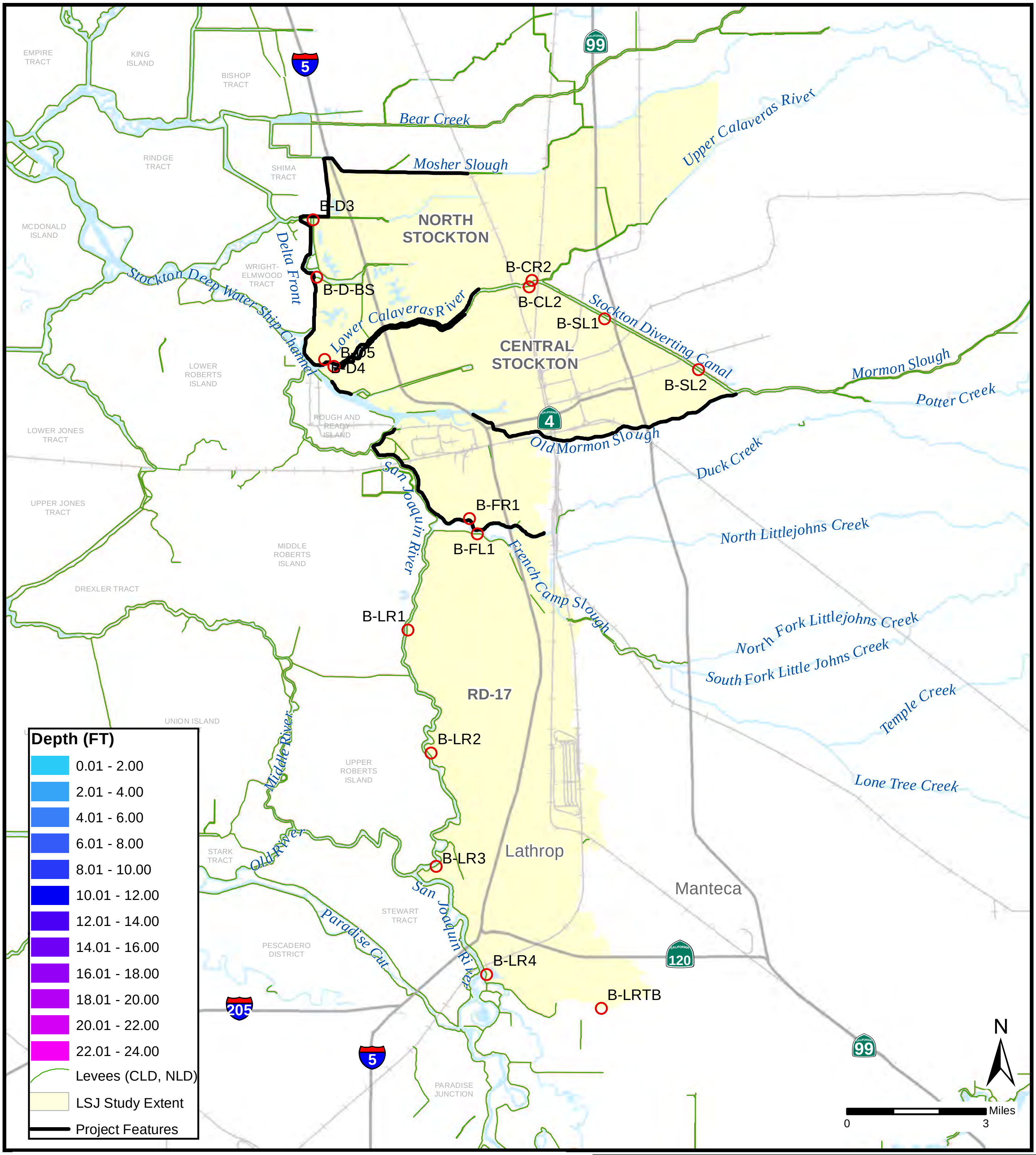
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9A 50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

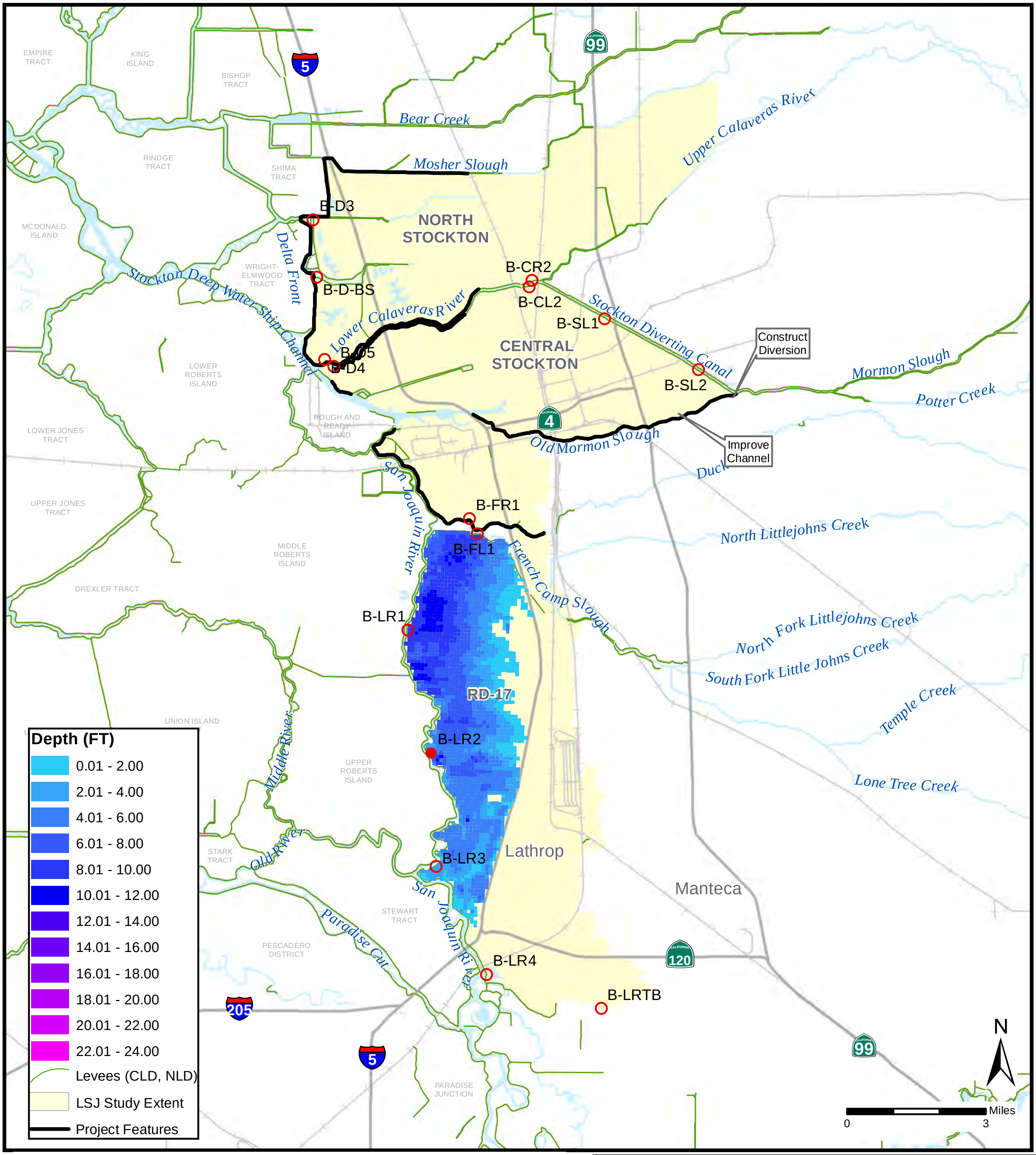
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9A 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



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Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

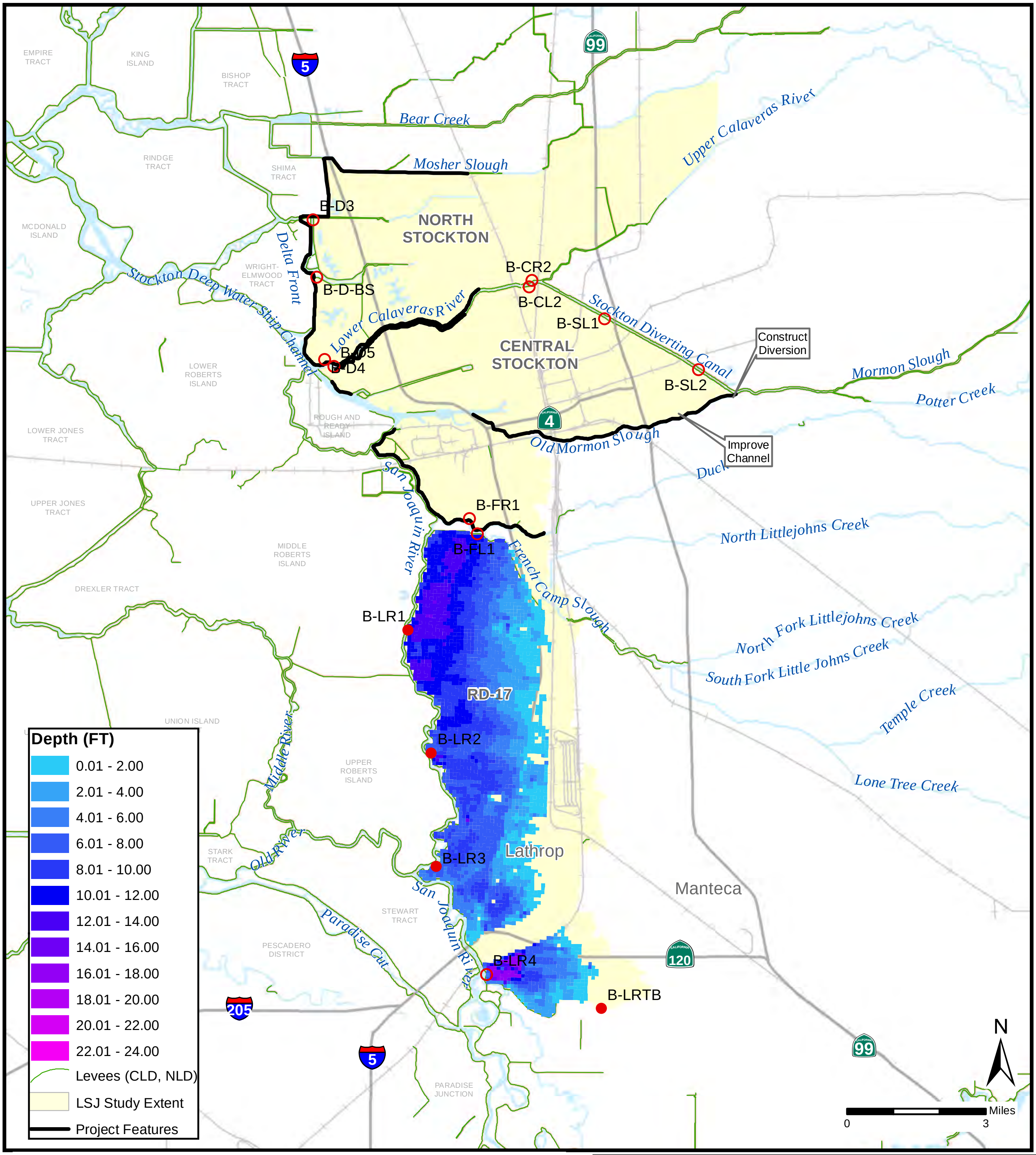
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9A
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



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Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

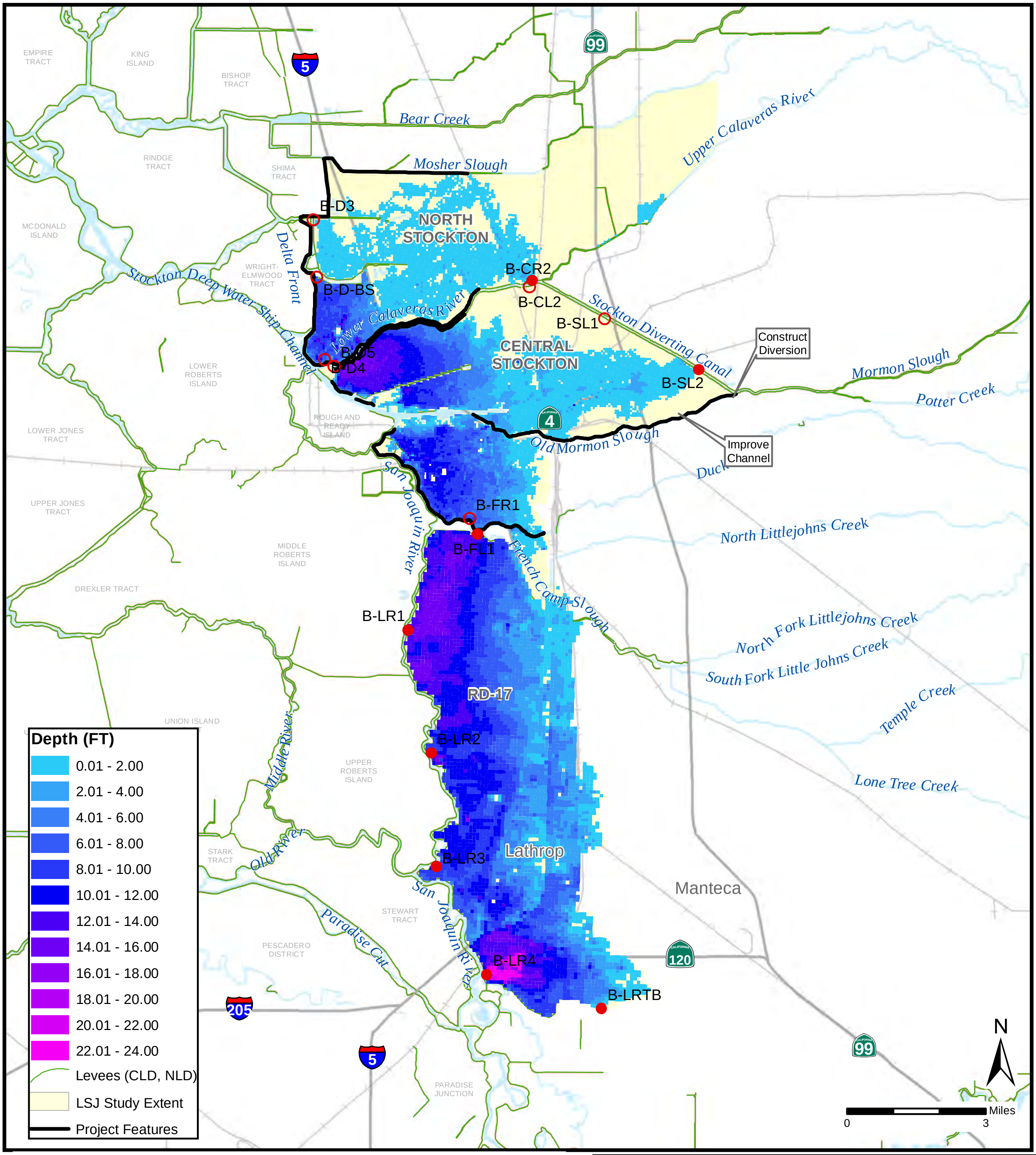
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9A 2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- ● Fails R&U Criteria
- ○ Meets R&U Criteria

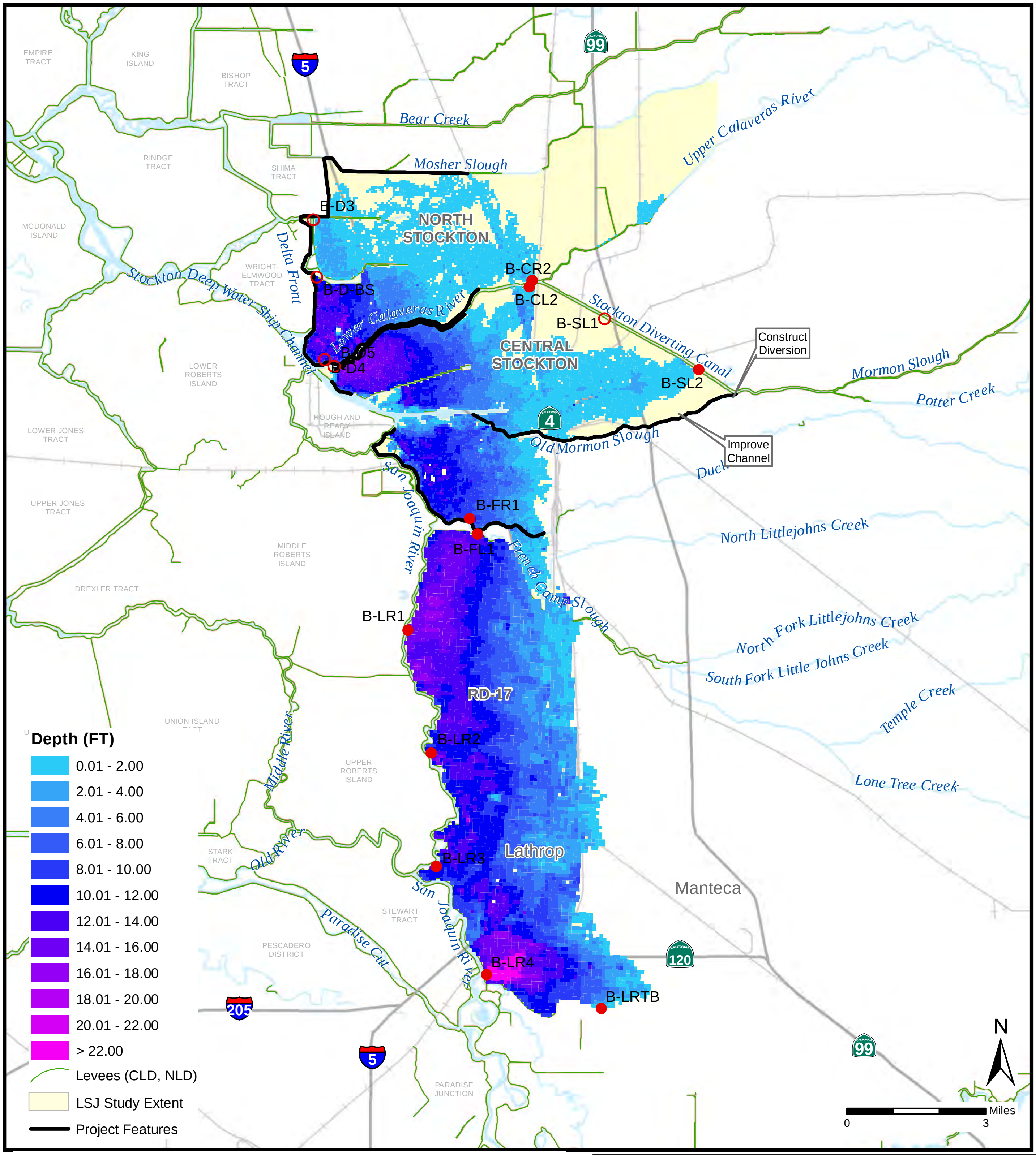
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9A 1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

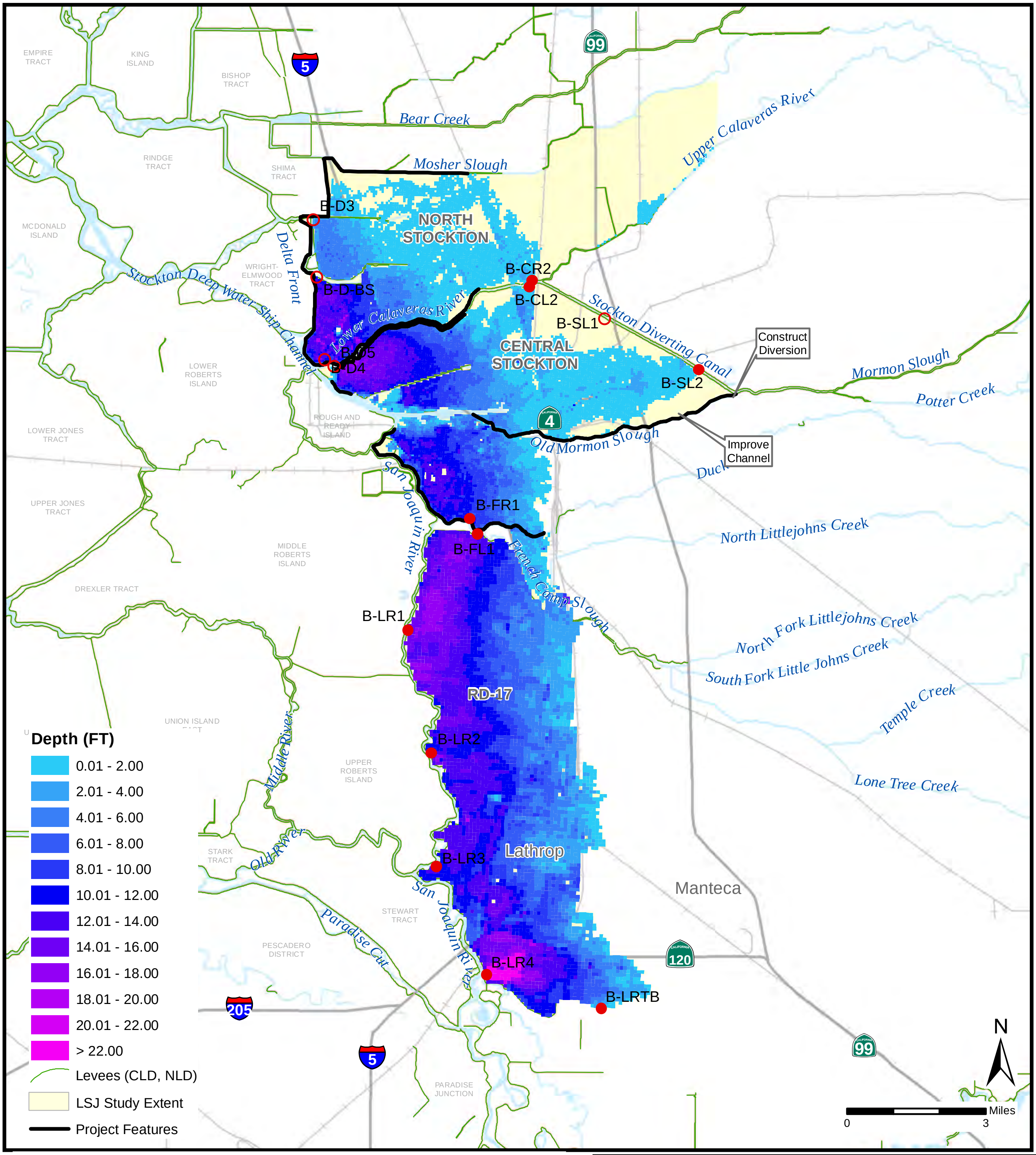
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9A 0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

OCT 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

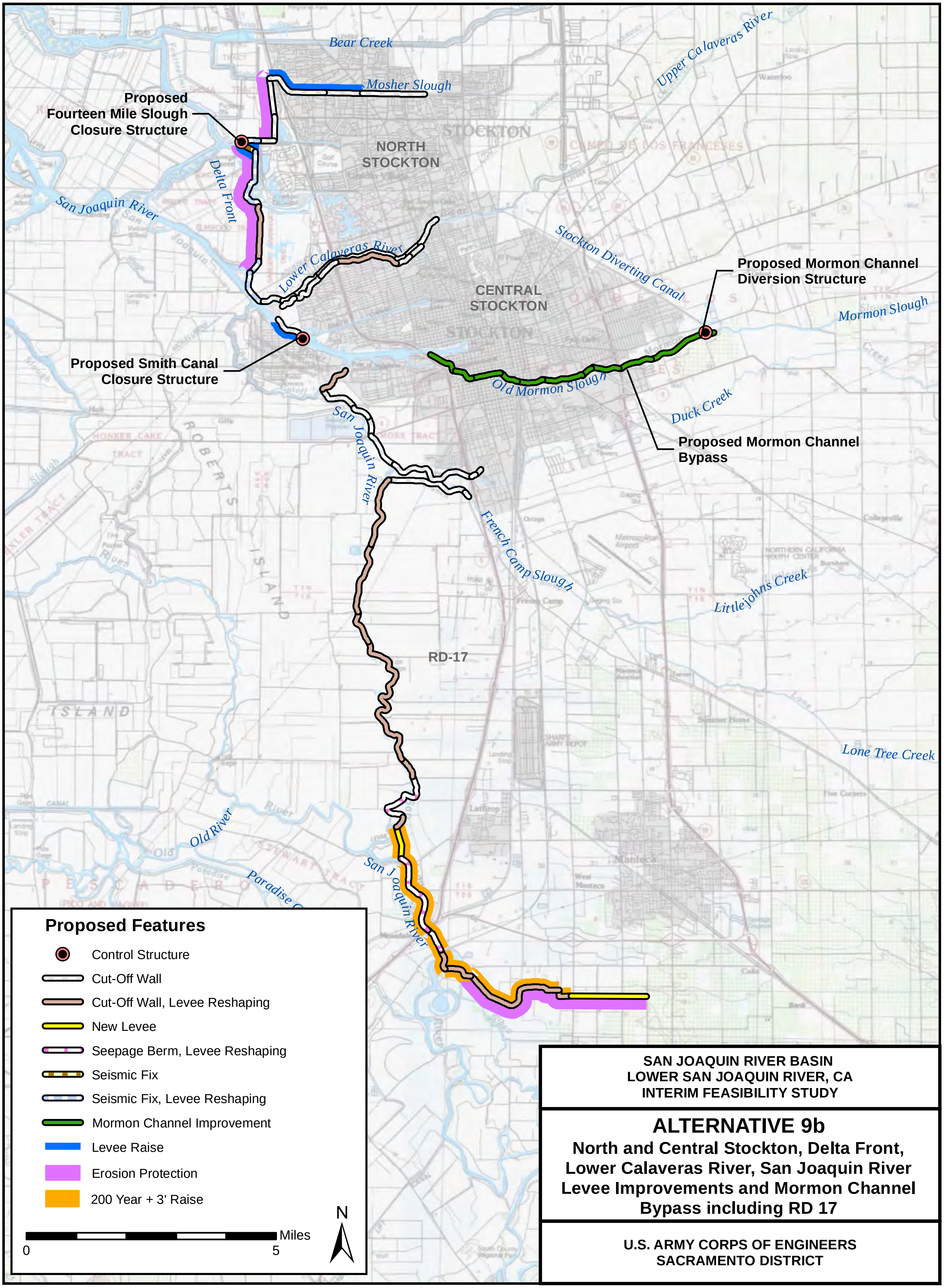
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9A
0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

OCT 2014



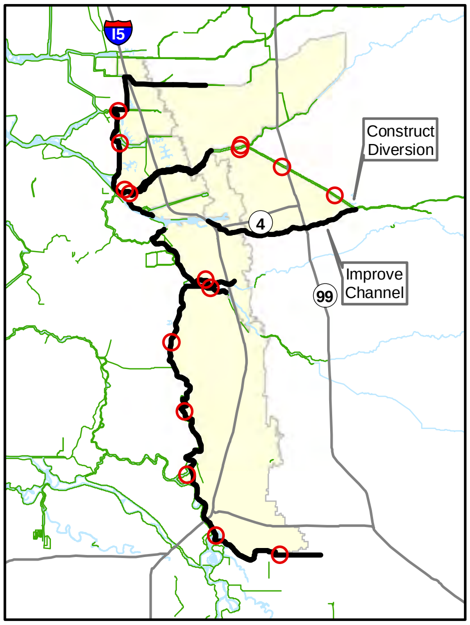
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

ALTERNATIVE 9b

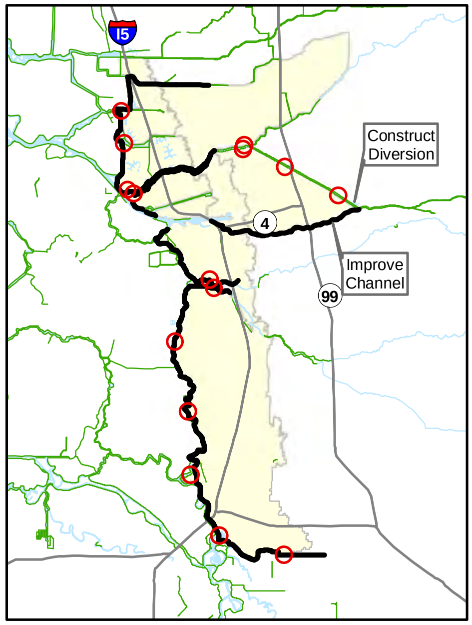
North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River Levee Improvements and Mormon Channel Bypass including RD 17

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

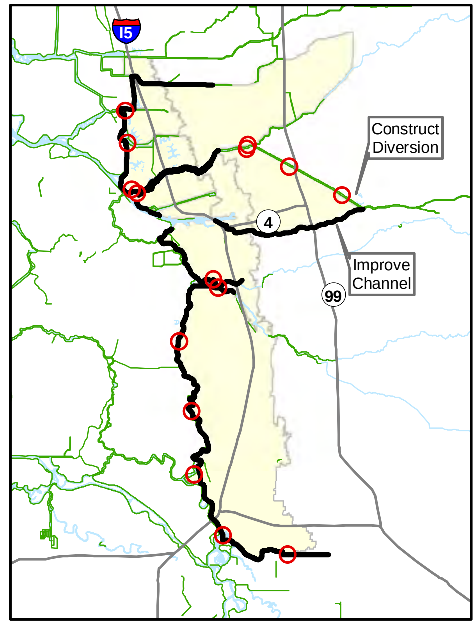
JUN 2014



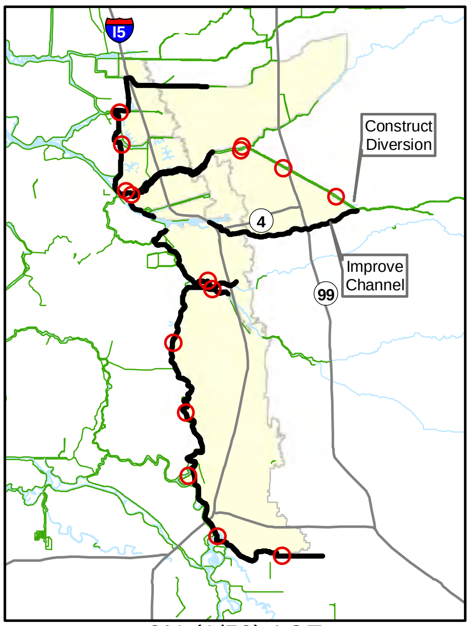
50% (1/2) ACE



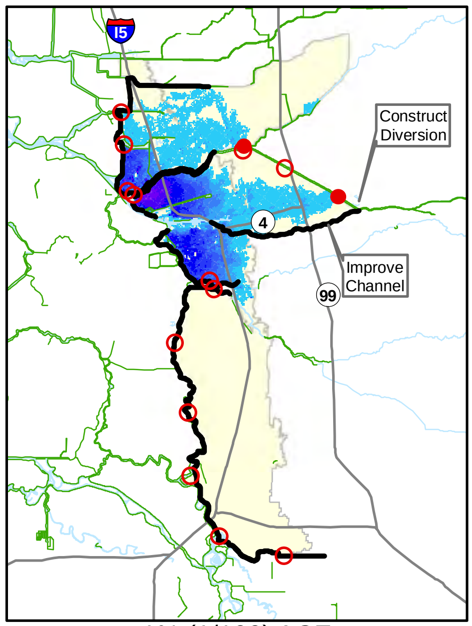
10% (1/10) ACE



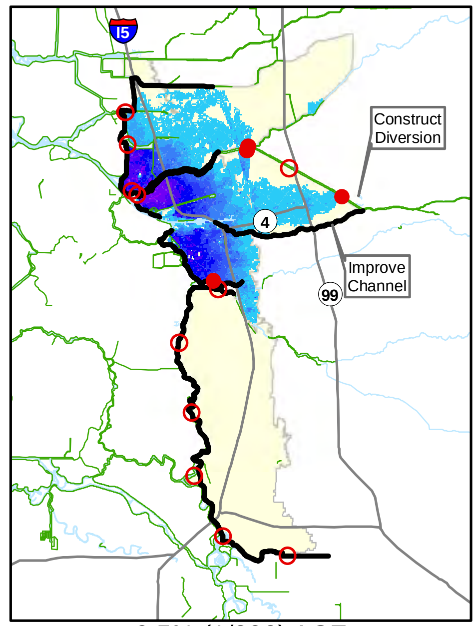
4% (1/25) ACE



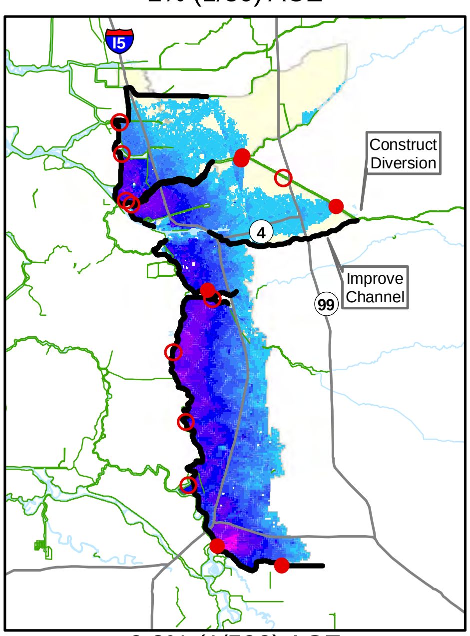
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Fails R&U Criteria
○ Meets R&U Criteria

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- > 22.0
- Levees (CLD, NLD)
- LSJ Study Extent

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

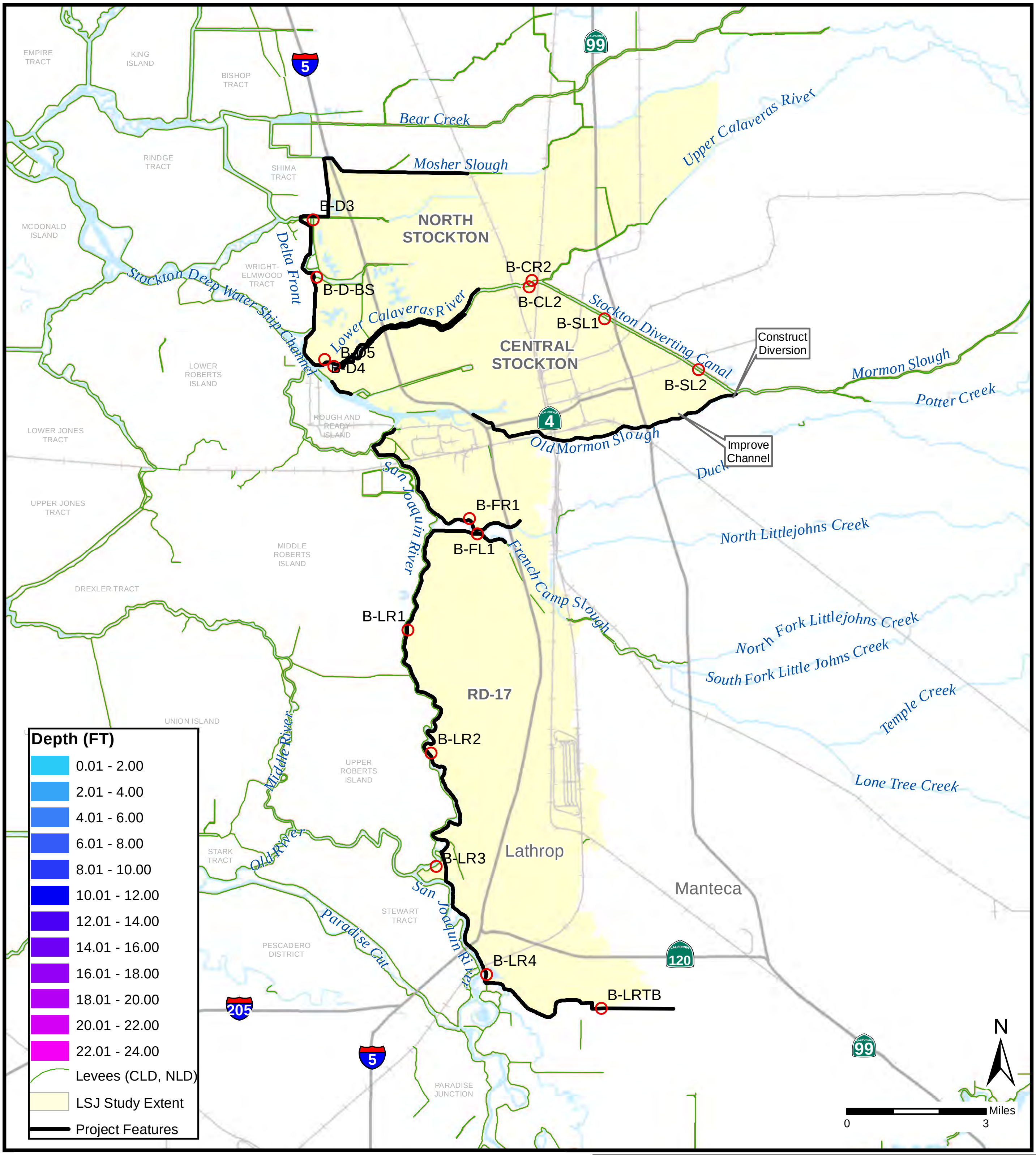
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

DEC 2015



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- ● Fails R&U Criteria
- ○ Meets R&U Criteria

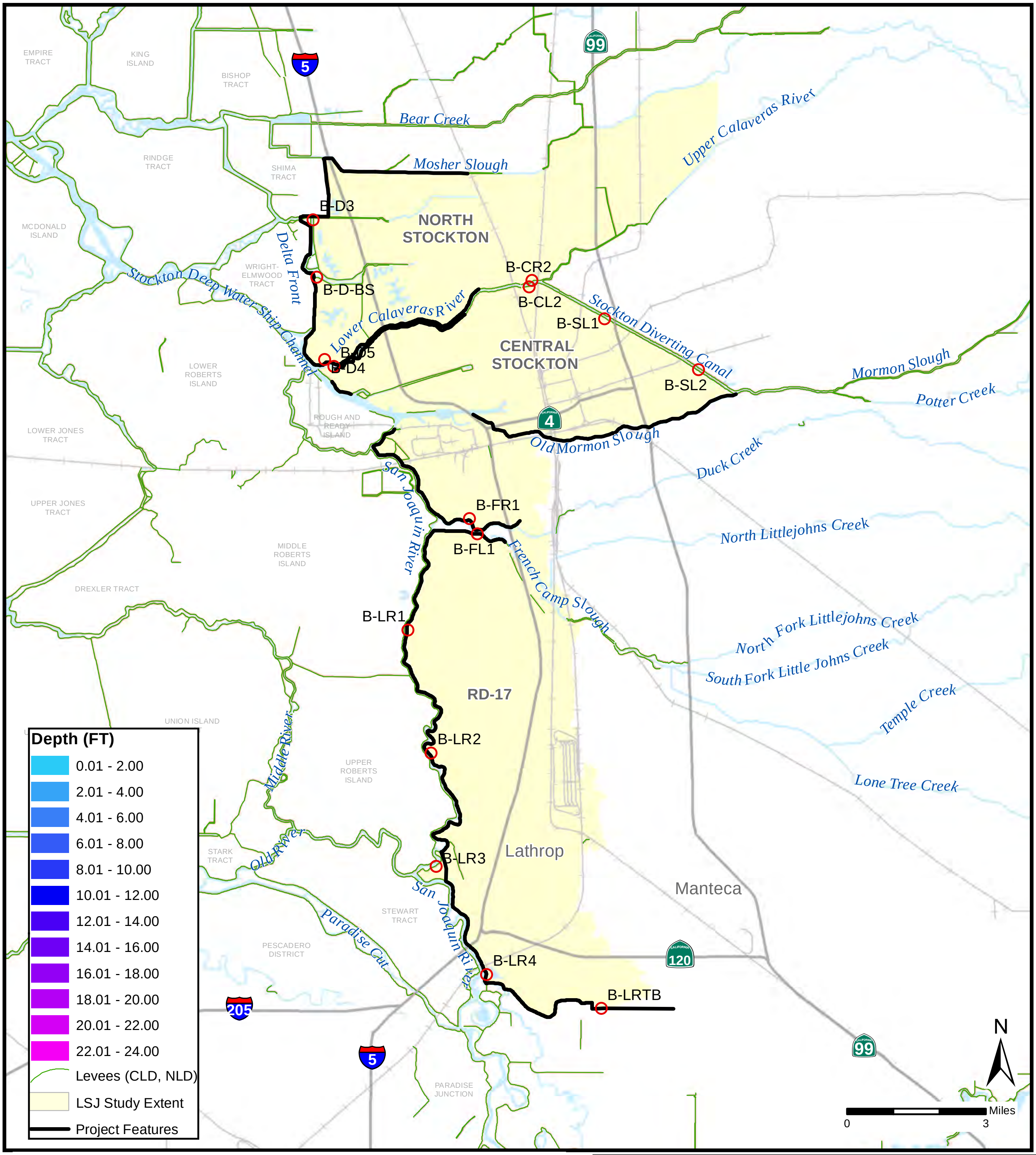
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B 50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

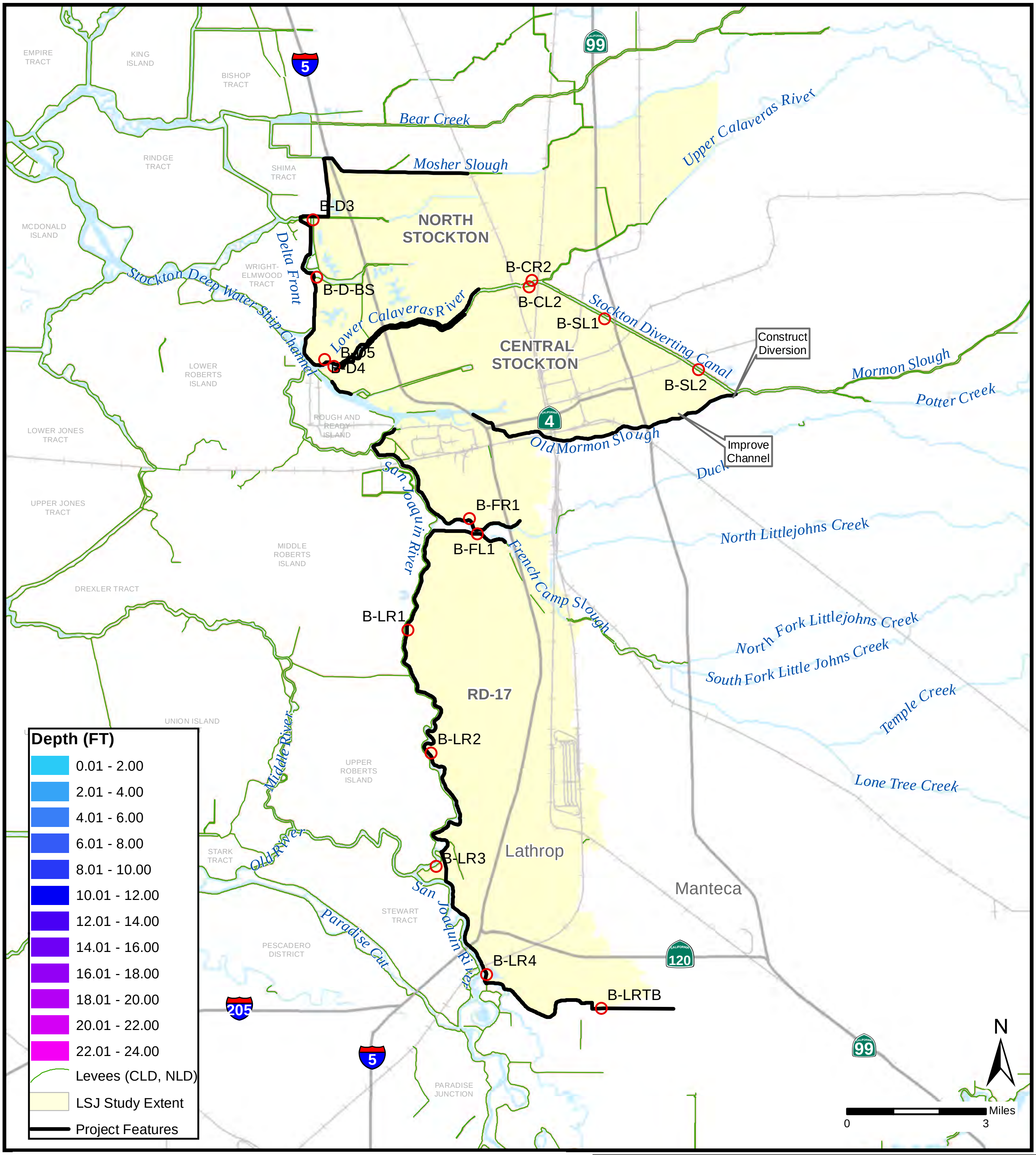
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 99



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

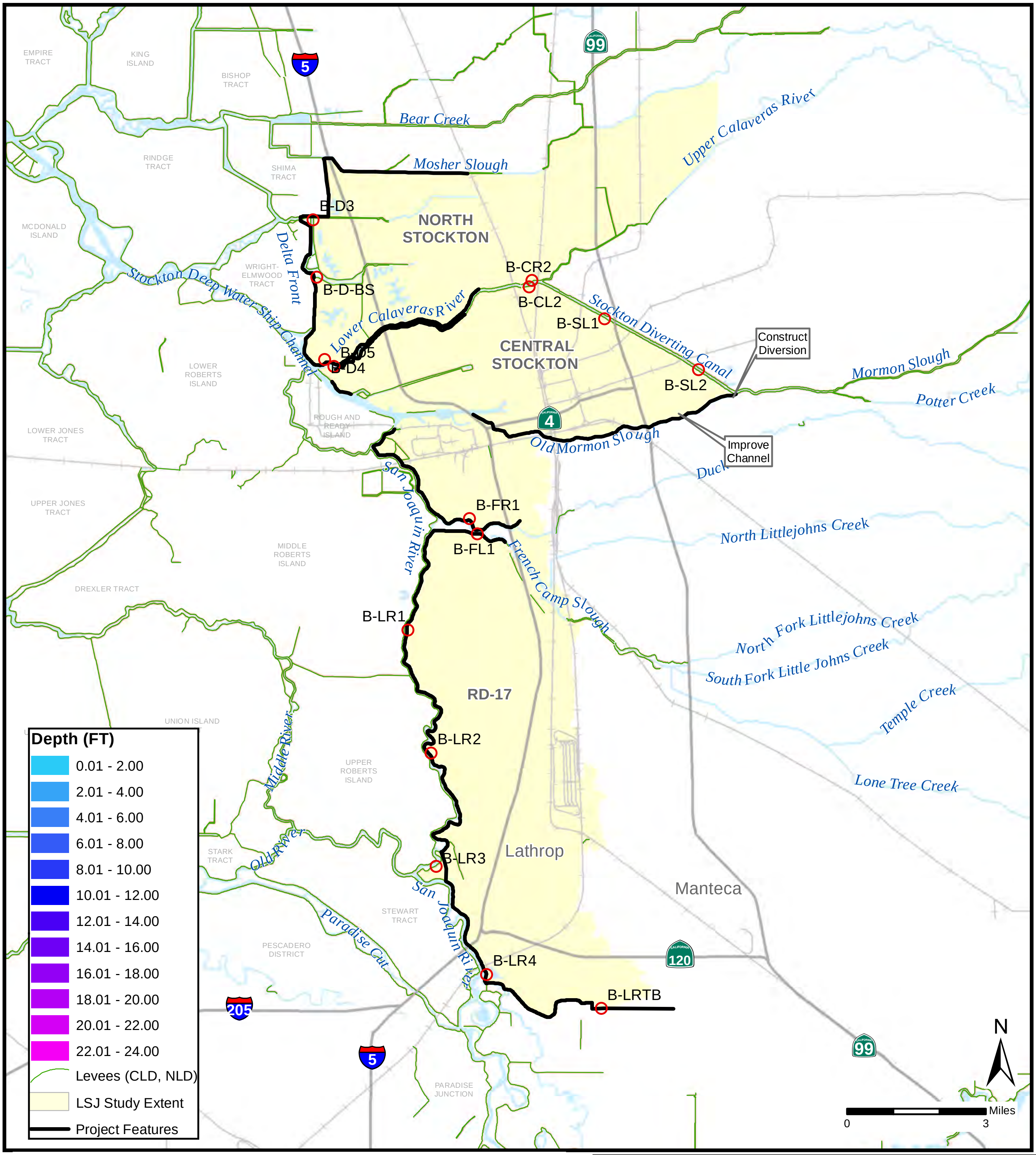
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B 4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 100



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

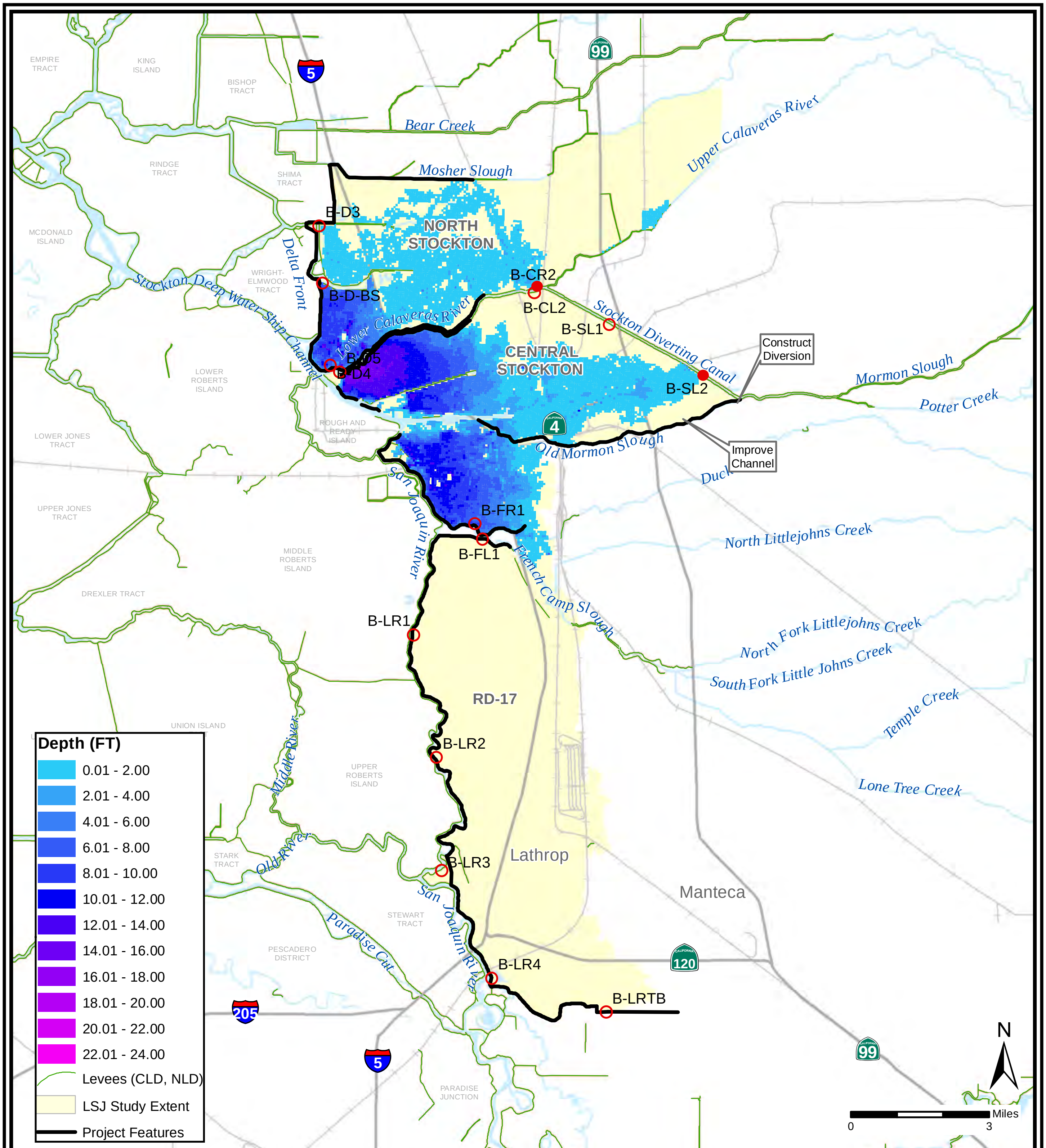
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B 2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 101



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

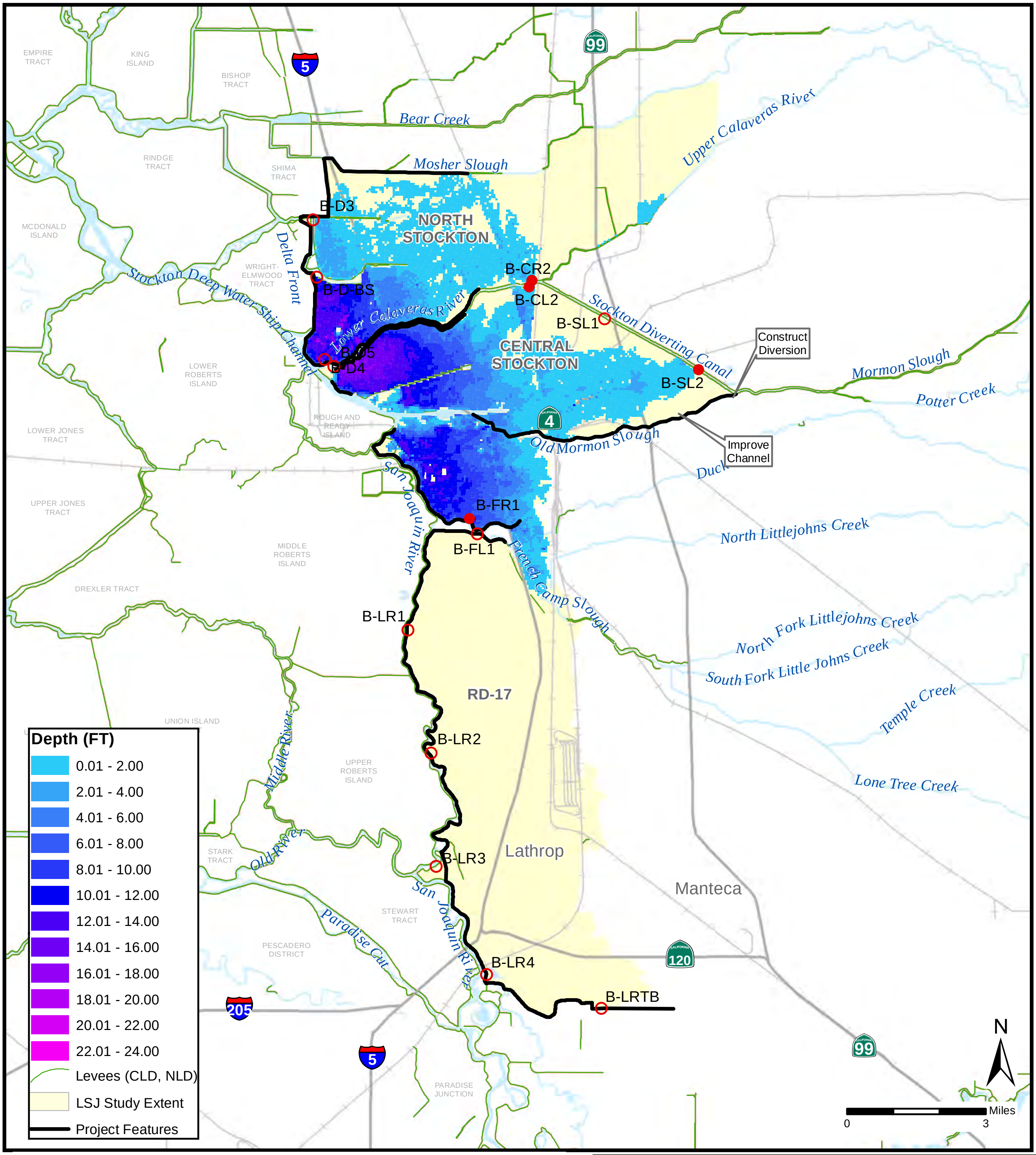
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B 1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

DEC 2015

PLATE 102



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

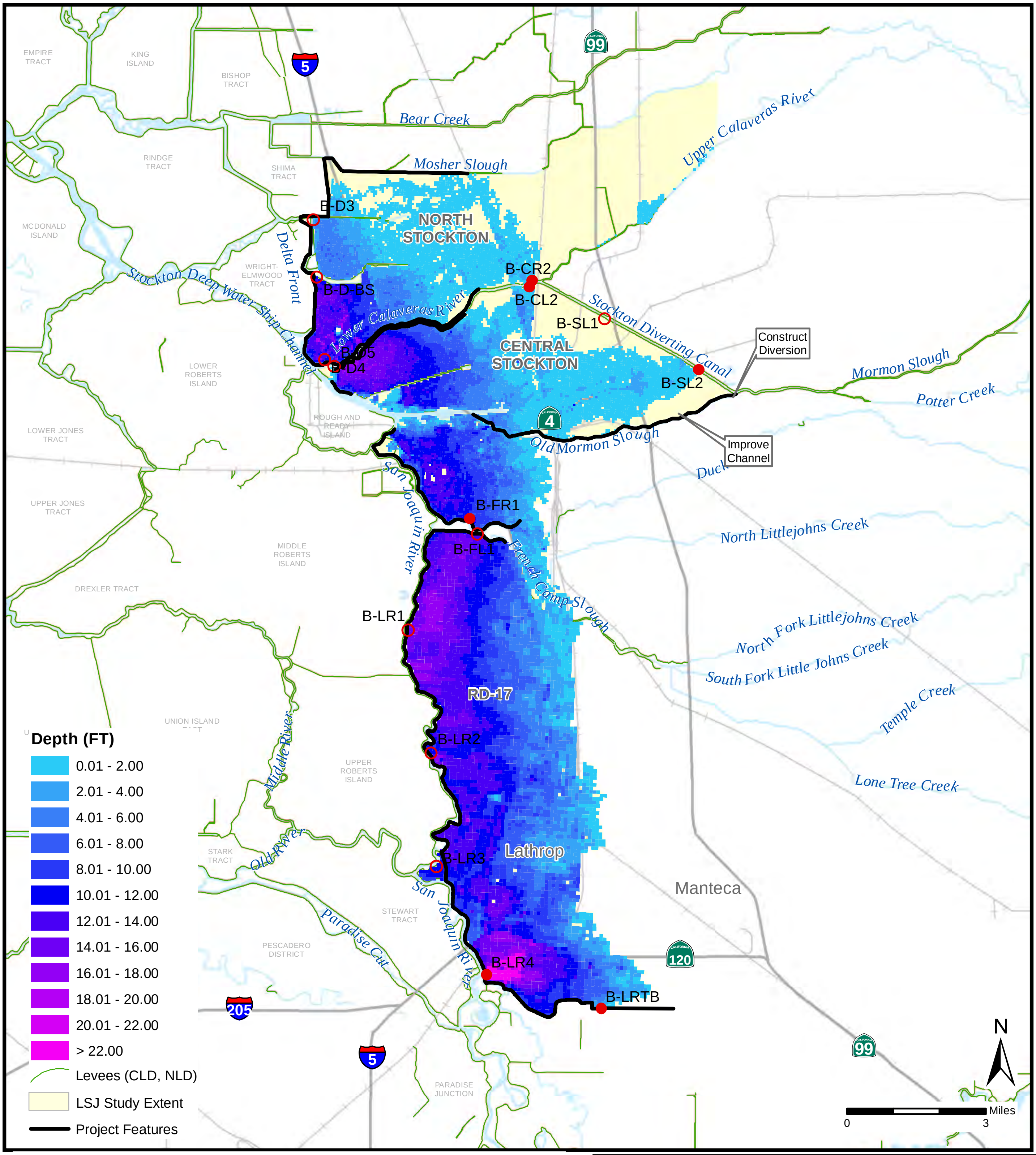
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B 0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- Fails R&U Criteria
- Meets R&U Criteria

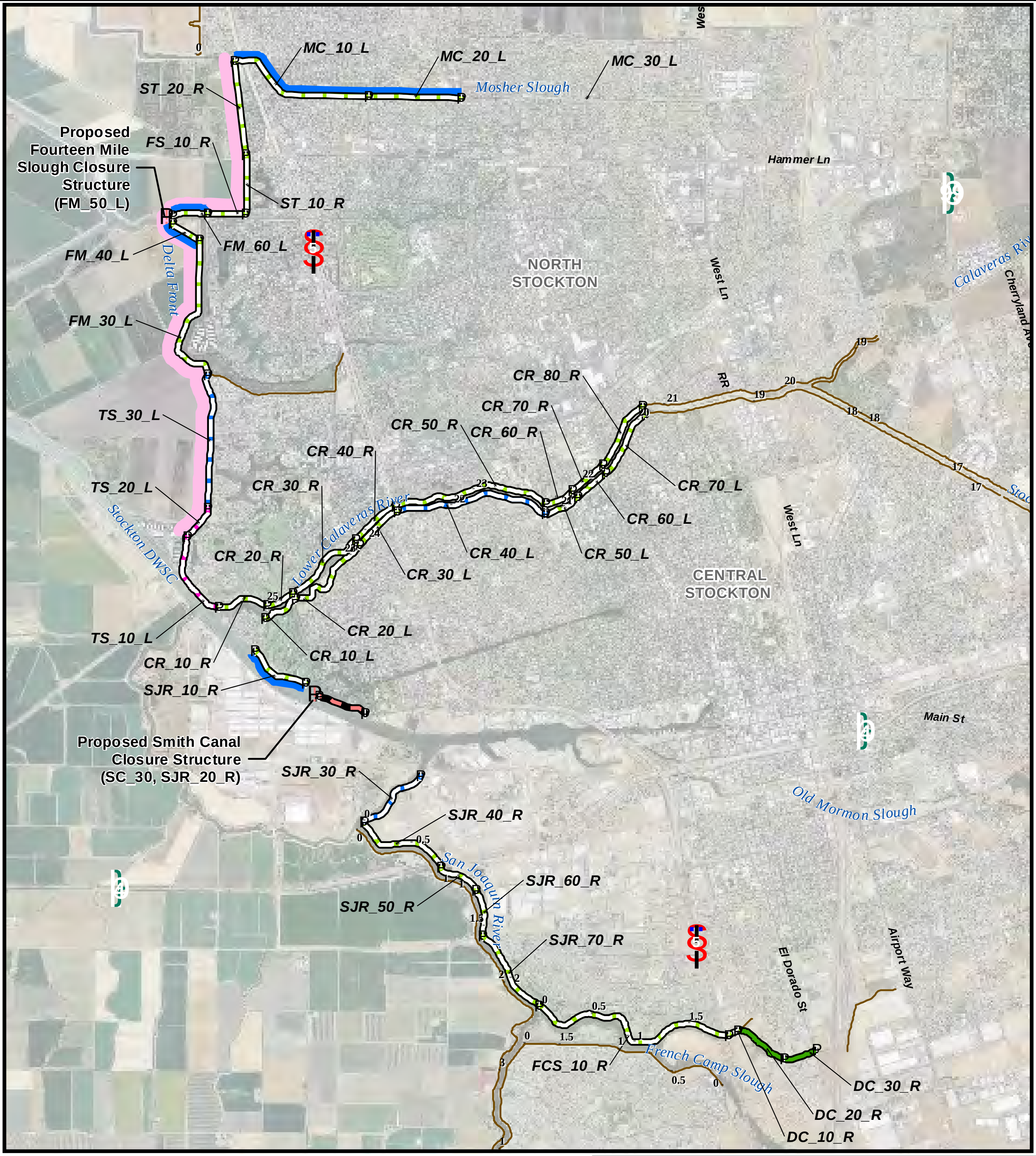
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9B
0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

OCT 2014



Control Structure
Cut-Off Wall
Cut-Off Wall, Levee Reshaping
Seismic Fix
Seismic Fix, Levee Reshaping
Adjacent Seismic Fix
New Levee
Smith Canal Improvements

Levee Raise
Erosion Protection
NLD Federal Levees
Levee Mile

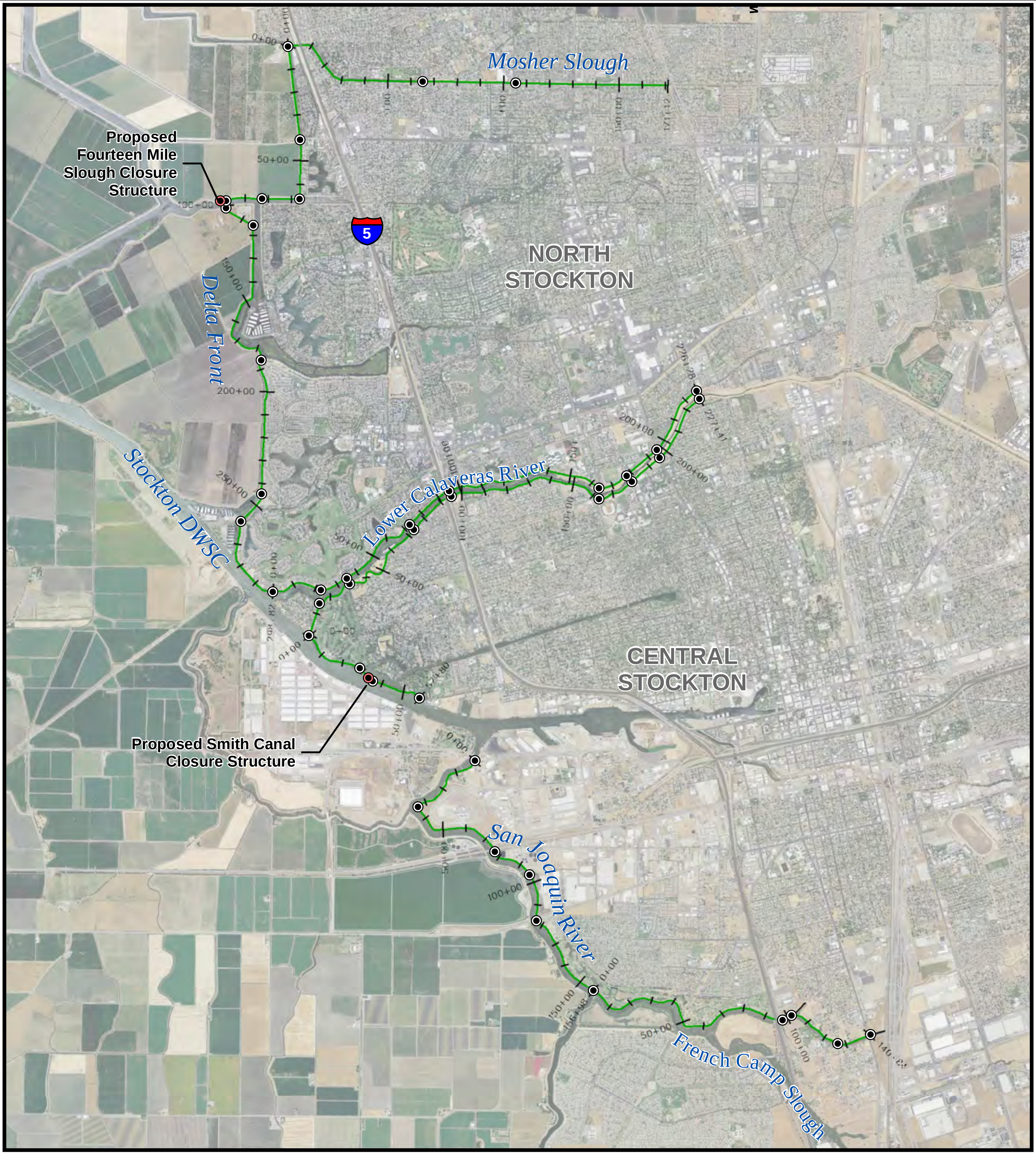
Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

LSJ LEVEE REACHES ALTERNATIVE 7a

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



Legend

- Stations
- Project Alignment
- Control Structure

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

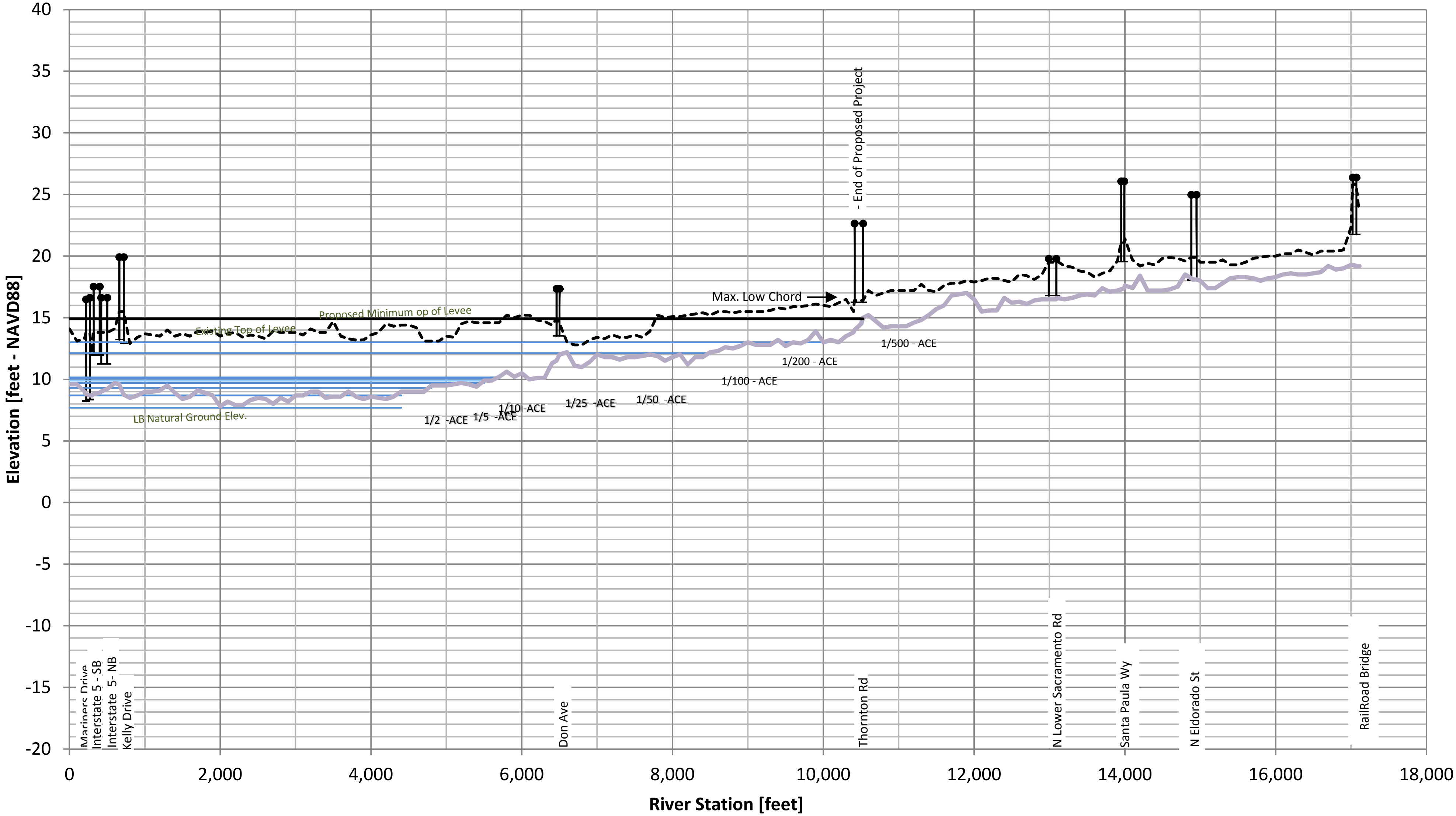
N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

LSJ PROJECT ALIGNMENT ALTERNATIVE 7a

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

DEC 2015



Notes:
1) Water surface profiles based on Delta peak stages only. Peak Stage profiles for Mosher Slough are not included.

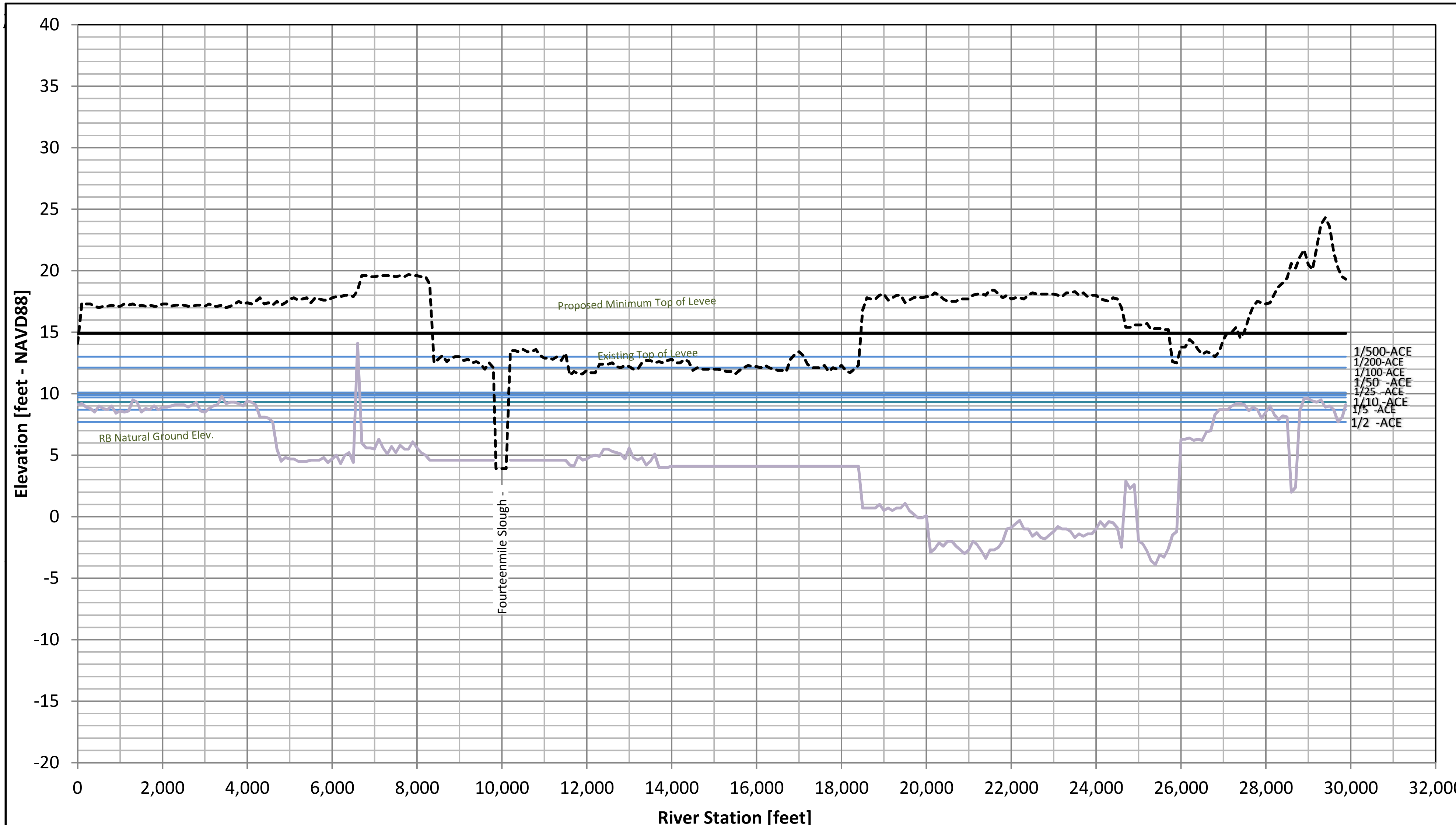
2) Levee would be reconstructed to Existing Levee Height or Minmum Levee Height, whichever is greater.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Recommended Plan Levee Profile
Mosher Slough Left Bank From Fourteen Mile Slough to RailRaod Bridge

United States Army Corps of Engineers
Sacramento District

Dec 2015



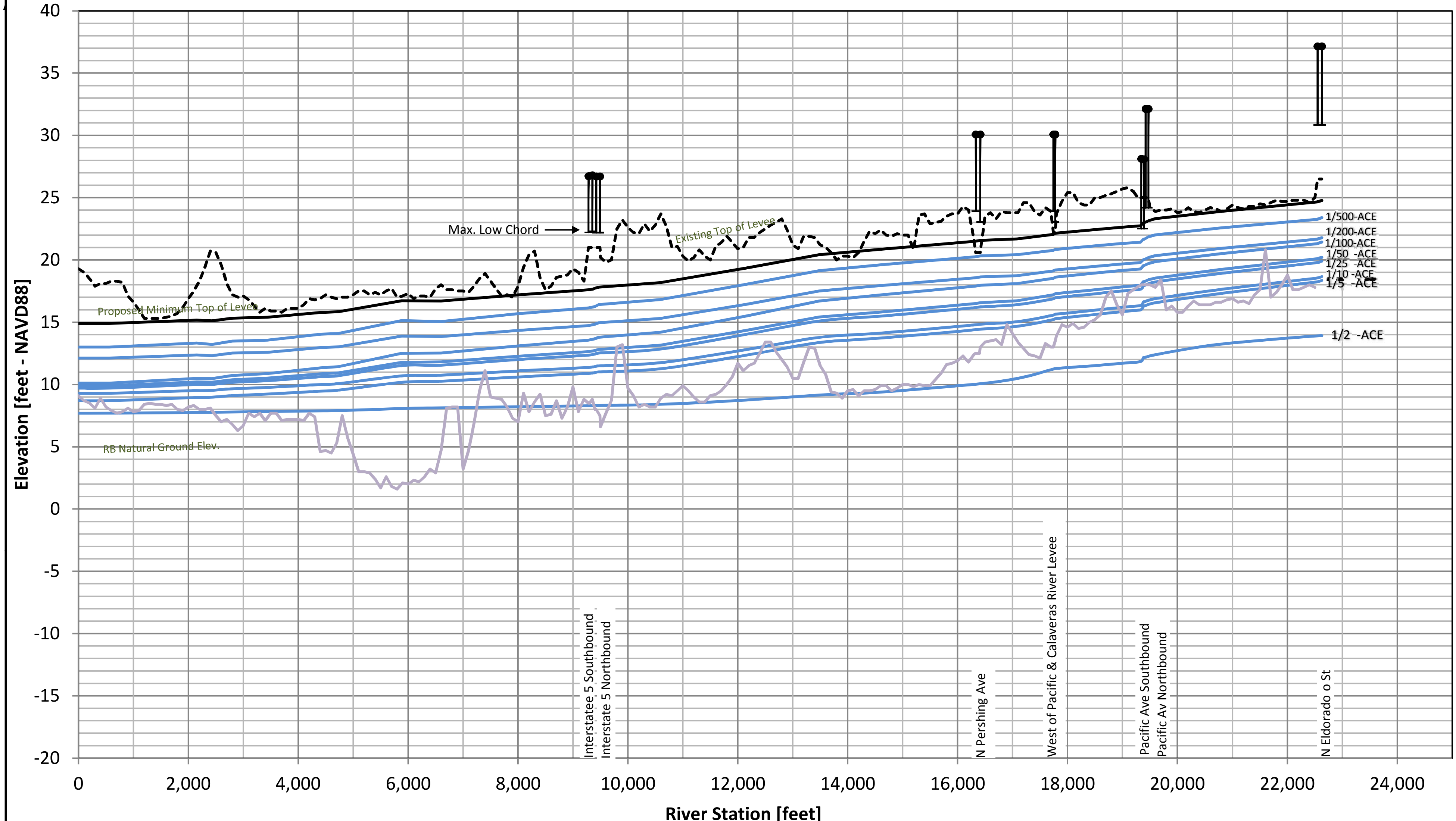
Notes:

1) Levee would be reconstructed to Existing Levee Height or Minmum Levee Height, whichever is greater.

SAN JOAQUIN RIVER BASIN LOWER SAN JOAQUIN RIVER, CA INTERIUM FEASIBILITY STUDY
Recommended Plan Levee Profile Delta Front Levee From Mosher Slough to Calaveras River
United States Army Corps of Engineers Sacramento District

Dec 2015

Plate 107b



Notes:

1) Levee would be reconstructed to Existing Levee Height or Minmum Levee Height, whichever is greater.

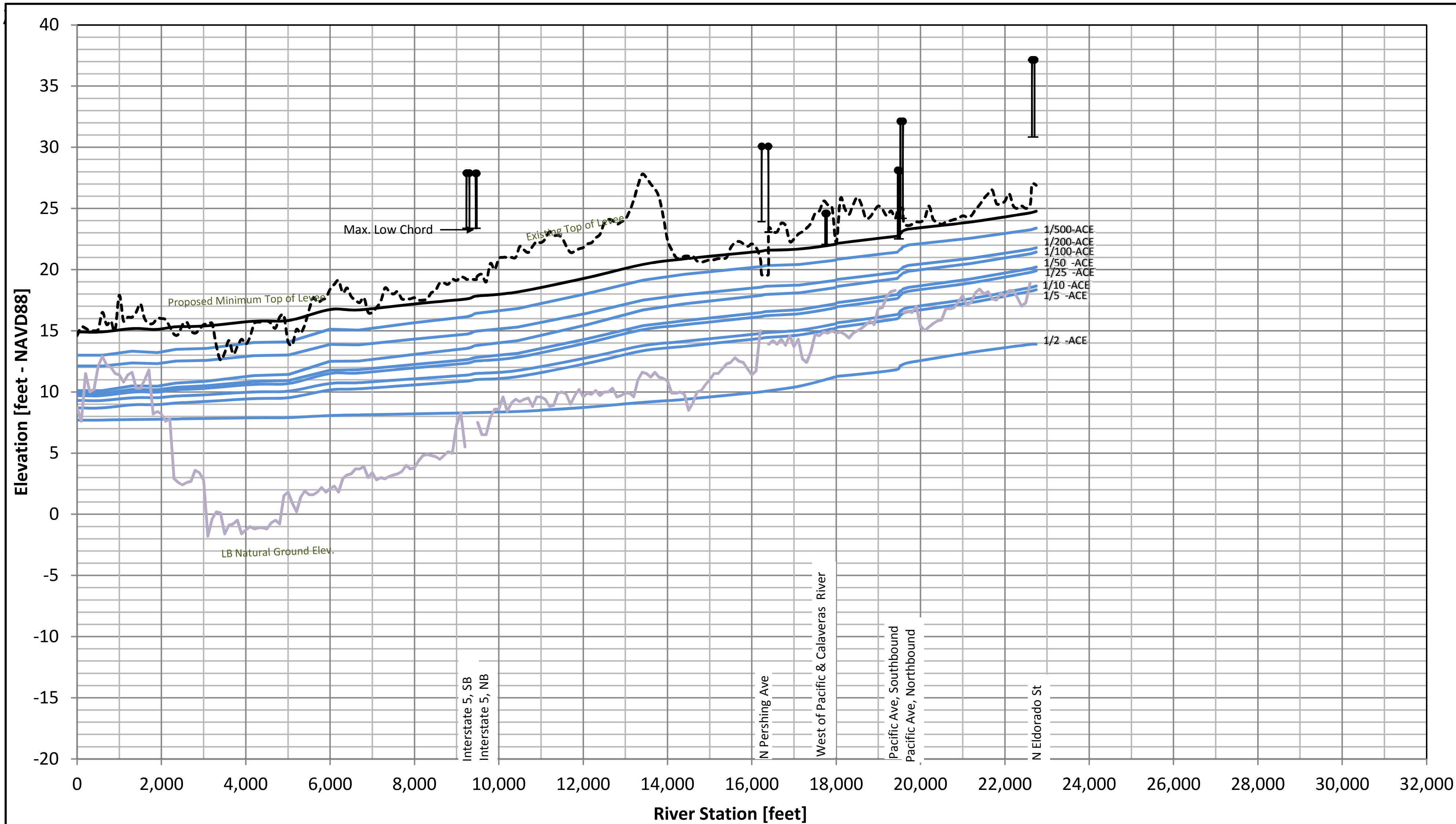
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Recommended Plan Levee Profile
Calaveras River Right Bank From
San Joaquin River to N El Dorado St

United States Army Corps of Engineers
Sacramento District

Dec 2015

Plate 107c



Notes:

1) Levee would be reconstructed to Existing Levee Height or Minmum Levee Height, whichever is greater.

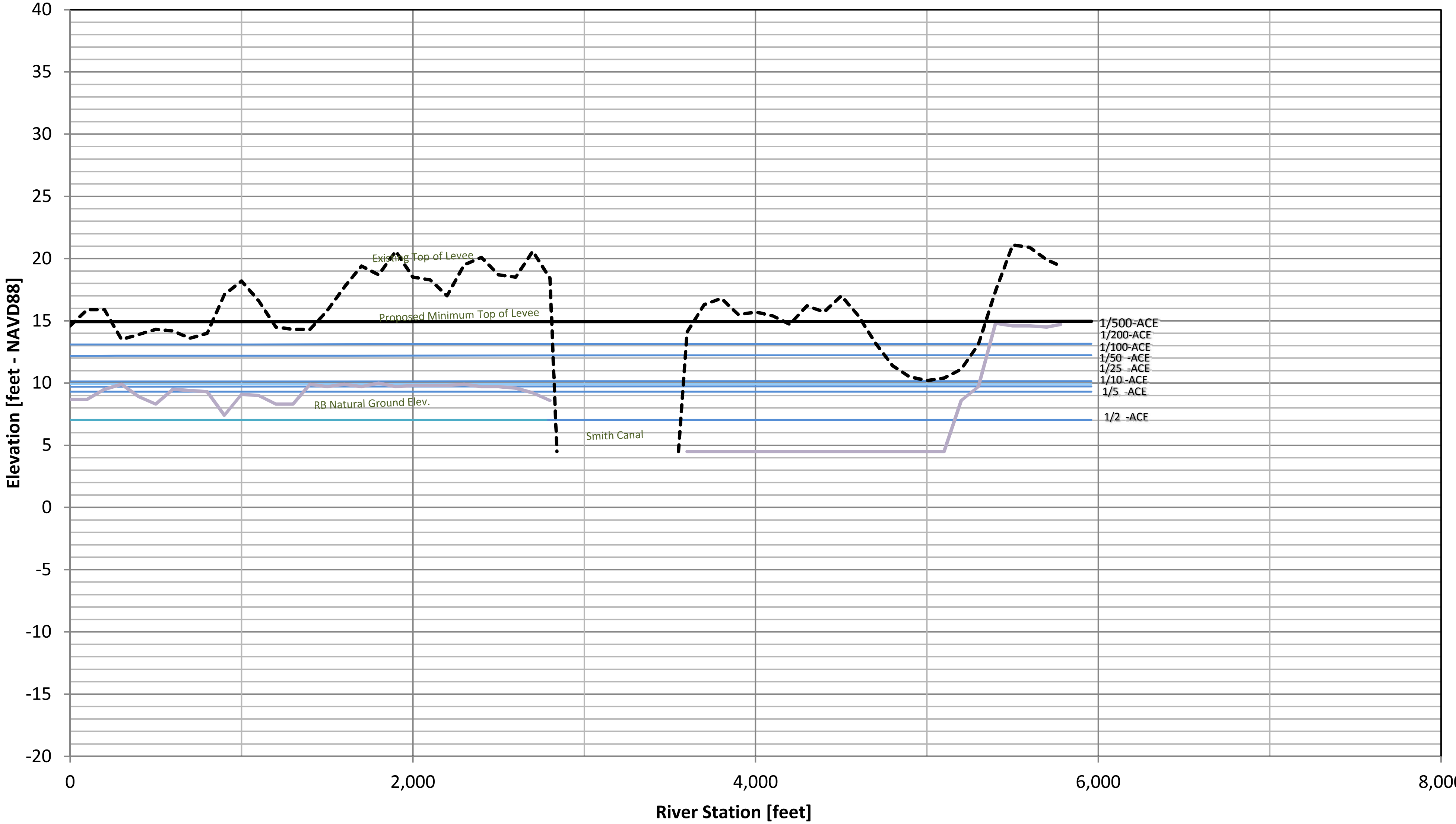
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Recommended Plan Levee Profile
Calaveras River Left Bank From San Joquin
To N Eldorado St

United States Army Corps of Engineers
Sacramento District

Dec 2015

PLATE 107d



Notes:

1) Levee would be reconstructed to Existing Levee Height or Minmum Levee Height, whichever is greater.

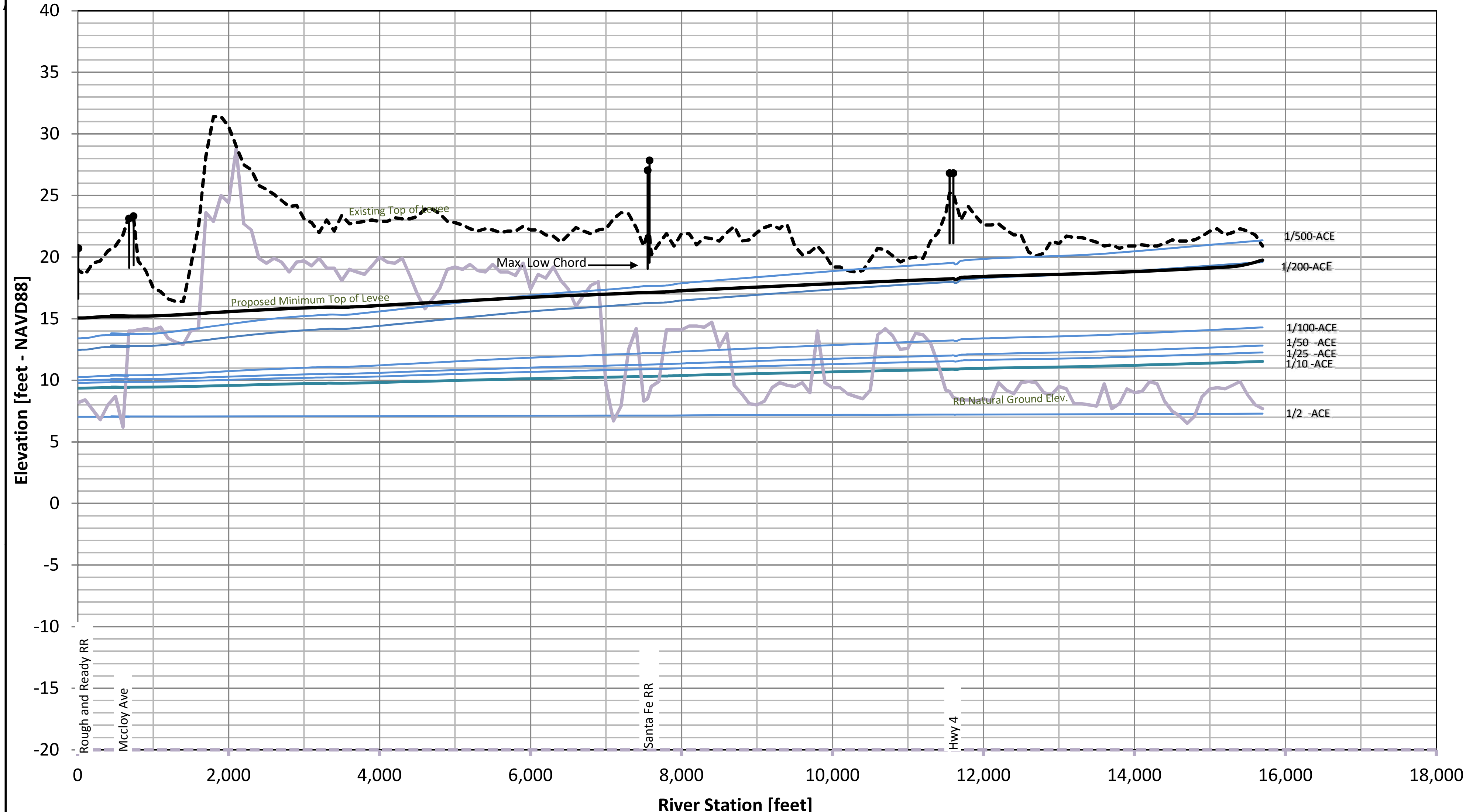
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Levee Profile
San Joaquin River Right Bank From
Calaveras River to Turning Basin

United States Army Corps of Engineers
Sacramento District

Dec 2015

PLATE 107e



Notes:

1) Levee would be reconstructed to Existing Levee Height or Minmum Levee Height, whichever is greater.

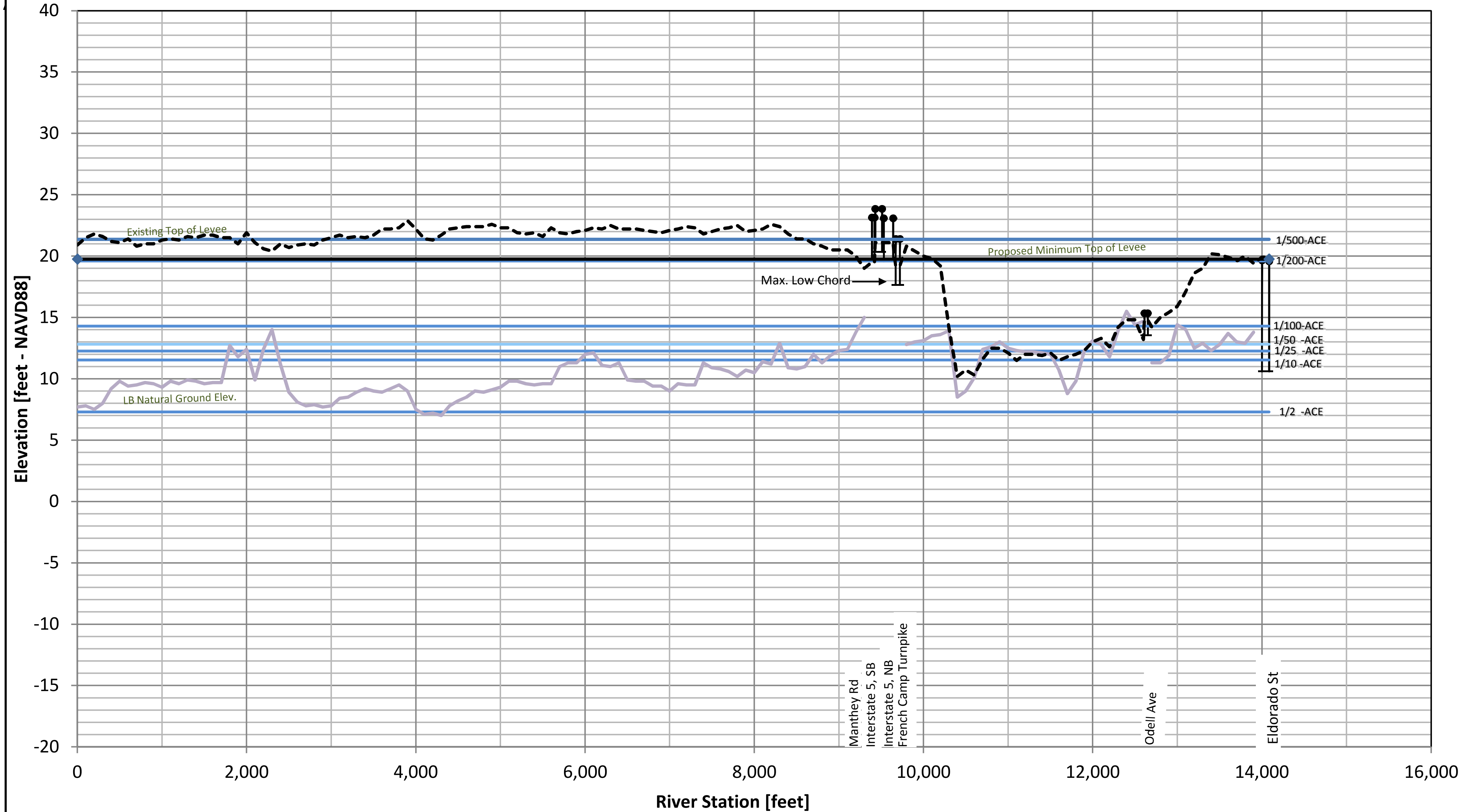
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Levee Profile
San Joaquin River Right Bank
From Turning Basin to French Camp Slough

United States Army Corps of Engineers
Sacramento District

Dec 2015

PLATE 107f



Notes:

1) Levee would be reconstructed to Existing Levee Height or Minmum Levee Height, whichever is greater.

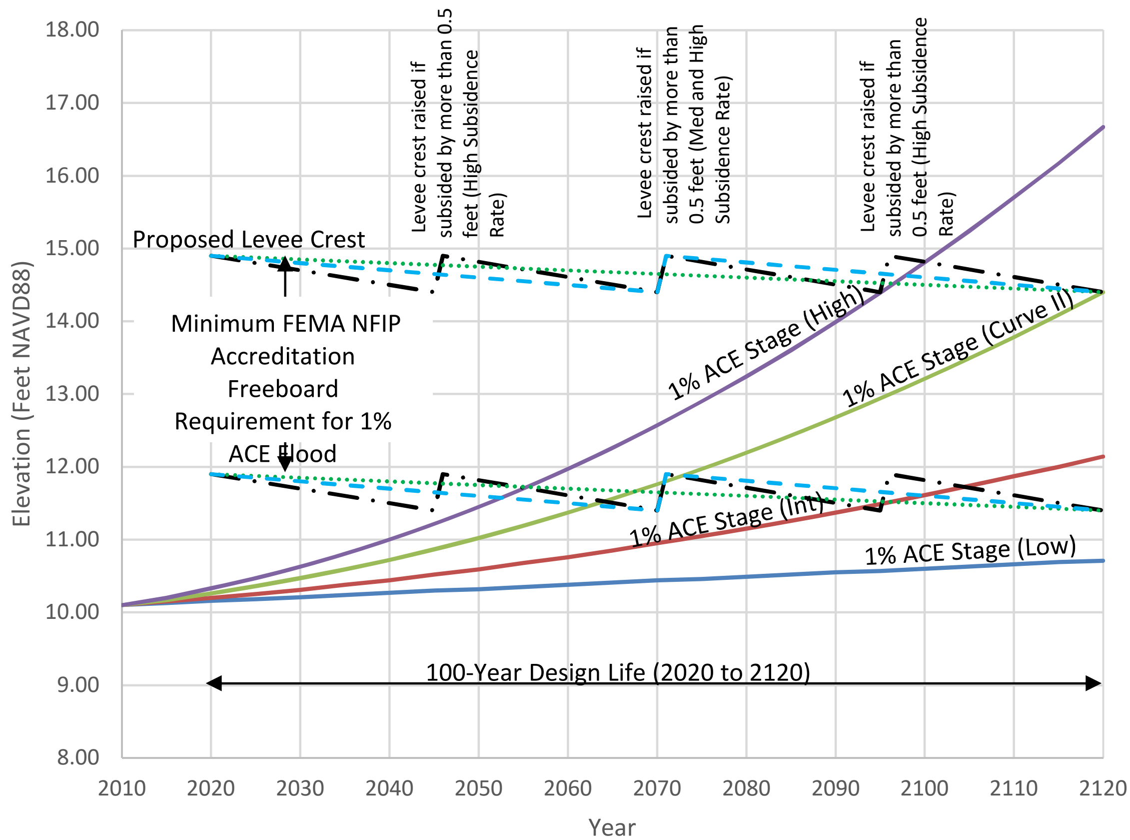
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Levee Profile
French Camp Slough Right Bank
From San Joaquin to UPRR

United States Army Corps of Engineers
Sacramento District

Dec 2015

Plate 106g



Year Water Surface Expected to Exceed Minimum Freeboard Requirement
Assuming Levee Crest Restoration after 0.5 feet of subsidence is observed.

Rate of Sea Level Change	Rate of Levee Subsidence Low	Medium	High
Low	>2120	>2120	>2120
Intermediate	2097	2100	2093
Curve II	2067	2063	2065
High	2055	2053	2055

Notes: Minimum freeboard requirment for Federal Emergency Managment Agency (FEMA) National Flood Inurance Program (NFIP) levee accreditation was not a design objective for sizing the recommended plan. These minimums are shown to demonstrate the potential change in performance of the recommended plan over time relative to a standard water surface elevation metric.
Low, Intermediate, Curve II, and High Rates of Sea Level Change based on ER 1110-2-8162.
Levee Crest assumed to be restored after 0.5 feet of observed subsidence.
Low levee crest subsidence assumed to be 0.005 ft per year.
Medium levee crest subsidence assumed to be 0.010 ft per year
High levee crest subsidence assumed to be 0.020 ft per year

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

YEAR 1% WATER SURFACE EXPECTED TO EXCEED
FEMA NFIP FREEBOARD REQUIREMENT FOR LEVEE ACCREDITATION AT D3 INDEX POINT

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

ATTACHMENT A

GEOTECHNICAL FRAGILITY CURVES

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

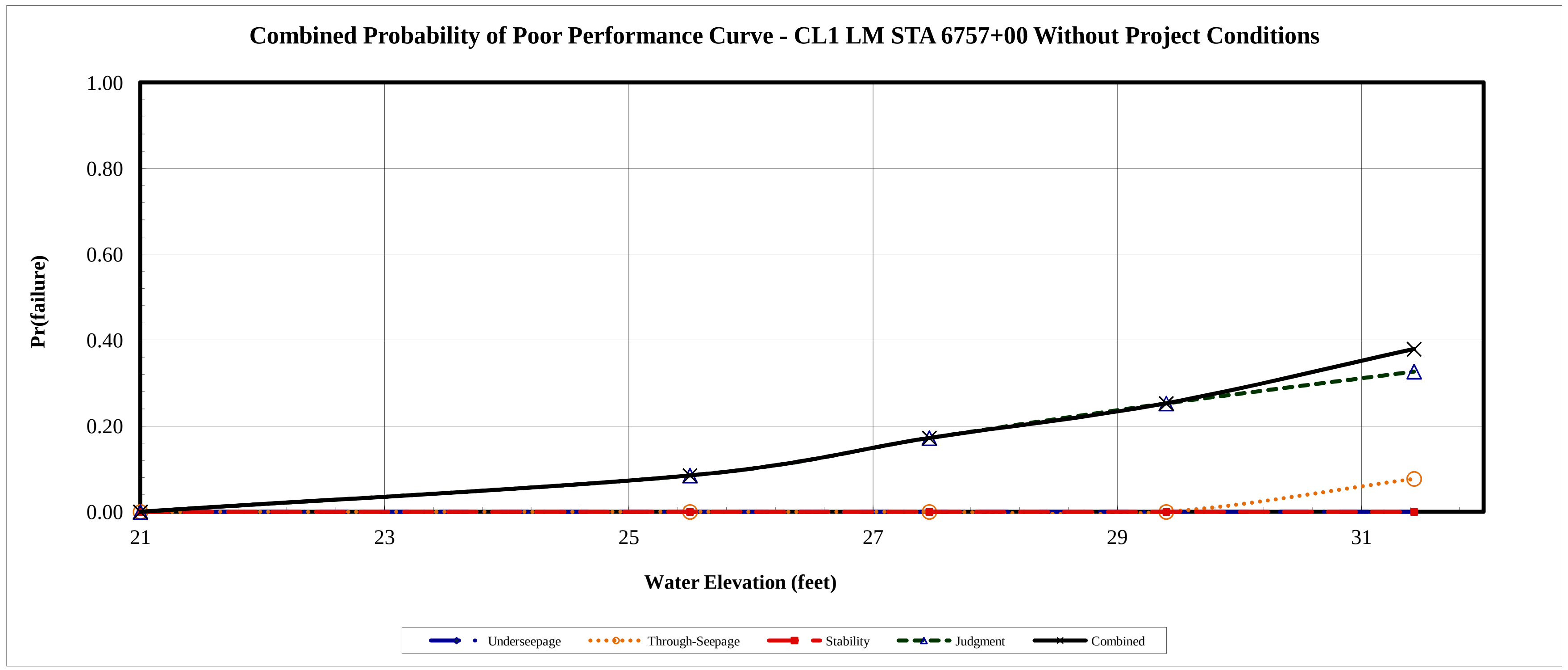
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: CL1

Levee Mile: STA 6757+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 31.43
L/S Toe Elev.: 21.00
W/S Toe Elev.: 26.94

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/24/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
21.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
25.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0845	0.9155	0.0845	0.9155
27.46	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1719	0.8281	0.1719	0.8281
29.40	0.0001	0.9999	0.0000	1.0000	0.0000	1.0000	0.2526	0.7474	0.2527	0.7473
31.43	0.0004	0.9996	0.0769	0.9231	0.0001	0.9999	0.3268	0.6732	0.3790	0.6210



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

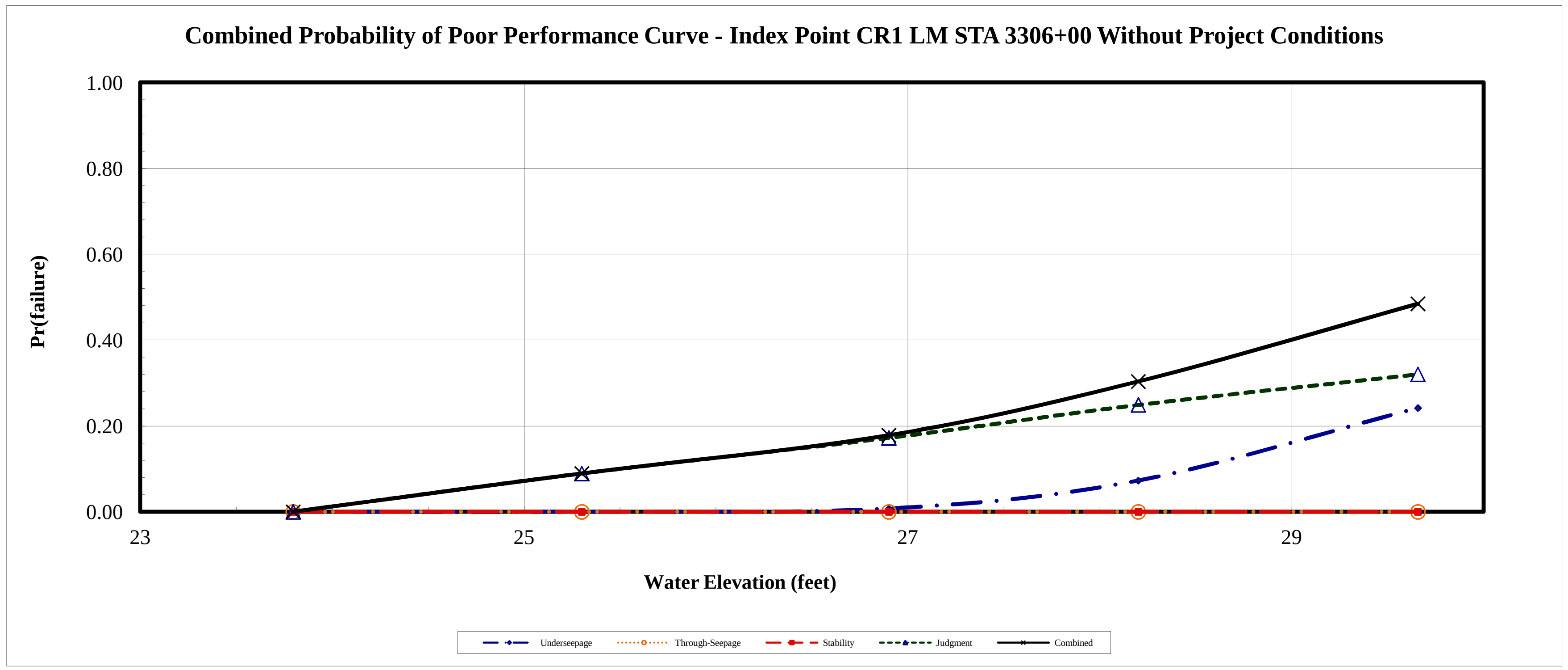
Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point CR1

Levee Mile: STA 3306+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 29.66
L/S Toe Elev.: 23.80
W/S Toe Elev.: 22.90

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/28/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
23.80	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
25.30	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0892	0.9108	0.0892	0.9108
26.90	0.0074	0.9926	0.0000	1.0000	0.0000	1.0000	0.1721	0.8279	0.1783	0.8217
28.20	0.0727	0.9273	0.0000	1.0000	0.0000	1.0000	0.2490	0.7510	0.3036	0.6964
29.66	0.2418	0.7582	0.0000	1.0000	0.0000	1.0000	0.3203	0.6797	0.4846	0.5154



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

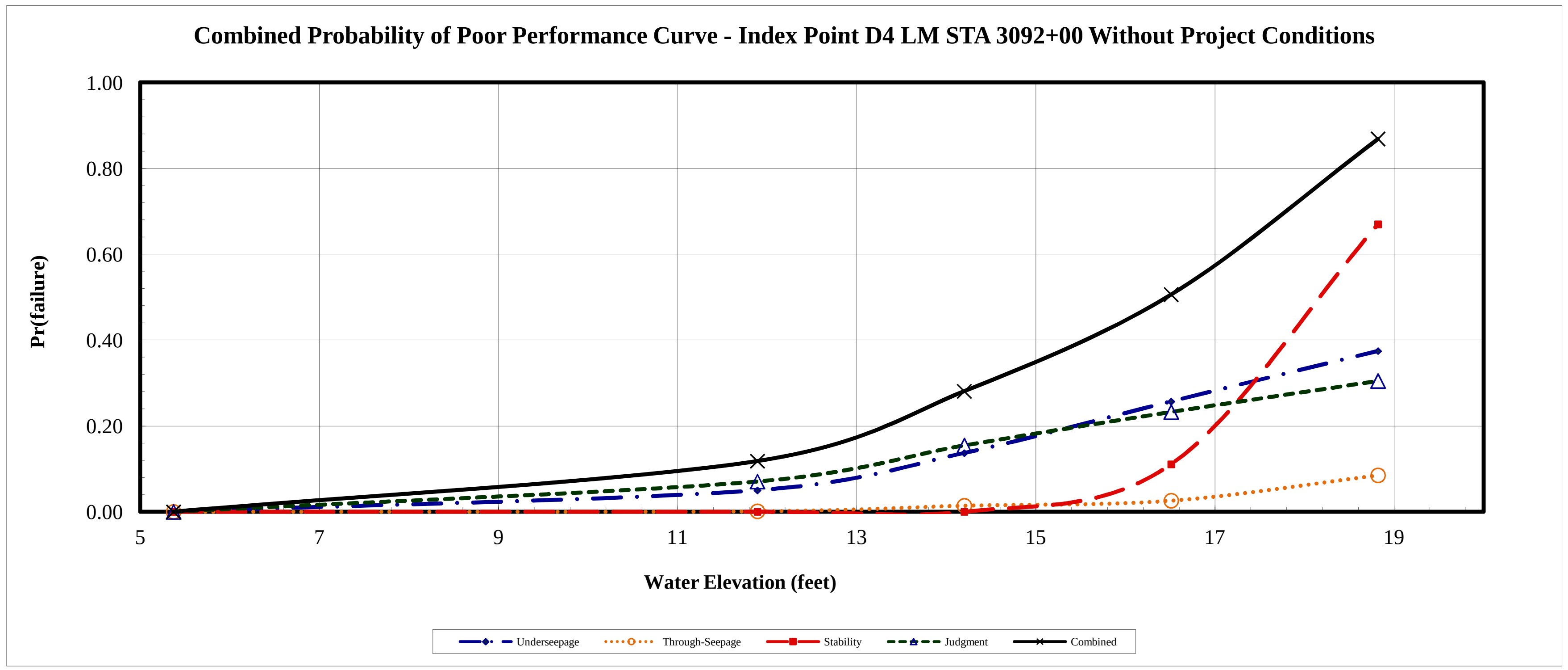
Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point D4

Levee Mile: STA 3092+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 18.82
L/S Toe Elev.: 5.37
W/S Toe Elev.: 3.18

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/25/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
5.37	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
11.89	0.0500	0.9500	0.0013	0.9987	0.0000	1.0000	0.0705	0.9295	0.1181	0.8819
14.20	0.1369	0.8631	0.0143	0.9857	0.0000	1.0000	0.1546	0.8454	0.2809	0.7191
16.51	0.2570	0.7430	0.0260	0.9740	0.1108	0.8892	0.2327	0.7673	0.5062	0.4938
18.82	0.3744	0.6256	0.0851	0.9149	0.6698	0.3302	0.3049	0.6951	0.8686	0.1314



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

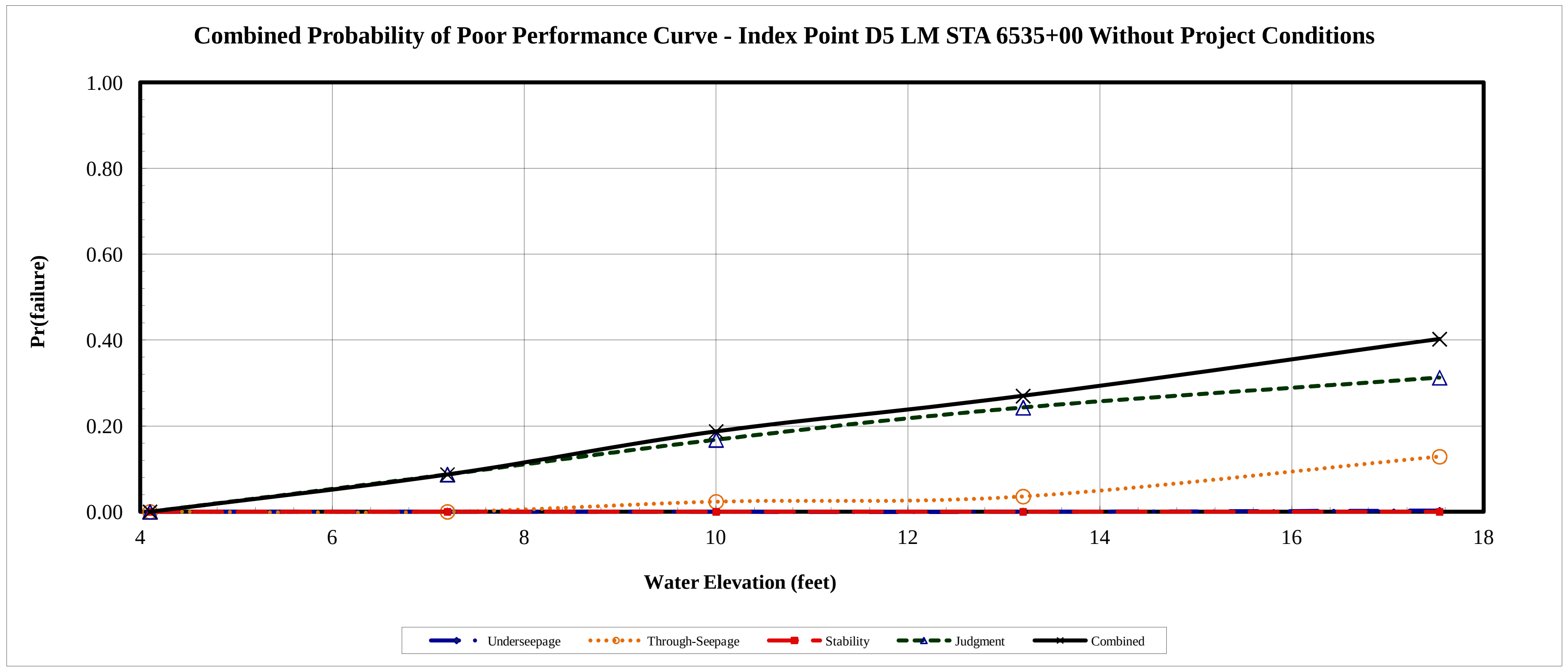
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: Index Point D5

Levee Mile: STA 6535+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 17.54
L/S Toe Elev.: 4.10
W/S Toe Elev.: -6.30

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/19/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
4.10	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
7.20	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0869	0.9131	0.0869	0.9131
10.00	0.0000	1.0000	0.0235	0.9765	0.0000	1.0000	0.1677	0.8323	0.1872	0.8128
13.20	0.0001	0.9999	0.0356	0.9644	0.0000	1.0000	0.2427	0.7573	0.2698	0.7302
17.54	0.0028	0.9972	0.1284	0.8716	0.0000	1.0000	0.3124	0.6876	0.4023	0.5977



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

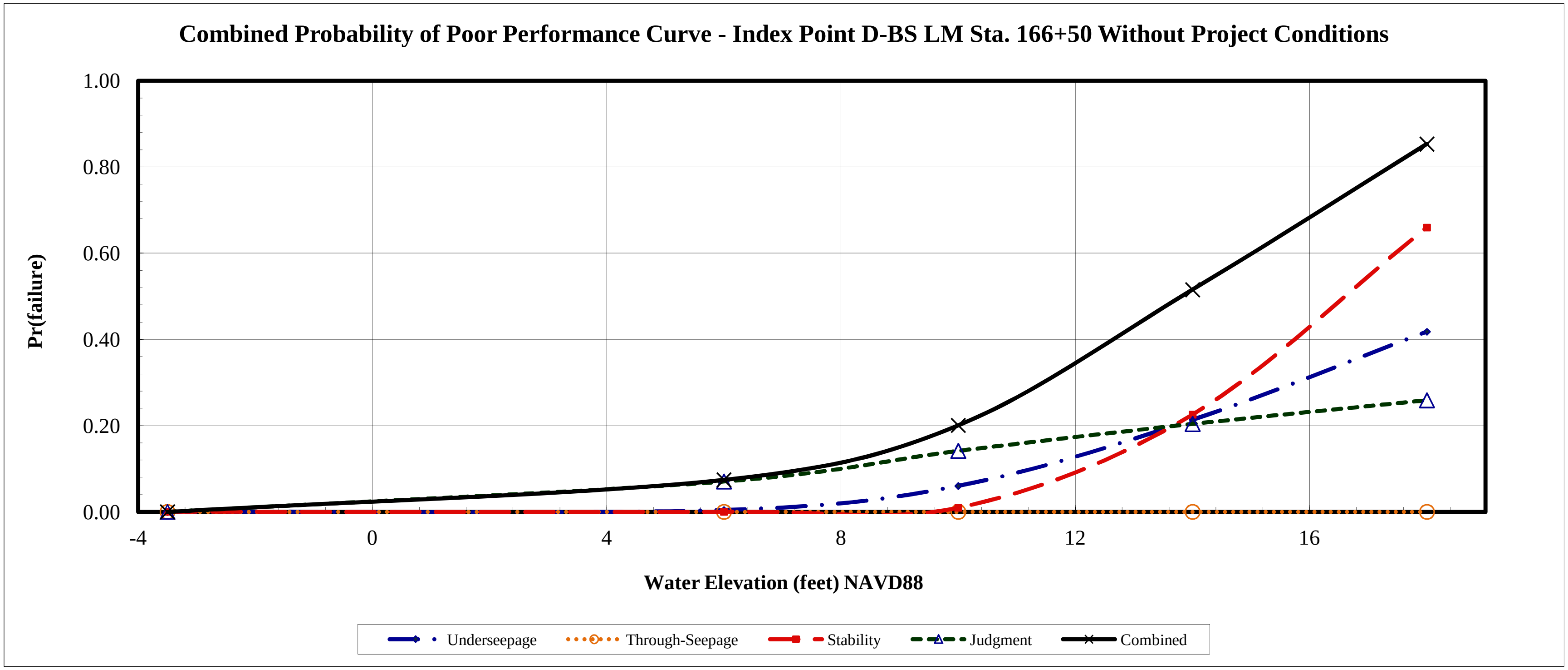
Project: Lower San Joaquin
Study Area: Delta Front Brookside Study Area
River Section: Index Point D-BS
Coordinates: State Plane (ft), N 2183200, E 6311320

Levee Mile: Sta. 166+50
River Mile: XXXX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 18.00
L/S Toe Elev.: -3.50
W/S Toe Elev.: -7.50

Analysis By: G. Johnson
Checked By: J. Hogan, M. Per
Date: 3/14/2013

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
-3.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
6.00	0.0041	0.9959	0.0000	1.0000	0.0000	1.0000	0.0705	0.9295	0.0743	0.9257
10.00	0.0600	0.9400	0.0000	1.0000	0.0094	0.9906	0.1415	0.8585	0.2006	0.7994
14.00	0.2136	0.7864	0.0000	1.0000	0.2256	0.7744	0.2040	0.7960	0.5153	0.4847
18.00	0.4180	0.5820	0.0000	1.0000	0.6597	0.3403	0.2589	0.7411	0.8532	0.1468



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

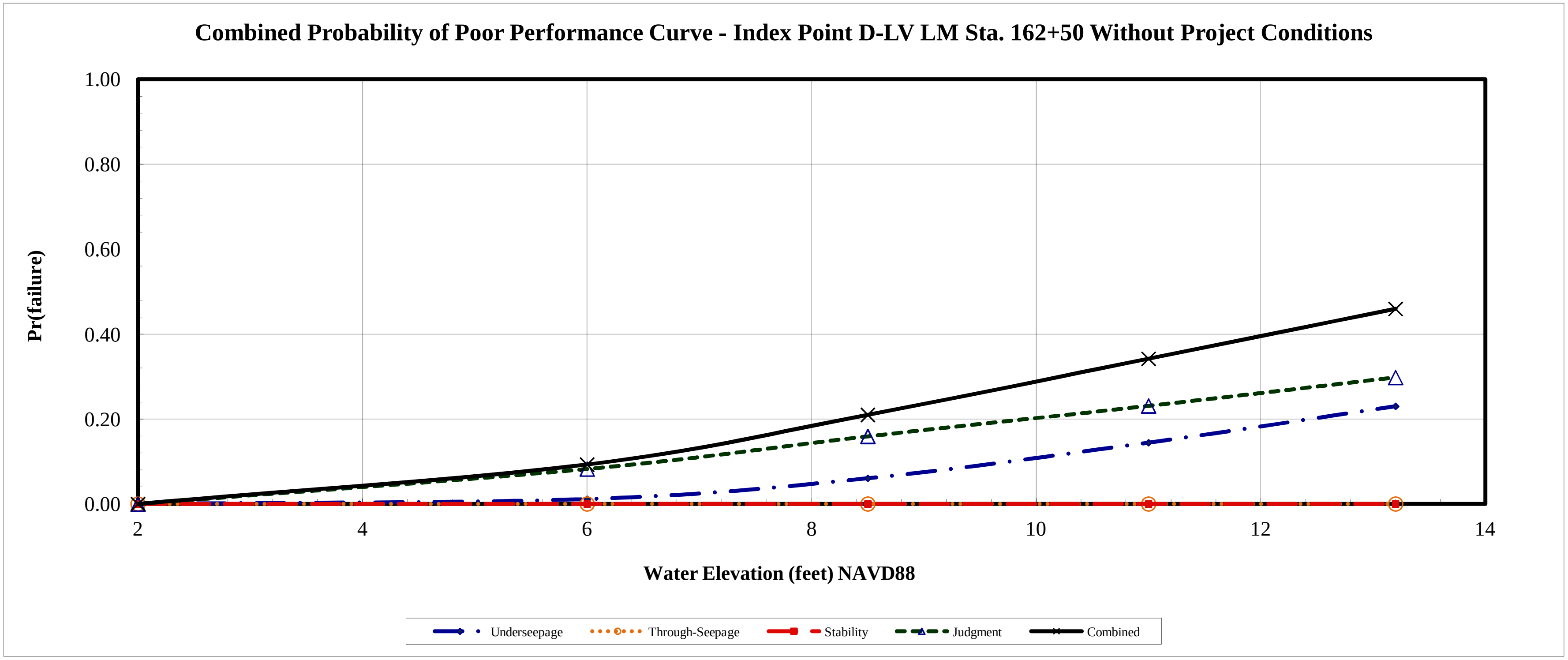
Project: Lower San Joaquin
Study Area: Delta Front Lincoln Village
River Section: Index Point D-LV
Coordinates: State Plane (ft), N 2185939, E 6315555

Levee Mile: Sta. 162+50
River Mile: XXXX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 13.20
L/S Toe Elev.: 2.00
W/S Toe Elev.: 3.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perle
Date: 4/9/2013

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
2.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
6.00	0.0115	0.9885	0.0000	1.0000	0.0000	1.0000	0.0822	0.9178	0.0928	0.9072
8.50	0.0602	0.9398	0.0000	1.0000	0.0000	1.0000	0.1591	0.8409	0.2098	0.7902
11.00	0.1443	0.8557	0.0000	1.0000	0.0000	1.0000	0.2309	0.7691	0.3419	0.6581
13.20	0.2299	0.7701	0.0000	1.0000	0.0000	1.0000	0.2979	0.7021	0.4593	0.5407



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

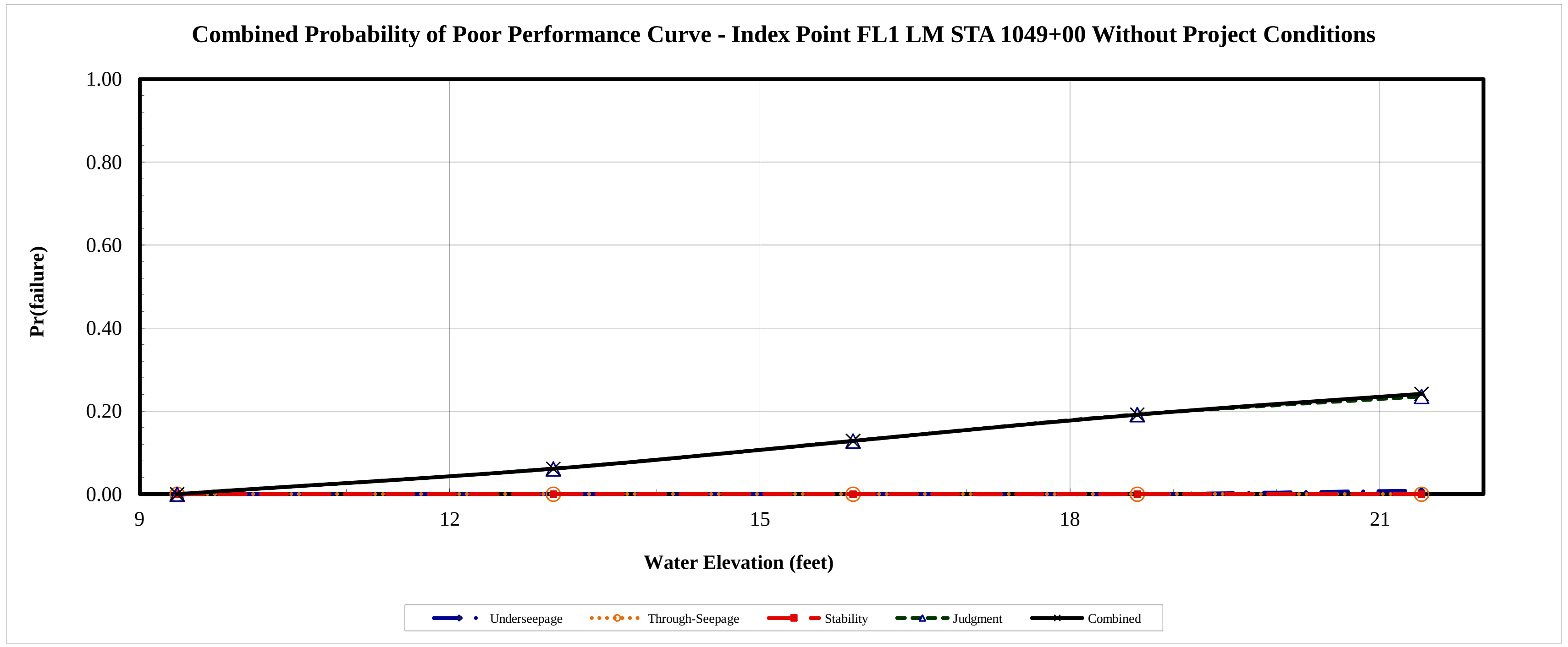
Project: Lower San Joaquin
Study Area: Left Bank French Camp Slough
River Section: Index Point FL1

Levee Mile: STA 1049+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 21.40
L/S Toe Elev.: 9.36
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
9.36	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
13.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0610	0.9390	0.0610	0.9390
15.90	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1282	0.8718	0.1282	0.8718
18.65	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1917	0.8083	0.1917	0.8083
21.40	0.0087	0.9913	0.0000	1.0000	0.0000	1.0000	0.2351	0.7649	0.2418	0.7582



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

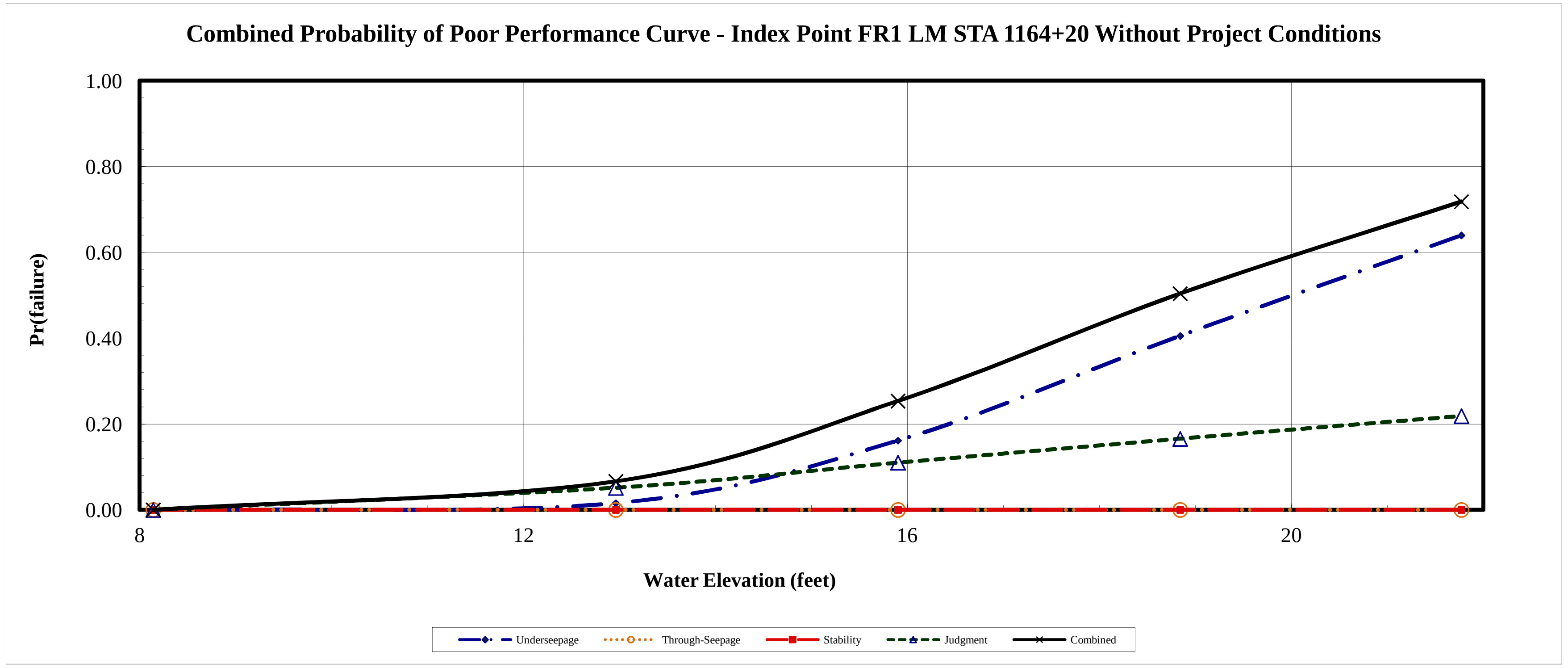
Project: Lower San Joaquin
Study Area: Right Bank French Camp Slough
River Section: Index Point FR1

Levee Mile: STA 1164+20
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 21.77
L/S Toe Elev.: 8.14
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/12/2012
Date: 12/10/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
8.14	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
12.96	0.0157	0.9843	0.0000	1.0000	0.0000	1.0000	0.0514	0.9486	0.0663	0.9337
15.90	0.1615	0.8385	0.0000	1.0000	0.0000	1.0000	0.1099	0.8901	0.2537	0.7463
18.84	0.4054	0.5946	0.0000	1.0000	0.0000	1.0000	0.1656	0.8344	0.5039	0.4961
21.77	0.6396	0.3604	0.0000	1.0000	0.0000	1.0000	0.2185	0.7815	0.7183	0.2817



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR1

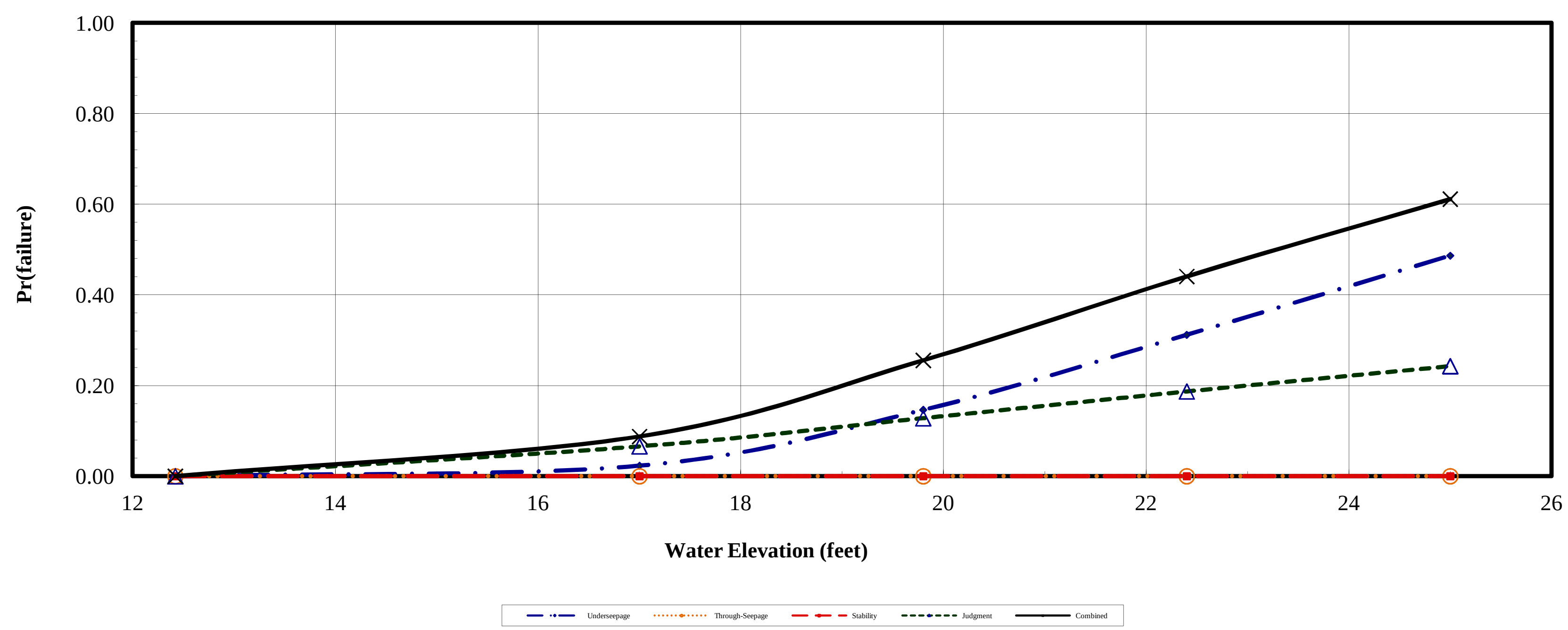
Levee Mile: 1292+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 25.00
L/S Toe Elev.: 12.42
W/S Toe Elev.: 11.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Per
Date: 12/18/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
12.42	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
17.00	0.0234	0.9766	0.0000	1.0000	0.0000	1.0000	0.0657	0.9343	0.0876	0.9124
19.80	0.1465	0.8535	0.0000	1.0000	0.0000	1.0000	0.1280	0.8720	0.2557	0.7443
22.40	0.3121	0.6879	0.0000	1.0000	0.0000	1.0000	0.1870	0.8130	0.4408	0.5592
25.00	0.4868	0.5132	0.0000	1.0000	0.0000	1.0000	0.2429	0.7571	0.6114	0.3886

Combined Probability of Poor Performance Curve - Index Point LR1 LM 1292+00 Without Project Conditions



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

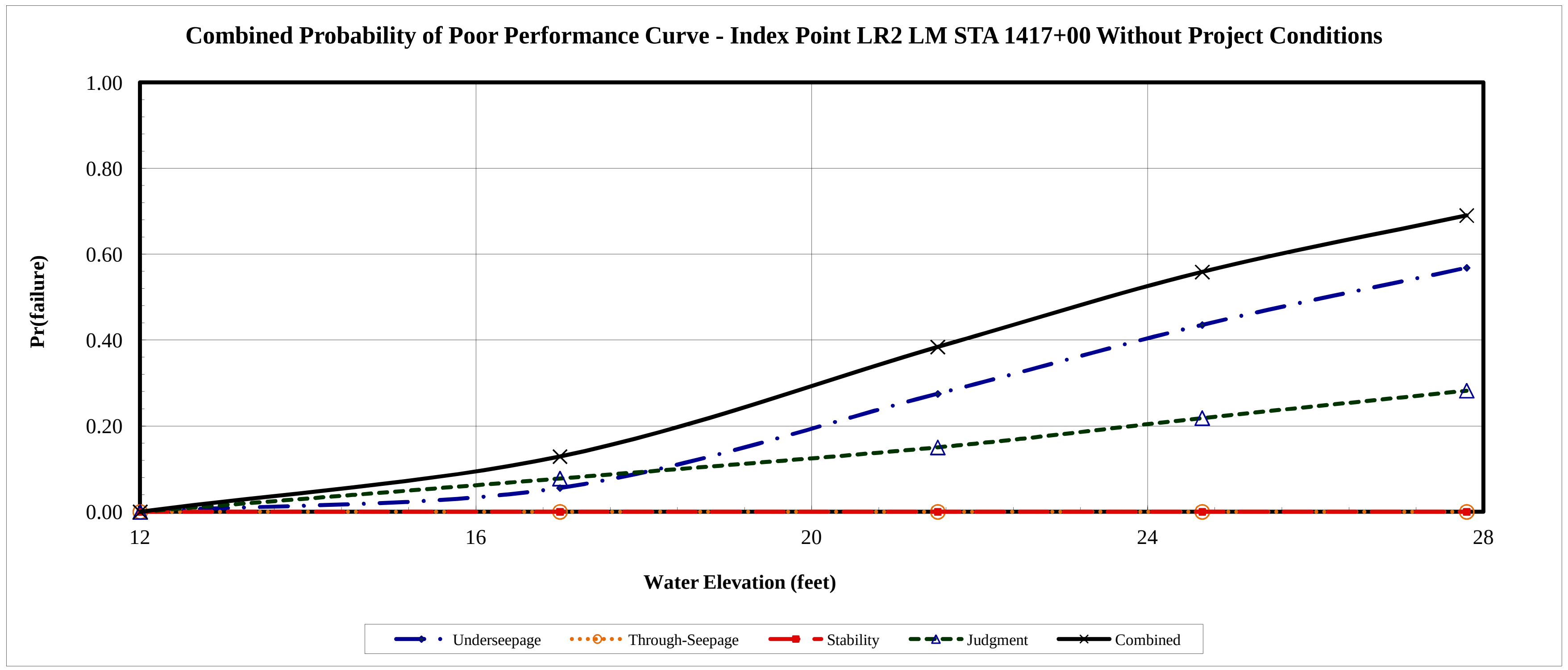
Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR2

Levee Mile: STA 1417+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 27.80
L/S Toe Elev.: 12.00
W/S Toe Elev.: 12.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
12.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
17.00	0.0555	0.9445	0.0000	1.0000	0.0000	1.0000	0.0775	0.9225	0.1287	0.8713
21.50	0.2749	0.7251	0.0000	1.0000	0.0000	1.0000	0.1503	0.8497	0.3839	0.6161
24.65	0.4353	0.5647	0.0000	1.0000	0.0000	1.0000	0.2185	0.7815	0.5587	0.4413
27.80	0.5685	0.4315	0.0000	1.0000	0.0000	1.0000	0.2823	0.7177	0.6903	0.3097



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

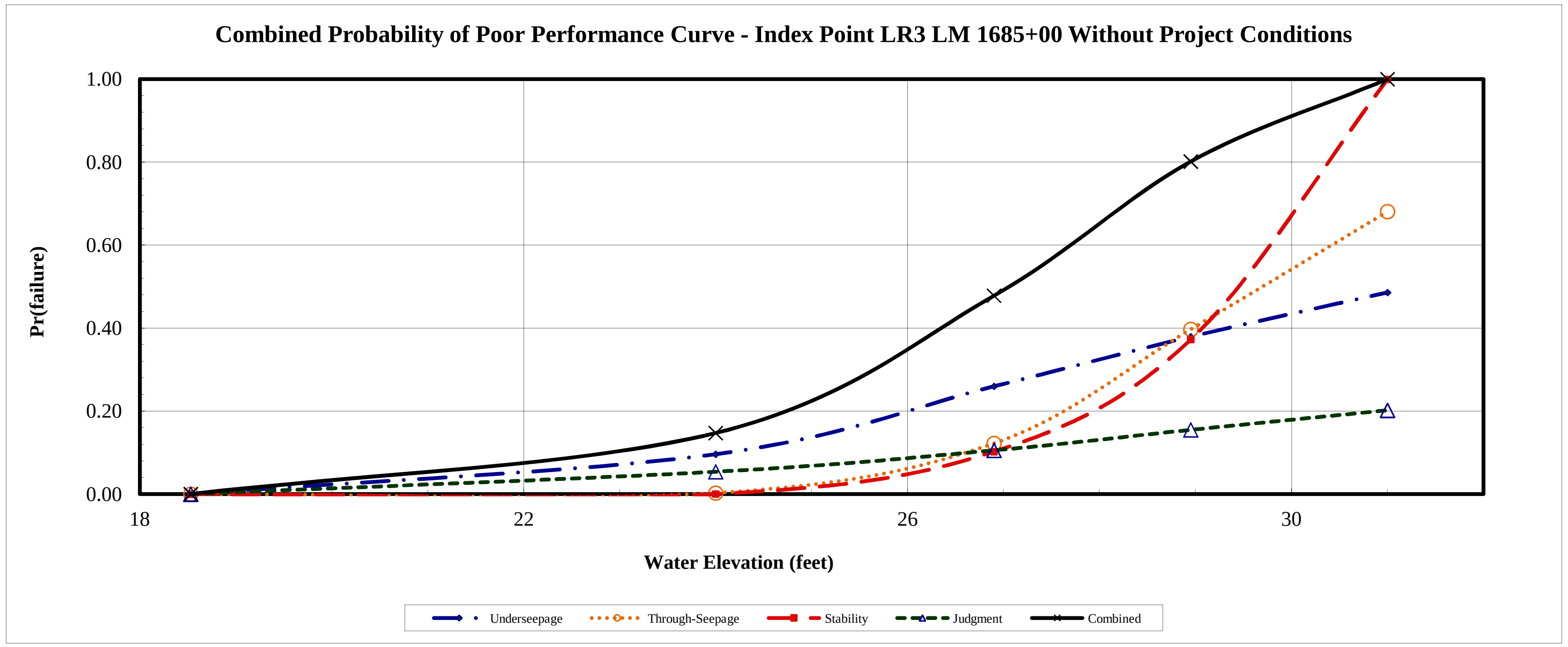
Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR3

Levee Mile: 1685+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 31.00
L/S Toe Elev.: 18.53
W/S Toe Elev.: 17.80

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/19/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
18.53	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
24.00	0.0961	0.9039	0.0026	0.9974	0.0003	0.9997	0.0538	0.9462	0.1472	0.8528
26.90	0.2596	0.7404	0.1222	0.8778	0.1025	0.8975	0.1054	0.8946	0.4782	0.5218
28.95	0.3790	0.6210	0.3971	0.6029	0.3725	0.6275	0.1547	0.8453	0.8014	0.1986
31.00	0.4857	0.5143	0.6809	0.3191	0.9993	0.0007	0.2019	0.7981	0.9999	0.0001



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

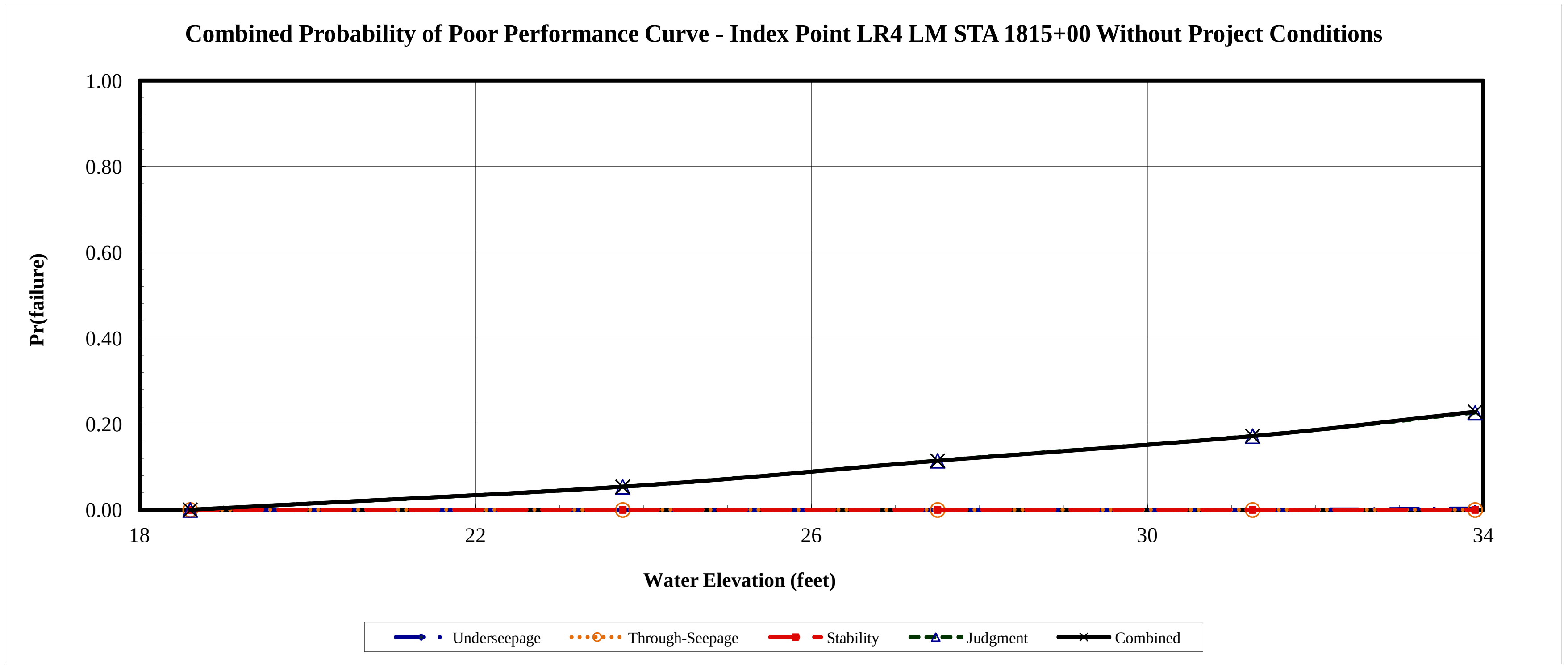
Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR4

Levee Mile: STA 1815+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 33.90
L/S Toe Elev.: 18.60
W/S Toe Elev.: 19.40

Analysis By: G. Johnson
Checked By: M. Perlea 12/13/2012
Date: 12/13/2012

Water Surface	Underseepage		Through-Seepage		Stability		Judgment		Combined	
Elevation	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
18.60	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
23.75	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0538	0.9462	0.0538	0.9462
27.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1144	0.8856	0.1144	0.8856
31.25	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1719	0.8281	0.1719	0.8281
33.90	0.0030	0.9970	0.0000	1.0000	0.0001	0.9999	0.2265	0.7735	0.2289	0.7711



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL2

Levee Mile: STA 976+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 44.56
L/S Toe Elev.: 34.30
W/S Toe Elev.: 34.79

Analysis By: J. Hogan
Checked By: M. Perlea, G. Joh
Date: 9/27/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
34.30	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
37.20	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0514	0.9486	0.0514	0.9486
38.80	0.0002	0.9998	0.0000	1.0000	0.0000	1.0000	0.1008	0.8992	0.1009	0.8991
40.40	0.0062	0.9938	0.0000	1.0000	0.0000	1.0000	0.1481	0.8519	0.1533	0.8467
44.56	0.2245	0.7755	0.0000	1.0000	0.0000	1.0000	0.1934	0.8066	0.3745	0.6255